HAWKE'S BAY WATERWAY GUIDELINES

STORMWATER MANAGEMENT

HAWKE'S BAY REGIONAL COUNCIL

SAFEGUARDING YOUR ENVIRONMENT KAITIAKI TUKU IHO

159 Dalton Street + Private Bag 6006 + Napier
Telephone (06) 835 9200
Fax (06) 835 3601
Regional Freephone (06) 0800 108 838
SAFEGUARDING YOUR ENVIRONMENT + KAITIAKI TUKU IHO

Hawke's Bay Waterway Guidelines

Stormwater Management

Prepared by:
Earl Shaver, Aqua Terra International Ltd

Reviewed by:
Gary Clode, Manager - Engineering

Acknowledgement
This document for the Hawke's Bay Region is based primarily on the Auckland Regional Council's Technical Publication No. 10 "Stormwater Management Devices: Design Guidelines Manual". Other information has been used where it was found suitable for use in the Hawke's Bay Region. The ARC gave permission to use their document and that permission is greatly appreciated.

Modifications to the ARC document have been made so there will be some differences to the ARC approach to account for advances in practice design and to reflect local conditions.

Note
This document is a living document and may be reviewed from time to time as industry standards change and best practice evolves. Please contact Hawke's Bay Regional Council to ensure the latest version is used.

May 2009
ISBN NO 1-877405-30-2
HBRC Plan Number 4104

Contents

1 Introduction

1.1 Objectives of these guidelines

The primary objective of these guidelines is to outline and demonstrate the Hawke's Bay Regional Council's preferred design approach for structural stormwater management devices. Specifically this includes design guidance for water quality and water quantity ponds, wetlands, filtration practices, infiltration practices, bio filtration practices and other practices that may be used.

The guidelines also have the following secondary objectives:

1. To provide the reader with a summary of the principles of stormwater management including an outline of environmental effects and management concepts,
2. To outline the statutory process and introduce the rules in the Hawke's Bay Regional Council related to stormwater discharges,
3. To provide a resource guideline for those involved with the design, construction and operation of stormwater management devices, and
4. To minimise adverse environmental effects of stormwater discharges through appropriate design, construction and operation of stormwater management practices.

1.2 What is the effect of impervious area on stormwater runoff?

Development of the Hawke's Bay Region has changed the character of the natural landform by covering land with impervious surfaces. Houses, shopping centres and office buildings provide places to live and work. Car travel between buildings is facilitated and encouraged by a complex network of roads and carparks. This infrastructure allows the successful operation of the cities, towns and region and encourages social and economic development.

However, this change from natural landforms and vegetative cover to impervious surfaces has two major effects on stormwater:

- Water quantity (urban hydrology)
- Water quality (non point source pollution)

1.2.1 Urban hydrology

Roofs, roads, parking lots, and other impervious areas stop water soaking into the ground, diverting it across the surface and increasing the quantity and rate of water discharging to streams and harbours. Impervious surfaces, compaction of soils and the absence of vegetation reduce the "sponge like" storage capacity of the ground surface, reducing infiltration and the volume of underground water that feeds groundwater resources and stream baseflows. These changes in the hydrological cycle cause flooding, stream erosion, sedimentation and loss of water for abstraction. Flooding and erosion can have direct effects on public safety, while erosion and sedimentation can affect the habitat of aquatic resources.

1.2.2 Non-point source pollution

Particles from car exhausts, tyres and brakes, silt, fertilisers, oils, litter and other by-products of urban life fall and collect on impervious surfaces. Many of these small particles adhere onto sediment which stormwater runoff transports to streams, estuaries and harbours. Where the water is still, these contaminants settle out and accumulate. Other contaminants dissolve as rain passes over them and change the physical-chemical composition of stormwater. The accumulation of sediment, contaminants and changes to the chemical make-up of stormwater affect water quality and can then have significant effects on the viability of aquatic resources.

These effects will be detailed further in Chapter Two.

1.3 Managing stormwater

Stormwater management aims to protect human and ecological values by preventing or mitigating the adverse effects of stormwater quality and quantity on the human and aquatic environment.

Stormwater management techniques are generally divided into:

- Non-structural practices (which prevent changes to the quality and quantity of stormwater by low impact designs, management practices or planning regulations), and
- Structural practices (which reduce or mitigate changes that have already occurred to stormwater by constructed treatment devices).

Non-structural practices may be further categorised into:

- Site design practices which reduce the quantity of stormwater runoff, and
- Contamination control practices, which minimise the risk of contaminants coming into contact with stormwater.

Structural, or treatment, practices assume that the increase in runoff or contamination of stormwater has already occurred and attempt to reduce the contaminants in the stormwater or hold runoff to reduce flooding and erosion.

1.4 Regulatory framework

The RMA defines BPO as:

"Best Practical Option means the best method for preventing or minimising the adverse effects on the environment having regard, among other things, to

(a) the nature of the discharge or emission and the sensitivity of the receiving environment to adverse effects; and

(b) the financial implications, and the effects on the environment, of that option compared with other options; and

(c) the current state of technical knowledge and the likelihood that the option can be successfully applied."

s2(1) Resource Management Act, 1991

The HBRC is empowered under various acts of law to regulate and monitor the use of our environmental resources to ensure these are used wisely and in such a manner that their existence and mauri[1] is assured for time to come. Through the Resource Consent process, project plans, designs and processes are assessed to ensure that degradation to the environment is avoided, remedied or mitigated. To ensure the Resource Consent process is properly followed, the following applies to any person planning works in, around or relating to waterways:

<u>Any person or agency considering an activity concerning the:</u>

a) <u>taking, use, damming or diversion of water, or;</u>
b) <u>the discharge of any contaminant into water or onto land such that it may enter water, or;</u>
c) <u>disturbance of the bed of a river or lake, or;</u>
d) <u>erection, maintenance, replacement, upgrade, placement or removal of any structure in, on or under the bed of a river or lake</u>

<u>is advised to seek advice from the Regional Council at the earliest possible stage in planning regarding Resource Consent requirements.</u>

The Appendix has Hawke's Bay Regional Council rules related to Stormwater Discharges to Land/Water.

1.5 Technical Objectives

This guideline provides information on the selection and design of structural stormwater management devices. The primary objectives therefore relate to the removal of contaminants from stormwater, reducing peak discharges, and reducing site runoff by volume control. However, prevention is better than cure. To fully meet stormwater objectives set by the Hawke's Bay Regional Council will require stormwater management solutions that are integrated with development and all opportunities should be taken to prevent and minimise stormwater effects.

The Hawke's Bay Regional Council's objectives for managing stormwater are:

1.5.1 Water quantity

The primary water quantity objective of treatment devices is to match the pre-development and post-development peak flow rates for the 50%, 20%, 10%, and 1% Annual Exceedence Probability (AEP) rainfall events.

Where significant aquatic resources are identified in a freshwater receiving environment, additional water quantity requirements may be required.

1.5.2 Water quality

The primary water quality objective of the treatment devices in this guideline is to remove 75% of total suspended sediment on a long-term average basis. Removal of sediment will remove many of the contaminants of concern, including; particulate trace metals, particulate nutrients, oil and grease on sediments and bacteria on sediments.

[1] **Life force.**

1.5.3 Aquatic resource protection

Aquatic resource protection is primarily concerned with maintaining the physical structure of the receiving system while promoting practices that provide habitat conditions conducive to a healthy ecosystem in receiving environments.

Designing for the detention, storage, and release of 1.2 times the water quality rainfall over a 24-hour period reduces physical structure effects.

Other practices include riparian vegetation maintenance or enhancement and a reduction in the volume of runoff through revegetation and use of roof runoff for domestic water purposes.

It is important to note that these are objectives only. They are not standard requirements. There will be situations where alternative approaches or design requirements may be appropriate.

Their application depends upon whether the stormwater issue they address is present and the degree of implementation depends upon site and catchment circumstances as determined by the Best Practicable Option. For example water quantity objectives are unlikely to be required where stormwater is discharged to an open coastal environment where erosion, sedimentation and flooding issues are not present. While water quality is a significant issue in urban areas, the degree to which the water quality objectives are implemented depends on the practices that can be retrofitted into the available space. The same issues also apply to aquatic resource protection.

In addition, the approval by the Hawke's Bay Regional Council of a catchment management plan for specific catchment that has been submitted by a local authority may provide for alternative requirements that have been defined through a catchment-wide analysis. Proposed individual developments should investigate whether an approved comprehensive catchment plan exists for a given catchment, and if so, should ensure that development is in accordance with that plan.

1.6 Structure of these guidelines

These guidelines are divided into the following chapters as follows:

Chapter One:	Introduction
Chapter Two:	Effects of Land Use on Stormwater Runoff
Chapter Three:	Receiving Environments
Chapter Four:	Stormwater Management Concepts
Chapter Five:	Choosing a Stormwater Management Device
Chapter Six:	Hydrology and Water Quality
Chapter Seven:	Detailed Stormwater Management Practice Design
Chapter Eight:	Landscaping
Chapter Nine:	Outlet Design
Chapter Ten:	Innovative Practices

Chapters 1 - 5 aim to provide all users with an introduction to the regulatory framework, effects of stormwater and the range of management concepts applicable to the Hawke's Bay Region.

Chapter 6 describes the hydrologic approach to stormwater management in the Hawke's Bay Region.

Chapter 7 provides detailed design procedures for the various practices contained in these Guidelines.

Chapter 8 provides discussion on landscaping to enhance site appearance and public acceptability.

Chapter 9 discusses the design of outlet structures to ensure that erosion does not occur at the outlet of stormwater management practices.

Chapter 10 relates to new practices and establishes a framework for the assessing performance expectations of new practices and the level of testing that is required for their widespread use in the Region.

1.7 Statement of intent

Applicants may propose alternative designs that meet the requirements of the Hawke's Bay Regional Plan, and the Hawke's Bay Regional Council will assess whether the design will achieve the Plan's goals and objectives.

In addition, this Guideline is being distributed primarily in digital format. One reason for that approach is the recognition that updates may be necessary due to increased knowledge relating to investigations or criteria changes both here and overseas. It is the intent of the Hawke's Bay Regional Council to update this Guideline whenever changes are warranted. Distribution can then be done more easily by posting changes on the Hawke's Bay Regional Council website.

2 Effects of Land Use on Stormwater Runoff

2.1 Urbanisation

2.1.1 The hydrological cycle

Water moves constantly between the atmosphere, ground and water bodies in an ongoing, worldwide cycle; the hydrological cycle. Processes such as rainfall, infiltration runoff, evaporation, freezing, and melting, continually move water between different physical phases, across regions, between fresh and saline waters and into the atmosphere. Some processes, such as freezing in polar areas or deep infiltration to slow aquifers, may keep water in one part of the cycle for long periods of time. All the time though, water is moving through the cycle.

Stages of Urban Land Use
Bush, Rural, Lifestyle, Urban

The total volume of water in the cycle is finite. The amount of water vapour in the atmosphere plus the amount of rainfall, freshwater, ground water, seawater and ice on the land is constant. Over time, physical factors such as climate or landform may change the volume of water at each stage in the cycle or sub-cycles, but in total no water leaves or enters the cycle.

Restricting the movement of water in one stage of the hydrological cycle will proportionally increase its movement in another. This occurs during urbanisation. The photographs above show the typical phases of urbanisation; through bush, pasture, subdivision and mature urban land use. In a natural state, bush, trees and grass cover a catchment, which intercept rainfall and let it infiltrate into the ground.

Urbanisation creates impervious surfaces, which reduce vegetative interception, depression storage, infiltration and surface roughness (flow retardation). The excess water now runs off more quickly and increases the flow rate and volume of stormwater for a given storm event.

To illustrate these changes, Table 2-1 gives estimates of the proportion of movement by each process before and after development. These figures represent typical proportions for non-volcanic soils.

Table 2-1
Components of a Typical hydrological cycle

Component	Pre-development (mm)	Post-development (mm)
Annual rainfall	1200	1200
Total runoff	320	700
Deep infiltration	60	10
Shallow infiltration	300	100
Evaporation/transpiration	520	390

2.1.2 Non-point source pollution

Impervious surfaces also collect contaminants derived from everyday urban life. These could be anything from litter, dust, decomposing vegetation or oils, to exhaust emission particles. Roads, in particular, collect by-products from vehicle wear and tear and combustion by-products. In the context of stormwater management and this guideline, these by-products are all termed "contaminants."

Typical Example of Street Runoff

Stormwater runoff moves contaminants off impervious surfaces, through drainage pipes and into water bodies. Litter and larger particles are washed off directly while the (very small) contaminant particles attach more to fine silt and clay particles and become readily transportable. Heavier particles drop out of suspension close to the ends of stormwater pipes while finer silts settle and accumulate further away in still, sheltered sections of water. This accumulation of contaminants from wide areas of developed urban land is termed "non point source" pollution.

The effects of non-point source pollution are diverse. Persistent contaminants such as metals and toxic organics accumulate in sediment and have toxic ecological effects. Other contaminants such as sediment physically affect habitat, for example by smothering.

In some cases, these contaminants occur naturally in the environment. However, it is important to remember that impervious surfaces and stormwater pipes collect contaminants together, transport them and allow them to accumulate in places that they would not normally end up, and in much higher volumes and concentrations.

2.2 Key effects

Many of the effects of stormwater are only significant when considered cumulatively. The water quality and flooding effects of stormwater from an individual site may be relatively minor. If we consider a 10% increase in peak flow from a 1 hectare site, downstream flood levels may only increase 1 mm or less. However, allowing an increase in flood levels on an individual site basis is an ad hoc approach, which neglects the sum total of all potential development in a catchment. Therefore, in addition to any site-specific effects, stormwater effects must be considered on a cumulative basis.

The three key effects of urban stormwater on the environment are:

1. Water quantity - flooding and erosion risks to humans and their property from altered hydrology and development too close to existing watercourses.
2. Water quality - threats to human health and receiving systems from changes to the physical-chemical nature of water and sediment.
3. Aquatic resources - loss of freshwater aquatic resources due to both altered hydrology and non point source pollution. In particular, this considers the physical effects of stormwater on the freshwater environment.

2.3 Water Quantity

2.3.1 General

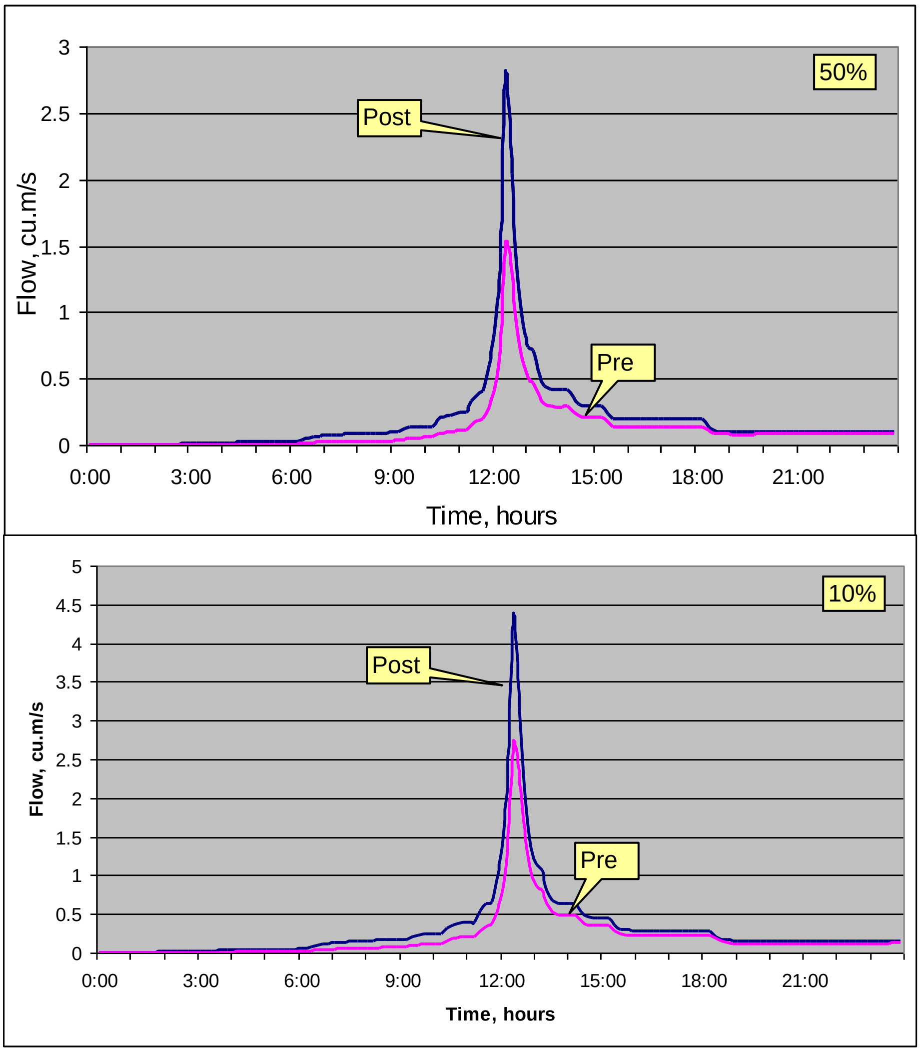
Stormwater drainage systems are generally designed for a moderate level of performance and adopt approximately a 10% AEP event for pipe sizing. However, the importance of more severe, less frequent events is acknowledged and allowance is made for overland flow paths for events up to 1% AEP. These two systems are termed the primary and secondary drainage systems. To protect the public and their property, habitable building floor levels are required to have a contingency freeboard above the 1% AEP flood levels.

Flooding adjacent to waterways naturally occurs but urbanisation can increase flood potential due to either a gradual increase in peak flows (as a result of upstream development described in the example below), or, where a constriction in the drainage channel (culvert, pipe drainage system) or stream channel reduces the flow capacity. However, the safe passage of flood flows is not always a case of "making the pipes big enough." Water flow can change with its location along the channel due to changes in topography, channel dimensions, roughness, pools and other factors. The flood level at a given point is therefore determined by how quickly upstream conditions deliver water and how quickly downstream conditions allow it to get away. The equilibrium sets the flood level. However, the flow rate also changes with time, as the flood passes down a catchment. The flood level will therefore constantly change as both the physical- spatial factors and the variation of flow with time balance.

2.3.2 Case study:

Figure 2-1 sets out the predevelopment and post development 50% and 10% AEP hydrographs for a 27.7-hectare residential development, which was previously pasture. The site changed from two houses to 297 lots of about 600 square metres. For average sized houses, garages driveways and subdivision roading, the imperviousness increases from less than 1% to 54%.

Figure 2-1
Pre- and Post-Development 50% and 10% Hydrographs



The hydrographs show that the peak flow rate for the 50% AEP event increases from 1.51 m^3/sec to 2.80 m^3/sec and for the 10% AEP event increases from 2.7 m^3/s to 4.37 m^3/s. The volume of stormwater runoff for the 50% AEP event increases from 10,200 m^3 to16,800 m^3.

Stormwater from the development discharges to a stream. The extra peak flow in the watercourse raises the flood level. The flood level equivalent to the predevelopment 50% AEP event now occurs more frequently, resulting in more frequent bankfull flows. This results in more stream bank erosion.

2.3.3 Examples of effects

1. Extent of flooding

Flood levels are determined by equating the rate of inflow, outflow and available storage. Where the outflow is smaller than the inflow, levels rise. In the adjacent picture the flooding has risen above the stream channel and spread across large sections of land - the natural floodplain.

Increased imperviousness upstream and loss of storage volume, by filling in the flood plain, would make the flood level higher still.

2. Channel constrictions

Channel constrictions such as culverts and bridges are potential flooding points. Constrictions usually include an overland flow path to pass events more severe than the design event and make allowance for blockage.

3. Lack of freeboard

To calculate freeboard and allow for the safe passage of flood flows, the ultimate development scenario upstream must be considered.

The consequences of getting it wrong are evident in the adjacent picture. A further rise in flood level will cause the bridge to become a constriction and raise upstream flood levels significantly.

4. Channel erosion

As bankfull flows increase in frequency with development, the channel erodes to become stable for the increased flow and velocity. As shown, this often results in a wider, “U” shaped channel, the most efficient shape for transporting the flow. During this process, aquatic habitat is lost.

5. Bank slumping

Stream flows are generally deepest and fastest on the outside of a bend. When flow velocities increase, the toe of a bank is often eroded, removing bank support. Eventually, the bank slumps. The recent slump is also susceptible to erosion and, unless stabilised, can keep retreating.

6. Channel Incision

The adjacent picture shows a stream where high velocity and frequent high flows erode the channel base. The clay channel invert here has been cut down 0.5 m to 1.0 m

Channel erosion is a significant source of sediment, which affects water quality and downstream habitat.

2.4 Water Quality

2.4.1 General

Evidence of the effects of urbanisation on water quality may be direct but is often indirect. When considered from a number of perspectives, a clearer picture of effects emerges. Three common methods for observing water quality effects include visual assessment, contaminant level measurement, and biological surveys.

A very simple way to note stormwater effects is to walk along an urban stream and note the changes as the land use changes. Areas with greater levels of imperviousness discharge higher quantities of contaminants and water volumes that quickly change the structure and quality of the stream. Effects are particularly evident where the upper reaches of a catchment are undeveloped. A visual survey can document comparative downstream changes, such as channel erosion locations, fish pass blockages, and areas of sedimentation.

Measuring water or sediment quality chemical parameters for comparison against accepted threshold values can also indicate effects on organisms. A number of studies of such urban runoff have been carried out in Auckland to monitor water quality effects. In addition, a number of biological studies have monitored chemical parameters in-situ and attempted to correlate the contaminant levels against the observed species condition and abundance. There is increasing evidence that catchment development strongly impacts on aquatic resources.

This section presents an introduction to common stormwater contaminants and includes an overview of visual and biological effects that are linked to development and non point source pollution.

2.4.2 What are the contaminants?

(a) Suspended sediments: These are soil, organic particles, and breakdown products of the built environment entrained in stormwater flow. They can be silt sized (63 *u*m) or smaller. Sediments reduce light transmission through water, clog fish gills, affect filter-feeding shellfish, smother benthic organisms, change benthic habitats and fill up estuaries. Larger soil particles above silt sized are also contaminants, but typically exhibit different physical characteristics and settle much more quickly. These particles are sometimes termed “bed load” sediment.

(b) Oxygen demanding substances: These are soil organic matter and plant detritus which reduce the oxygen content of water when they are broken down by chemical action and by bacteria. Chemical oxygen demand (COD), total organic carbon (TOC) and biological oxygen demand (BOD) are three measures of the consumption of oxygen in water. Fish generally need at least 5 g O_2/m^3 to stay alive. A large proportion of fish kills in the Region are caused by spills and oxygen demanding substances such as sewage.

(c) Pathogens: Pathogens are disease-causing bacteria and viruses, usually derived from sanitary sewers. Organisms such as faecal coliform and enterococci are often used as indicators of the presence of pathogenic organisms. However, the presence of an indicator organism does not necessarily prove a pathogen is present; merely that the risk is higher.

Concentrations of indicator organisms in stormwater in the pipe before discharge may exceed Ministry of Health guidelines for contact recreation and shellfish collection. However, dilution with receiving waters will usually mean public health criteria are not exceeded.

(d) Metals: A variety of trace metal compounds are carried in stormwater in both solid and dissolved forms. The most commonly measured metals of concern are zinc, lead, copper and chromium. Metals are persistent; they don't decompose and they accumulate in sediments, plants and filter feeding animals such as shellfish. Elevated levels of metals cause public health issues and organisms avoid the affected habitat area (leading to a reduction in the number and diversity of fauna.) At higher levels still, intergenerational deformities and tumours may occur, as has been recorded overseas.

(e) Hydrocarbons and oils: The hydrocarbons in stormwater are generally those associated with vehicle use. They may be in the form of a free slick, oil droplets, and oil emulsion, and in solution or absorbed to sediments.

(f) Toxic trace organics and organic pesticides: A large range of trace organic compounds has been found in stormwater in Auckland. Polyaromatic hydrocarbons (PAHs) are one major group. PAHs are a group of over 100 different chemicals that are formed during the incomplete burning of coal, oil, and gas. Soot is a good example of a PAH. Organo- chlorine pesticides such as dieldrin, Lindane and Heptachlor constitute another main class of toxic organics.

(g) Nutrients: Nutrients in stormwater are usually nitrogen and phosphorus compounds that stimulate plant and algal growth. This can cause daily fluctuations in dissolved oxygen concentrations, including phases of aerobic decomposition, which removes dissolved oxygen from the receiving waters.

(h) Litter: Litter in stormwater is often referred to as gross pollution. It has a high visual and amenity impact, but limited effect on public health and ecological standards.

In addition to the above contaminants, stormwater discharges have other physical and chemical effects that affect aquatic organisms and change how contaminants react. These include changes to temperature, pH, dissolved oxygen, alkalinity, hardness and conductivity.

2.4.3 Measurement of water quality effects

The concentration of contaminants in stormwater varies during a storm, from storm to storm, and from catchment to catchment. The event-mean concentration (EMC) is a measure of the average pollutant concentration during a storm. It is the pollutant load for the storm divided by the volume of runoff and will vary from storm to storm. The variation of pollutant concentration with time through a storm is termed a pollutograph.

When comparing concentrations with water quality criteria, it should be borne in mind that individual samples may exceed the EMC by a large factor. Exceeding water quality guidelines does not necessarily lead to effects on the environment. An EMC value in stormwater may exceed water quality guidelines "in pipe" but may not following dilution in receiving water. Water quality criteria are therefore more often used as an indicator of receiving environmental health rather than a regulatory standard.

Once contaminated sediments accumulate, their effect depends on factors such as spatial distribution, duration of exposure, dilution from deposition with cleaner sediments, and the rate at which the contaminants are assimilated (bioavailability) by organisms in the receiving environment.

Pollutant toxicity is described as chronic (effects are the result of a gradual accumulation over time) or acute (effects are the result of a sudden pulse).

2.4.4 Examples of effects

The following photographs illustrate the issues discussed.

1. Stream contaminants

The adjacent picture shows the urban stream water and sediment quality in an inner city stream. Effects include litter, inorganic material, some turbidity in the water column, vegetative detritus, and sediments.

The adjacent picture is a close-up of the same environment.

2. Sediment

Sediment from urban land uses and stream channel erosion often settles in estuaries. Low velocities and the saline environment assist particulate settling. Continual sediment delivery reduces light penetration and prevents plant food sources growing in the estuary, thereby

affecting bottom dwelling organisms such as worms, crabs and shellfish, the base of the marine food web.

3. Litter

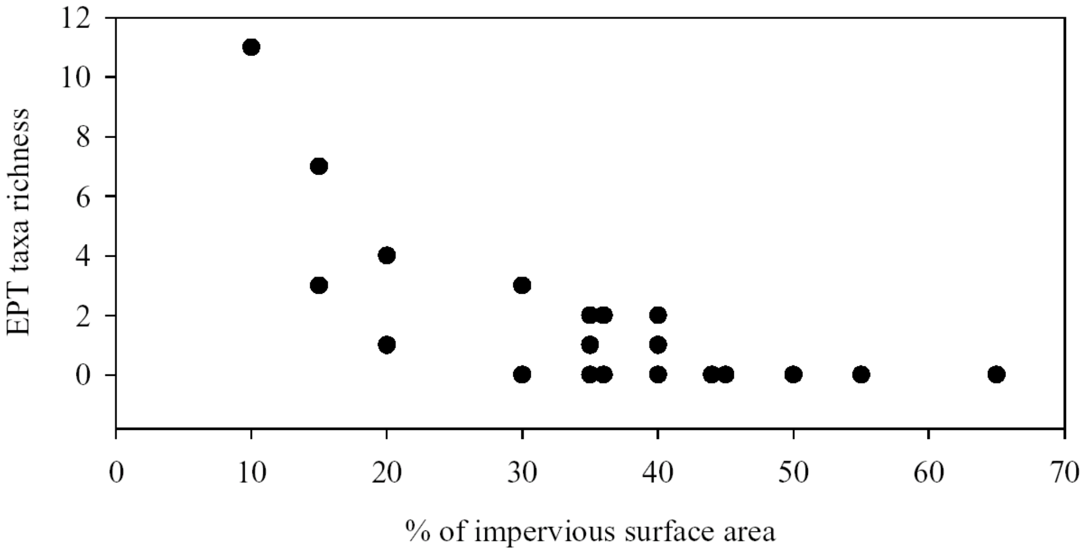
Stormwater systems typically receive inflow via a catchpit. “Back entry” catchpits have a slot set into the kerb behind the grate to improve the hydraulic capacity. However, the size of the slot (50 mm minimum) is sufficient to pass pieces of litter into the stormwater system and water bodies. The adjacent picture shows debris trapped in culvert bars.

Litter will then travel downstream from where it is generated and is an obvious example of how far stormwater pollutants may travel. Litter affects recreational amenity values and may compromise species habitat.

4. Benthic community health

Benthic species are creatures living in aquatic bottom sediments. Figure 2-2 gives an indication of benthic community health related to percentage of impervious surfaces in a catchment. While this specific graph is from Auckland, similar results have been obtained in the U.S., which would indicate that trends elsewhere would be expected to be similar. Clearly, greater levels of imperviousness adversely impact on sensitive aquatic insects.

Figure 2-2
Sensitive Aquatic Organisms versus Impervious Surface Percentages



2.5 Aquatic habitat

2.5.1 **General**

Stream health is affected by all the water quality and water quantity factors that have been discussed in the previous sections. Hydrological factors are thought to be key factors in causing sedimentation and erosion of physical stream structure. However, it is very difficult to identify the combination of different factors that cause specific problems in stream health. Surrogate indicators are therefore used to indicate stream health.

One form of life that exists in streams is macroinvertebrates. Macroinvertebrates are aquatic insects that include grazers, shredders, collectors/browsers, piercers, suckers, and filter feeders on detritus and predators. The presence of a diverse macroinvertebrate community indicates consistently good water quality and a stable stream structure (available habitat). Any alteration of either of these parameters will be reflected in the macroinvertebrate community. So where they are present, they are extremely valuable.

Fish are another barometer of health with their absence or presence providing a picture of the overall health of a stream. Typical fish found in the Hawke's Bay Region streams include banded kokopu, inanga, common bully, as well as eels and freshwater crayfish.

The increased frequency and magnitude of peak flows destabilises stream banks and increases sedimentation. Sedimentation can smother stable and productive aquatic habitats such as rocks, logs, and aquatic plants. The roots of large trees are undercut and fall into the stream while new growth has less opportunity to become established. Deliberate removal of vegetation exposes soil on stream banks, a common feature of urban streams that makes them more vulnerable to erosion. The structural stability of the stream channel has a significant effect on the health of the aquatic ecosystem.

Horner (2001) assessed the effectiveness of structural practices at protection of stream aquatic resources from a catchment-wide perspective. Horner makes a number of interesting statements although they need to be further documented. Key findings were:

- Until catchment total impervious area exceeds 40%, biological decline was more strongly associated with hydrologic fluctuation than with chemical water and sediment quality decreases. Accompanying hydrologic alteration was loss of habitat features, like large woody debris and pool cover, and deposition of fine sediments.
- Structural stormwater management practices at current densities of implementation demonstrated less potential than the non-structural methods (riparian buffers, vegetation preservation) to forestall resource decline as urbanisation starts and progresses. There was a suggestion in the data, though, that more thorough coverage would offer substantial benefits in this situation. Moreover, structural BMPs were seen to help prevent further resource deterioration in moderately and highly developed catchments. Analysis showed that none of the options is without limitations, and widespread

landscape preservation must be incorporated to retain the most biologically productive aquatic resources.
- Structural BMPs can make a substantial contribution to keeping stream ecosystem health from falling to the lowest levels at moderately high urbanisation and, with extensive coverage, to maintaining relatively high biotic integrity at light urbanisation.

The following pictures and text detail aquatic resource impacts related to stream channel modification, barriers to migration, and sedimentation.

<u>1. Stream structure</u>

Urban streams are often straightened and “improved” to increase the hydraulic capacity as seen in the adjacent picture. This process removes habitat such as stream meanders, pool/ riffle structures. Food sources from in stream vegetation and sediments are lost.

<u>2. Barriers</u>

Culverts, weirs and other in-stream structures form barriers to fish passage. This culvert is above the base flow water level preventing fish migration. Climbing fish species cannot pass through the culvert because it overhangs the stream and the shallow depth of water inside the pipe gives high velocities. The culvert shown has also caused channel and stream bank erosion, producing turbulence, which discourages migration by slow swimming fish species.

3. Sedimentation

Low flowing sections of streams are susceptible to sedimentation as seen in the adjacent picture. This can remove habitat in a similar way to channel lining, by infilling pool and riffle stream stretches and smothering food sources and bottom dwelling animals.

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3 Receiving environments

Having an awareness of where water goes and the sensitivity of receiving systems will determine, to a large extent, requirements for stormwater management. For the most part, people don't think of where contaminants go once they leave a site other than they "go away". Having a greater understanding of where water drains to and the recognition that those receiving systems have value, are threatened and require a greater level of protection should improve awareness and action.

Receiving systems include the following systems:

- Streams and rivers
- Ground
- Estuaries
- Harbours
- Open coasts
- Lakes

Each of these systems will be discussed individually to provide context for their value.

3.1 Streams and rivers

Streams and rivers provide a means of conveyance of stormwater from the tops of catchments to lakes, estuaries, harbours and open coast areas.

As water in streams and rivers only moves in one direction (down hill) there is a constant loss of organisms and materials to the sea. The stream and river community is totally dependent on materials entering the system from mostly terrestrial ecosystems, typically as particulate matter (leaves, organic and inorganic matter). As a result, different streams and reaches of streams have different aquatic communities. Upland, fast-flowing streams with stony beds differ from slow-moving lowland rivers with muddy bottoms.

Example of a Hawke's Bay Stream Feature

The dynamic nature of wet-weather flow regimes and water quality make it difficult to assess the impact of urbanisation and stormwater on aquatic ecosystems. The best way to determine whether a given stream or river is healthy is to consider two main components of stream systems:

- Habitat
- Biology

Urbanisation destabilises stream and riverbanks and increases sedimentation and transport of urban contaminants into streams. Sedimentation can smother bottom dwelling organisms and increased sun light can increase stream temperatures. Ecosystem function and quality increases with increased complexity, and the more complex the habitat, the more complex the ecosystem functions.

Biology in streams and rivers includes the following:

- Periphyton – algae, bacteria and fungi that covers the bottom of slow moving streams and blue-green and filamentous green algae that flourish in hard rocky substrates that provide firm footing.
- Macrophytes – plants that are usually rooted and mostly submerged or floating. Macrophytes act as a physical surface for periphyton and insects.
- Benthic macroinvertebrates – bugs that process and utilise the energy entering streams from either organic materials or waste from human or animal sources. Macroinvertebrates are an excellent means to assess stream health, as certain species only exist where there is good water quality.
- Freshwater fish – Absence or presence of fish may provide a picture of overall health of a stream or river. Absence of fish from a stream or river could be related to barriers to fish passage downstream, habitat loss or water quality issues.

The main factors influencing stream and river biology include:

- Physical habitat
- Temperature
- Dissolved oxygen
- Suspended sediments
- Stream flow
- Nutrients
- Light
- Contaminants
- Instream barriers
- Loss of riparian vegetation

Stable Stream with a good Riparian Cover

In urban streams and rivers it is generally hard to ascribe a specific reason for poor biology, as it often is a combination of most of the factors contained in the above list.

For projects that drain to them, the main issues of concern relate to both water quantity and water quality. Depending on the location of the project in a catchment peak flow control may be an issue. In addition stream channel physical structure may be a concern and consideration given to either extended detention or reducing total volume of stormwater flows by either infiltration or evapotranspiration.

Water quality is also a concern on urban stormwater discharges on streams and rivers and will generally be an issue that must be considered and mitigation provided in regional plans.

Hawke's Bay is home to seven major rivers and tributaries (Wairoa, Mohaka, Esk, Tutaekuri, Ngaruroro, Tukituki and Waipawa), which have ecological, social and economic values.

The quality of these rivers is affected by local human activities such as agricultural runoff and point source discharges from farming activities, oxidation ponds and industrial sites. Monitoring of these rivers has shown the average nutrient concentrations and water clarity of most Hawke's Bay REC types are often below national averages. (Hawke's Bay Regional Council, October 2005)

3.2 Ground

There are two issues related to ground and potential contamination.

- Contamination of soils
- Migration of contaminants to groundwater

3.2.1 Contamination of soils

Contamination of soils can occur as a result of past or present land use of a given site that could include:

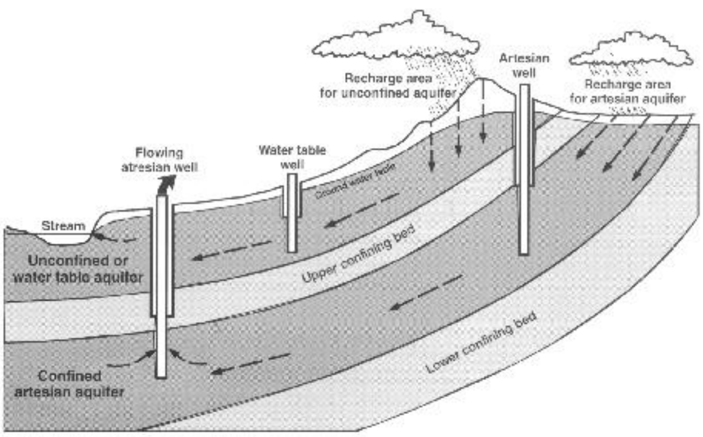
- Use of agricultural chemicals (particularly glasshouses, orchards, vineyards, market gardens)
- Disposal of wastes
- Accidental spillage or leakage of chemicals
- Storage or transportation of raw materials, finished products or wastes
- Migration of contaminants into a site from neighbouring land, either as vapour, leachate or movement of liquids through the soil.

Land where contaminants are present in the soil, sediment, groundwater or surface water could indicate a short or long-term risk to human health and the environment. Impacts on human health from contaminated soil can arise from ingestion of soils, consumption of vegetables from the site, uptake and subsequent bioaccumulation by plants and animals.

Impacts on the environment can occur from a number of routes including direct uptake of contaminants by plants and animals, or migration of contaminants to ground or surface waters. Some contaminants, such as copper, are far more toxic to aquatic plants and animals than to humans.

3.2.2 Migration of contaminants to groundwater

Figure 3-1 Groundwater Movement



Passage of water through the ground is part of the water cycle where water soaks into the ground and flows through it to an aquifer. It is mainly derived from rainfall that has soaked into the ground rather than runoff that travels over the ground surface. It can also be derived from water soaking into the ground from streams or lakebeds.

Water that soaks into the ground moves down through soil pores or rock fractures until it hits the water table.

The zone above the water table is known as the unsaturated zone. Below the water table soil pores or rock fractures are fully saturated and the groundwater mainly moves laterally through these pores and fractures.

Groundwater underlies most of New Zealand. However, differences in geology, hydraulic properties of the soil or rock, topography, recharge rates and relationships with surface waters mean that groundwater flow and bore yields are greater in some areas than others.

In terms of contamination of groundwater, most of the groundwater quality in the country is good but there are areas having groundwater aquifers where fractures in bedrock make for rapid infiltration of surface runoff and the potential for transfer of contaminants to groundwater could potentially occur.

Principal concerns relating to groundwater are water quality and groundwater recharge. Poor stormwater runoff quality can contaminate groundwater and increased impervious surfaces can reduce groundwater recharge. While recharge of groundwater can be important (90% of groundwater in the Region is used for irrigation), it is not recommended that infiltration practices accept untreated stormwater runoff for three reasons:

- Potential clogging of the infiltration system,
- Potential migration of contaminants to groundwater, especially during accidental spills, and
- The ground itself is a receiving system and contamination of soils needs to be prevented

The Institute of Geological and Nuclear Sciences Ltd. Reported on groundwater quality in New Zealand (2007) and identified two major national-scale groundwater quality issues:

- Contamination with nitrate and/or microbial pathogens, especially in shallow wells in unconfined aquifers, and
- Naturally elevated concentrations of iron, manganese, arsenic and/or ammonia, especially in deeper wells in confined aquifers

The health-related guideline values for nitrate and indicator bacteria are exceeded at 5% and 20% of the monitoring sites for which indicator data were available, respectively.

Water quantity issues are only indirectly related in that storage of excess runoff needs to be provided if the runoff rate exceeds the rate of infiltration.

The Hawke’s Bay Regional Council has a monitoring network of 68 wells across the Region where water quality is sampled.

3.3 Estuaries

Estuaries are low energy, depositional zones where the sea meets streams and rivers. They tend to be semi-enclosed coastal bodies of water with one or more rivers or streams flowing into them and with a free connection to the sea. Estuaries are often associated with high rates of biological productivity.

From a NZ perspective, estuaries seethe with bacteria, mud worms, crabs, migrating fish, mangroves and oystercatchers. This system has evolved in the mud flats and is vulnerable to time, tide, erosion, contamination and other effects of human activity.

An estuary is typically the tidal mouth of a river and they are often characterised by sedimentation from silts carried from terrestrial runoff. They are made up of brackish water. Estuaries are marine environments, whose pH, salinity, and water level are varying, depending on the tributaries that feed them and the ocean that provides the salinity. There are several types of estuaries:

- Salt wedge – in this situation the river output greatly exceeds the marine input and there is little mixing
- Highly stratified – river outputs and marine input are more even, with river flow still dominant. Turbulence induces more mixing of salt water upward.
- Slightly stratified – river input is less than the marine input. Turbulence causes mixing of the whole water column
- Vertically mixed – river input is much less than marine input, such that the freshwater contribution is negligible.
- Inverse estuary – these are located in areas with high evaporation and where there is no freshwater input.
- Intermittent estuary – this type of estuary varies dramatically depending on freshwater input, and is capable of changing from a wholly marine embayment to another estuary type.

Estuary at Low Tide

Due to estuaries being low energy environments and having a high salinity, they are depositional zones where sediments and contaminants become deposited. Environmental monitoring by the ARC has identified increasing trends of contaminants such as zinc and copper in estuaries and is a cause for concern (ARC, 2004). It needs to be re-emphasised that metals do not decompose. Estuaries are sinks where contaminants accumulate and concentration levels can be expected to increase.

In terms of stormwater management, neither peak flow nor stream erosion are considered concerns and the main issue is water quality. In addition, water quality may relate to a wide range of contaminants.

From a Hawke's Bay context, Hawke's Bay has a number of estuaries, which are considered as valued receiving environments due to their wide range of natural habitats, biological diversity and opportunities for recreational and commercial use.

The Ahuriri Estuary, located on the northern outskirts of Napier, is a remnant of the Ahuriri Lagoon, which was reduced in size in 1931 when Napier Earthquake raised the bed of the lagoon several metres. The Estuary, a wildlife refuge, still hosts significant wildlife habitats and marine fisheries and is now highly valued in terms of ecological and social consequences.

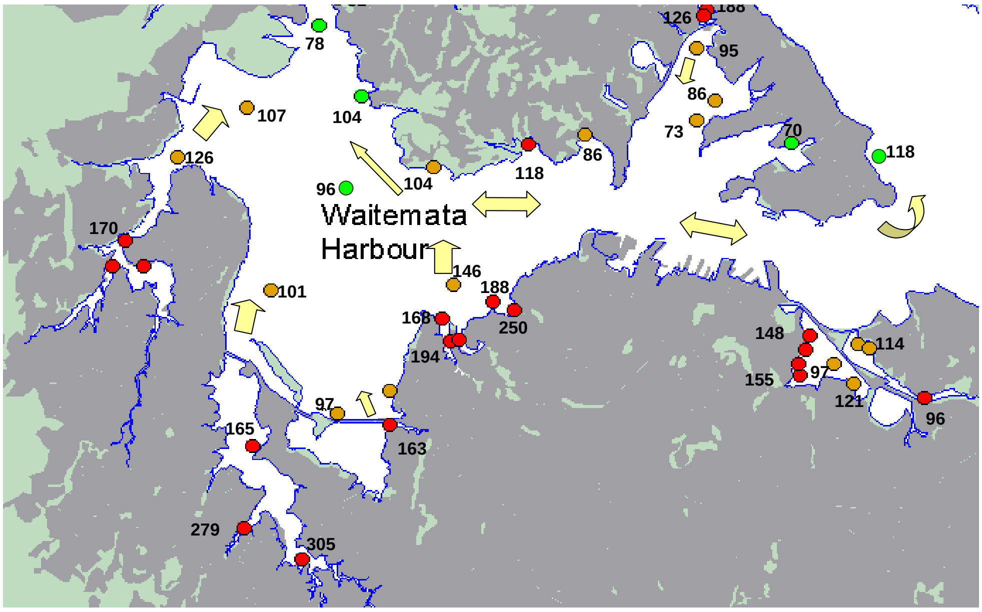
3.4 Harbours

Ahuriri Harbour

Harbours are primarily natural landforms where a body of water is protected and deep enough to furnish anchorage for ships. They differ from estuaries in that tidal action is greater and rates of deposition of sediments are less. Sedimentation does still occur and most harbours of the world require dredging to maintain shipping channels.

New Zealand is fortunate to have a number of good harbours. For the most part they occupy drowned valley systems cut in marine sediments of Miocene Age (15 – 25 million years ago). The following figure shows flow characteristics of the Waitemata Harbour and the relative health of the Harbour (ARC, 2004). The red dots denote areas with high contaminant loads, the orange dots are areas where environmental quality is in transition and the green dots denote healthy areas. As can be seen, the main harbour area is greener with estuarine areas being the most degraded in terms of sediment quality.

Figure 3-2 Contaminant Status of Waitemata Harbour Sediments



From a stormwater management perspective, neither water quantity peak flows nor stream channel erosion is considered as issues needing to be addressed if harbours are the receiving system of concern.

From a water quality perspective, harbours are not as sensitive as estuaries and streams from a contamination standpoint and implementation of stormwater management will probably relate to the magnitude of the project being proposed and the requirements of the regulatory authority.

3.5 Open coasts

Hawke's Bay Coastline

Open coasts are the line of demarcation between the land and the ocean. They are dynamic environments and go through constant change. Natural processes, particularly sea level rise, waves and various weather conditions have resulted in erosion, accretion and reshaping of coasts as well as flooding and creation of continental shelves and drowned river valleys.

Coasts face many environmental challenges relating to human-induced impacts. The human influence on climate change is considered to be a major factor of the accelerated trend in sea level rise. In addition urban development of coastal land, contributes to aesthetic problems and reduced natural coastal habitat.

While not as serious as pollution issues in streams, estuaries or harbours pollution can be an ongoing concern on coasts with garbage and other contaminants littering beaches and coastlines. A large part of the global population inhabits areas near the coast, partly to take advantage of marine resources but also to participate in activities that occur at port related areas.

Depending on littoral drift, the major concern on roading adjacent to open coasts would be litter control. When looking at impacts related to open coasts, a primary concern has been sewage contamination of beaches, which is not a concern with roads. Litter is a visible contaminant and can be addressed through a number of actions including routine cleanup or maintenance.

Hawke's Bay has 333 km of coastline, which includes estuaries, salt marshes, cliff, intertidal rock platforms and sand and gravel beaches.

3.6 Lakes

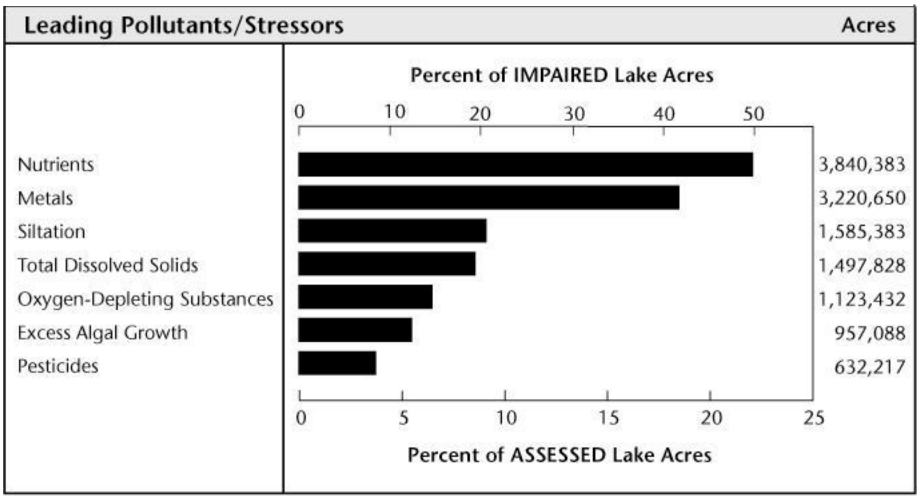
A lake is a body of water that is contained in a body of land and, in the context used here, contains fresh water. Most lakes have an outfall but some do not. Lakes can be manmade or natural. There are a number of natural lakes but most lakes in New Zealand are manmade.

Pollution of lakes can occur through a number of factors. The amount of nutrients entering a lake can cause eutrophication. This is caused by nutrient loadings stimulating excessive plant growth, which in turn decreases the amount of oxygen in the water and eventually causes fish and animal kills. Ecology of lakes is very different from that of streams due to standing water, temperature effects, and contaminant accumulation.

Healthy lakes contain nutrients in small quantities from natural sources. Extra inputs of nutrients (nitrogen and phosphorus) disrupt the balance of lake ecosystems by stimulating population explosions of algae and aquatic weeds. The adjacent picture of Lake Tutira is an example of a lake that has algae problems. The algae sink to the lake bottom after they die, where bacteria decompose them. The bacteria consume dissolved oxygen in the water while decomposing the dead algae. Fish kills and foul odours may result from oxygen depletion. Metals such as copper, zinc, lead, mercury, etc. can also impact on aquatic life by contaminating organisms. By moving up the food chain from worms to insects to fish could then cause a human health problem.

Due to lower horizontal velocities, materials that enter a lake tend to remain in the lake. They are, in effect, sinks where contaminants can accumulate. The following lake information from the U.S. in Figure 3-3 provides an indication of the causes of stressors in U.S. lakes.

Figure 3-3 Causes of Lake Stressors in the U.S.



In a similar fashion to lakes in the U.S., New Zealand lakes are primarily impacted by nutrients. Sediment can also reduce lake clarity but, on road projects, the primary cause of sediment relates to erosion and sediment control during construction rather than sediment-generated post-construction.

NIWA reported on lake water quality (NIWA, 2006) and summarised the current status of 121 lakes. The land use that drained to the lakes was related to four land-cover classes: alpine, native forest/scrub, exotic forest and pasture. Urban land uses were not identified nor considered. NIWA considered phosphorus, nitrogen, clarity, suspended solids and temperature. Median values of total nitrogen, total phosphorus and chlorophyll *a* were four to six times higher in pasture classes than in native bush.

The broad national picture is of high water quality in deep lakes at high altitude and in unmodified catchments, and of lower water quality in modified catchments, especially in small, shallow and warm lakes. Although lake water quality was degraded in both exotic forest and pastureland catchments, pasture use was associated with the worst water quality, most notably in the cases of extreme deterioration.

Extrapolation of the lake environment categories to the nationwide database of 3,820 lakes suggests that approximately 60% of New Zealand lakes are still likely to have excellent or very good water quality; these are lakes in cold regions with high native and low pasture cover. However, approximately 30% of lakes are likely to have very poor to extremely poor water quality. Lowland lakes are especially likely to have poor water quality.

3.7 Overall discussion of Stormwater and Receiving Environments

To put the previous discussion into a context for stormwater management, the following Table 3-1 provides a brief snapshot of receiving environments and stormwater issues. The Table is meant as a general guide and does not substitute for regulatory requirements required by consenting authorities. Contact should be made with the appropriate local council to ensure that any local requirements are complied with.

Table 3-1 Receiving Environments and Stormwater Issues

Receiving system	Flooding issues	Stream erosion issues	Water Quality
Streams	May be a priority depending on location within a catchment	High priority if the receiving stream is a natural, earth channel	High priority
Ground	Not an issue depending on overflow	Not an issue	High priority
Estuaries	Not an issue	Not an issue	High priority
Harbours	Not an issue	Not an issue	Moderate priority
Open Coast	Not an issue	Not an issue	Lower priority
Lakes	Not an issue	Not an issue	High priority

3.8 Bibliography

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Hawke’s Bay Regional Council, Sensitive Receiving Environments in Hawke’s Bay, Report, October 2005.

Auckland Regional Council, Regional Discharges Project Sediment Quality Data Analysis, Technical Publication No. 245, June 2004.

NIWA, Lake Water Quality in New Zealand: Status in 2006 and recent trends 1990 - 2006, Published by the Ministry for the Environment, Publication number: ME-832, December 2006.

4 Stormwater Management Concepts

4.1 Background

When considering the RMA, every person has a duty to avoid, remedy or mitigate any adverse effect on the environment arising from an activity. It also should be recognised that avoidance or remedy are much more cost effective options than mitigation. In the context of highways those three duties can be defined as the following:

4.1.1 Avoid

This includes practices that prevent stormwater becoming contaminated in the first place. Examples include the following:

- Use of building or safety materials or paints that do not leach contaminants, or
- Picking an approach to development that has fewer adverse environmental effects, or
- Reducing the amount of impervious surface that is constructed, or
- New products that do not contain materials that, when wearing down, discharge contaminants

4.1.2 Remedy

Poor Engine Tuning Increasing Contamination

In a similar fashion to avoidance, preventing practices or locations that generate contaminants from coming into contact with stormwater can remedy an existing problem.

Practices that remedy problems are to a large extent associated with non-structural practices. Non-structural practices, such as street sweeping, have been implemented in urban areas to reduce constituent loadings in stormwater runoff, thereby reducing the need for more expensive structural practices. In a study of stormwater characteristics for various land uses in the city of Austin (City of Austin, 1990) constituent median event mean concentrations (EMCs) were reduced in areas where street sweeping occurred at least once per week, versus those areas that did not receive maintenance. The important element here is the frequency of sweeping. Reducing the frequency of sweeping reduces the contaminant reduction benefits.

Examples of practice remedy could include the following:

- Road and storm drain maintenance practices such as street sweeping (using high efficiency regenerative sweepers) and catch pit cleaning,
- Controls on illegal dumping,

- Landscaping practices that reduce or eliminate the use of fertilisers and pesticides,
- Storage practices for de-icing compounds and grit,
- Fleet vehicle maintenance programmes,
- Covering of contaminant generation areas on industrial sites, and
- Reduce, reuse or recycling programmes.

While avoiding a problem is much easier when consideration is given prior to construction being done, subsequent maintenance by substitution of products or by developing an environmental management plan for maintenance activities also can reduce or eliminate a contaminant problem. Individual actions, when taken in conjunction with other actions, can reduce contaminant loadings over time.

4.1.3 Mitigate

Typical Highway Contaminants

Mitigation has been the historical approach to reducing stormwater contaminants downstream. Mitigation involves the construction of stormwater treatment practices to reduce the quantity of stormwater and the level of contaminants in stormwater runoff.

The purpose of this guideline is to provide design guidance for stormwater management practices and thus it primarily is a mitigation guideline for stormwater effects. Any one practice, on its own, is unlikely to achieve the stormwater management objectives for a given project. For this reason it is necessary to consider the objectives early in the design process when competing demands can be carefully balanced and an integrated solution achieved. The need for, and size of, treatment devices is then minimised, as is their installation and maintenance costs. The combination of a number of different tools or practices to achieve an overall stormwater objective is normally referred to as a “treatment train”.

Stormwater management on new projects will mostly fall in the mitigation category until low impact design principals are promoted and adopted and require stormwater management practices to reduce downstream contaminant levels.

4.2 Stormwater Treatment Processes

Stormwater treatment practices attempt a difficult task; the removal of contaminants entrained in stormwater flows. Significant proportions of contaminants are dissolved in stormwater, and many others are attached to fine particles of silt and clay, which do not easily settle. Processes that reduce contaminant levels include the following:

- Sedimentation
- Aerobic and anaerobic decomposition
- Filtration and adsorption to filter material
- Biological uptake
- Biofiltration
- Flocculation

These processes will be discussed individually in the following subsections.

4.2.1 Sedimentation

Most stormwater management programmes in New Zealand and internationally started initially with an intention to mitigate the effects of excess sedimentation into streams and estuaries. The logic was that capture of sediment, while being beneficial, would also provide capture of other contaminants that are attached to the sediments. The following tables and figure provide discussion of sediment particle size, contaminants associated with various sized particles, fall velocities for various sediment particle sizes and lastly a representation of how particle size determines whether they can be removed by sedimentation.

The first table, Table 4-1 provides a listing of various particle classes and their sizes (Chow, 1964).

Table 4-1 Particle Characteristics

Size		Class
Millimetres	Microns	
64 - 32		Very coarse gravel
32 - 16		Coarse gravel
16 - 8		Medium gravel
8 - 4		Fine gravel
4 - 2		Very fine gravel
2 -1	2,000 - 1000	Very coarse sand
1 - 0.5	1,000 - 500	Coarse sand
0.5 - 0.25	500 - 250	Medium sand
0.25 - 0.125	250 - 125	Fine sand
0.125 - 0.062	125 - 62	Very fine sand
	62 - 31	Coarse silt
	31 - 16	Medium silt
	16 - 8	Fine silt
	8 - 4	Very fine silt
	4 - 2	Coarse clay
	2 - 1	Medium clay
	1 - 0.5	Fine clay
	0.5 - 0.24	Very fine clay

Sediment coarser than medium silt settles rapidly, but much longer settling times are required for finer particles to settle. Particles less than 10 µm tend not to settle discretely according to Stokes Law (1851), but exhibit flocculent settling characteristics. Particle shape, density, water viscosity, electrostatic forces, and flow characteristics affect settling rates.

Stokes Law $V_s = 2/9(r^2g(p_p - p_f)/ή)$

Where: V_s = settling velocity (m/s)
R = particle radius (m)
G = standard gravity (m/s)
p_p = particle density (kg/m^3)

p_f = fluid density (kg/m^3)
ή = fluid viscosity (Pascal-second (pa-s))

Table 4-2 discusses particle size and contaminants associated with them in general stormwater runoff (Ding et al, 1999).

Table 4 - 2 Metals Distribution and Particle Sizes

Particle Size (µm)	Metals Distribution (%)							
	Cd	Co	Cr	Cu	Mn	Ni	Pb	Zn
<10	46	60	71	63	71	63	73	60
10 - 100	36	31	24	30	21	29	23	35
>100	18	9	5	7	8	8	4	5

As can be seen, significant portions of the contaminant loads are attached to finer sediments. It should be noted that there is variation of the above table by various researchers and better information should be obtained before definitive statements are made. The important point is the trend, which indicates that metals tend to be associated with fine sediments.

Table 4-3 shows particle settling velocities based on Auckland data (Semadeni-Davies, 2006) and includes the proportion of particles in each size category.

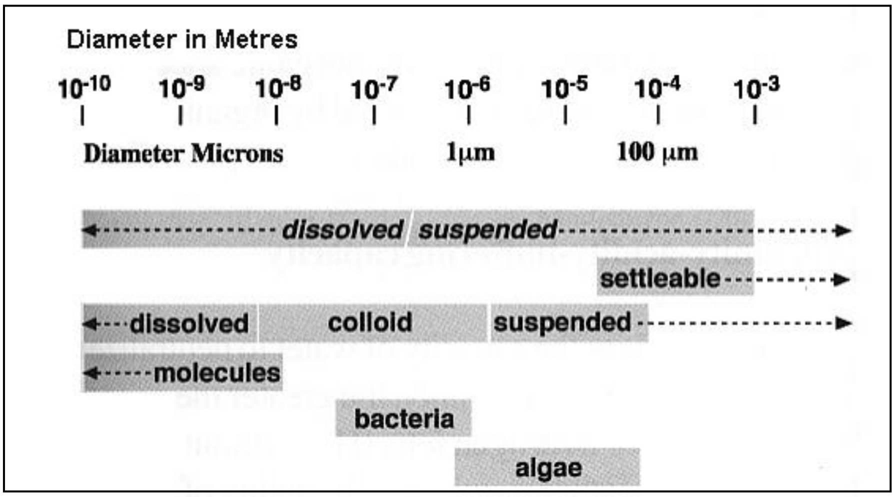
Table 4-3 Particle Size versus Settling Velocity

Particle Diameter (µm)	Proportion of Particles (%)	Cumulative Proportion (%)	Particle Density (kg/m^3)	Settling Velocity (m/h)
3	5	5	1100	0.002
6	8	13	1300	0.021
10	5	18	1600	0.118
15	6	24	1900	0.397
20	5	29	1900	0.706
25	4	33	1900	1.102
30	3	36	2150	2.028
50	12	48	2300	6.366
75	19	67	2500	16.524
100	12	79	2650	32.31
150	15	94	2650	67.732
200	5	99	2650	94.086
300	1	100	2650	149.517

It should be noted that actual settling velocities in the field are often significantly lower than the theoretical values, especially for finer particles. This can be due to turbulence but can also be due to a reduction in settling velocities that occurs the more particles are present. The greater the concentration of suspended sediments, the less the settling velocity can be. Measurements of reductions in settling velocities of 50% and greater have been recorded in high sediment laden water when compared to the same soil particle sizes in clear water. This is not a major factor in permanent stormwater practices but would be a consideration for sediment control ponds.

Figure 4-1 shows sediment particle diameter with the ability to remove various particle sizes with sedimentation (Minton, 2002).

Figure 4-1 Particle Size and General Classification



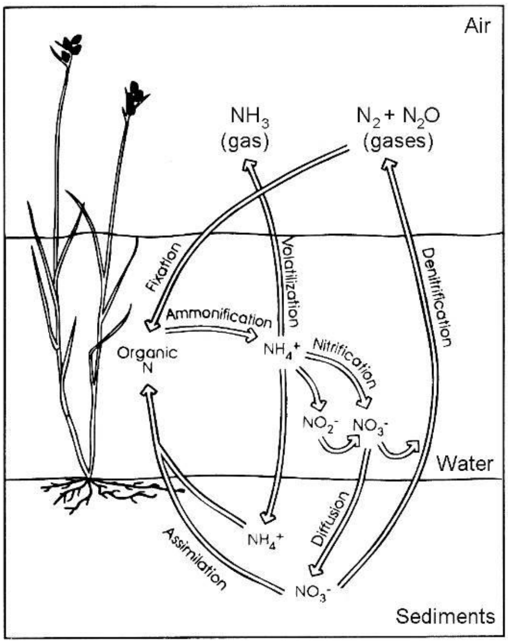
As can be seen from the above tables and figure, the ability to use sedimentation as a means of contaminant reduction is limited to larger particle sizes. Depending on the contaminants of concern, removal of suspended solids by sedimentation alone may not remove the contaminants of greatest concern. It is important to identify the contaminants of greatest concern in order to determine what processes can remove a given contaminant.

4.2.2 Aerobic and Anaerobic Decomposition

Another process by which contaminants are removed is by microorganisms reducing soluble BOD (biological oxygen demand) and breaking down nutrients and organic compounds by aerobic and anaerobic decomposition. The primary practice that uses aerobic and anaerobic decomposition is wetlands.

Once the aerobic microorganisms have taken up contaminants they die and settle to the bottom of ponds where further anaerobic oxidation may take place. In anaerobic conditions, microorganisms can remove nitrogen by de-nitrification. This is an important process in constructed wetland function. Figure 4-2 (Kadlec, Knight, 1996) shows a simplified wetland nitrogen cycle.

Figure 4-2 Wetland Process for Denitrifiation



This process is important when considering road construction or retrofit in areas where nutrient enrichment of receiving systems (primarily lakes) is a problem.

Denitrification is a reduction process where electrons are added to nitrate or nitrite nitrogen, resulting in the production of nitrogen gas, nitrous oxide (N_2O) or nitric oxide (NO). This can only occur when dissolved or free nitrogen is absent. In other words there has to be an anaerobic layer at the bottom of the wetland for denitrification to occur.

Having an anaerobic layer develop in a wetland can have other less desirable effects as water can become acidic and mobilise contaminants already captured. If nutrients are not a concern in a given catchment and wetlands are proposed due to their enhanced ability to capture dissolved metals, it is important to maintain an aerobic environment to prevent remobilisation.

4.2.3 Filtration and Adsorption to Filter Material

As sediment particles pass through a filter bed or through soil, the following filtration processes may remove them:

- Settling into crevices
- Enmeshment (entangling) in interstices
- Impingement onto filter particles followed by sticking onto particles (by electrostatic or other bonding)

Filtration has been used for years in wastewater treatment to remove solids from liquids. In the late 1980's filtration was being applied to stormwater treatment, primarily for sediments and oils and grease removal. It functions by interposing a medium to fluid flow through which the fluid can pass, but the solids in the fluid are retained. Its function is determined by the pore size, the thickness of the medium and the live storage elevation above the medium, which drives the fluid through the medium. The path for the fluid to pass through the medium is tortuous and particles are unable to move through the medium.

Adsorption is the accumulation of dissolved substances on the surface of a media such as plants or filters. Dissolved substances can also be removed by adsorption to filter material and biological uptake by microorganisms living among the filter material.

Adsorption is a process that occurs when a liquid solute accumulates on the surface of a solid or forms a film on the surface. It is different from absorption where the substance diffuses into the solid. Atoms of the clean surface experience a bond deficiency and it is favourable for them to bond with whatever happens to be available. Adsorption is a key removal mechanism for dissolved metal reduction in stormwater runoff.

4.2.4 Biological Uptake

Wetlands and bioretention areas use the interaction of the chemical, physical, and biological processes between soils and water to filter out sediments and constituents from stormwater. They also use interaction of plants to enhance the treatment process. Constituents are first absorbed, filtered and transformed by the soil and then taken up by the plant roots. Table 4-4 provides some discussion of contaminant uptake by vegetation (Kadlac, Knight, 1996).

Table 4-4 Ability of Biota to Uptake Contaminants	
Nitrogen	Nitrogen reduction by plants is extremely complicated and depends on the form of nitrogen, pH, growing season, climate, etc. Most of the information available relates to performance of wetland plants with little information on nitrogen uptake by biofiltration systems. Organic nitrogen compounds are a significant fraction of the dry weight of plants.
Phosphorus	Plants require phosphorus for growth and incorporate it in their tissue. The most rapid uptake is by microbiota (bacteria, fungi, algae, etc.) because they grow and multiply at high rates. Phosphorus is a nutrient and its addition stimulates growth.
Metals	Metals reach plants via their fine root structure, and most are intercepted there. Some small amounts may find their way to stems, leaves and rhizomes. Upon root death, some fraction of the metal content may be permanently buried, but there is no data on metal release during root decomposition.

Plants do take up nutrients or metals from stormwater via absorption processes. However they may also re-release them to the water column when they die and decay. An example of this is a swale that is periodically mowed. Unless the grass cuttings are physically removed from the catchment, they will eventually decompose and the contaminants (primarily nutrients) will again be available for transport downstream.

Biological uptake is a less important process in swales, filter strips and rain gardens than it is in wetlands where, for nutrients, it can be an important process.

4.2.5 Biofiltration

A variation to the filtration mechanism is to use plants as the filter media. Biofiltration is a contaminant control technique using living material to capture and biologically degrade and process contaminants. Contaminants adhere to plant surfaces or are absorbed into vegetation. This mechanism is a combination of filtering, reduced settling time and adhesion.

An example of biofiltration is a swale or rain garden where the combination of soils and vegetation provide natural biofiltration. Rain gardens operate by filtering runoff through a soil media prior to discharge into a drainage system. The major contaminant removal pathways are (Somes, Nicholas and Crosby, Joe, 2007):

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

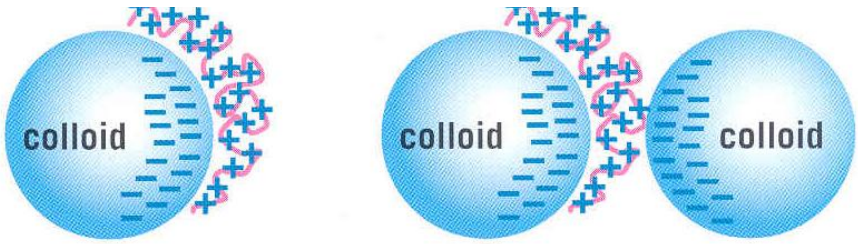
The major issues with performance of biofiltration as a contaminant reduction practice is maintenance of low flow velocities and hydraulic loading during storms too large to permit sedimentation of silts and clays, even with dense vegetation (Mazer, Booth, and Ewing, 2001).

4.2.6 Flocculation

Flocculation is a process of contact and adhesion whereby the particles of a dispersive medium form larger size particles, which then settle to the bottom of a liquid. Clay particles are colloidal particles that have electrostatic surface charges. In general, most colloidal material has a negative charge. Particles with like charges tend to repel each other, preventing the forming of coagulated particles. These characteristics cause the colloidal particles to remain in solution. Destabilising colloidal material to allow coagulation and settlement to occur is achieved by adding reagents that develop positive charges. Positively charged ions in the solution act to destabilise the colloidal matter and allow settlement of coagulated material to occur.

Flocculation occurs after the addition of chemical to destabilise the charges on the colloidal particles in suspension. The particles adhere to each other via the flocculant ions on the surface of the particles. These charged ions provide an opportunity for charged particles in a system to adhere to them, thereby merging individual particles (Figure 4-3). This results in larger, denser flocs that settle more rapidly (ARC, 2003).

Figure 4-3 Process of coagulation of Colloids due to Ion Exchange



The Auckland Regional Council and the New Zealand Transport Agency have been using flocculation for approximately 7 years on sediment control ponds to improve performance at removal of clay particles. Monitoring to date has indicated enhanced sediment removal performance using polyacrylamide, alum and polyaluminium chloride (PAC). For a number of reasons use has gravitated to PAC as the flocculant of choice.

There has been some experimentation with flocculation in New Zealand for lake phosphorus control but that has been done on a trial basis and not implemented long term (Environment BOP, 2004). It has been used overseas, primarily using alum in lakes for nutrient and macrophyte control. Results have been generally positive but there have been indications of potentially toxic concentrations of aluminium with dosing (Carr, 1999). In addition, the issues of collection and disposal of flocculated sediments is an issue that needs better direction and understanding (Harper, Harvey, H., undated).

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5 Choosing a Stormwater Management Device

5.1 Introduction

Stormwater management practices are asked to provide water quantity control, water quality control or sometimes both. It is important to recognise that stormwater management practices do not perform equally and in all situations. A practice using infiltration of runoff as the method of choice is not going to function in soils that do not allow passage of water through it due to limited permeability rates. In the same regard, a practice such as a stormwater management pond may be good at removal of suspended solids but provide little benefit for dissolved metals reduction.

It is important to recognise the potential effectiveness of different stormwater practices on the contaminants generated at a specific site and for a given receiving environment. Consideration should be given to contaminants of concern and stormwater management practices appropriate to remove those contaminants.

5.2 Site Considerations

The success of any management practice depends on selecting the appropriate options for the sites control objectives and conditions at an early stage. The objectives must be clearly defined at the outset and site conditions investigated in enough detail to match the practice to the site so as to meet the objectives. Decisions need to be made whether quantity control, quality control or ecosystem protection or enhancement are required and which contaminants need to be treated and how.

Deciding whether a practice is relevant means looking at the following issues.

- Soils in the location of the intended stormwater management practice,
- Slopes,
- Catchment area draining to individual practices, and
- General constraints

The following sections discuss each of these items in more detail.

5.2.1 **Soils**

Underlying soils are very important to determine whether a given stormwater management practice will function as intended. More permeable soils can enhance the operation of some practices, but adversely affect the performance of others. As an example, a constructed wetland may not retain water if the underlying soils are sandy.

For a number of practices, having soil of a given permeability may not present fatal problems. If a constructed wetland were intended for a given site that has sandy soils, a clay or geotextile liner would prevent infiltration of water and maintain a normal pool level.

On the other hand infiltration practices rely on passage of water through the soil profile, and more permeable soils transmit greater volumes of water. Having poor permeability in subsoils would preclude the use of infiltration practices for a given area. From a general context, the following Table 5-1 provides a discussion of various soils and their approximate infiltration rate.

Table 5-1 Infiltration Rate for Various Soil Textural Classes

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

The location of the red line in the Table indicates a normal minimum permeability limit for when infiltration practices are suitable for a given site. If the infiltration tests indicate an infiltration rate of less than 7 mm/hour then infiltration is not normally considered as an appropriate practice.

The following Table 5-2 provides a view of practices and their suitability for various soil textures.

Table 5-2 Soil and Suitability of Various Stormwater Management Practices

Ponds/ Wetlands				
Sand Filters				
Rain Gardens				
Infiltration				
Swales/Filter strips				
	Sand	Loam	Silty Clay	Clay

Blue colour denotes acceptable practice range related to soil types

To some people there is confusion over what a loam soil is. Loam is soil that is composed of sand, silt and clay in relatively even concentration (approximately 40-40-20% respectively). Loam soil contains the right amount of sand, silt, clay and organic material. It is known as a "garden soil" that is good for plants. They generally contain more nutrients than does sandy soils. Silty loam is generally considered as the soil having the minimum permeability rates for use of infiltration practices. Loamy soil is also commonly recommended for use in rain gardens.

5.2.2 Slopes

Slope is important when selecting a stormwater management practice. Steeper slopes may:

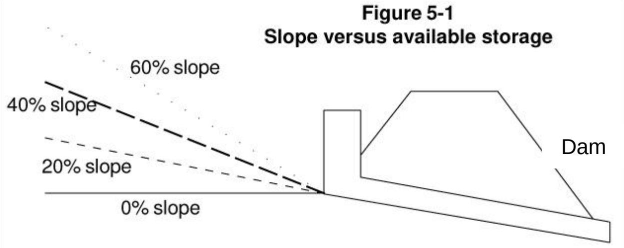
- Eliminate some practices from consideration,
- Require practices to be modified from a more desired approach, or

- Have little impact on the use of others.

Stormwater management practices that rely on storage of water have slope limitations as adequate storage may necessitate significant cuts and fills to meet storage requirements. The following simplified Figure 5-1 (ARC, 2003) shows how slope steepness can impact on storage ability of a pond. The same analogy applies to filter systems that have a live storage requirement.

Other practices, such as vegetated swales may be adapted for steeper slopes if the swales are placed along the contours, rather than up or down slopes. The ability to manipulate direction of swales is limited and slope may well determine whether swales or filter strips can be used on a given project. Swales and filter strips are normally limited to approximately a 5% slope to ensure that adequate residence time will be provided for significant contaminant reduction and to ensure that flow velocities don't cause erosion.

Figure 5-1
Slope versus available storage



Slope (%)	Area of storage available (bottom to crest) in m²
0	7.5
20	5.0
40	2.8
60	1.9

The following Table 5-3 provides some discussion of stormwater management practices and their limitations related to slope.

Table 5-3 Slope Limitations of Various Stormwater Management Practices

Practice	Slope Limitation
Ponds/ Wetlands	As the slope increases the amount of cuts and/or fills increases. Ponds generally are not suitable on slopes > 10%
Sand Filters	Sand filters can either be pre-fabricated units or constructed in place. For prefabricated units, generally live storage can be provided within the unit so slope is not a critical issue. For open systems, the slope problems are similar to ponds or rain gardens
Rain Gardens	Similarly to ponds, live storage is a problem on steeper slopes. The surface of the rain garden has to be level to ensure an even flow through the media
Infiltration	Infiltration practices are not recommended on steeper slopes or on fill slopes. There is a potential for slope instability with seepage coming out on the slope below the practice or for lateral flow to occur at the natural ground/fill interface. Infiltration should only be used when a geotechnical engineer certifies it as an appropriate use.
Swales/Filter strips	Not suitable for slopes > 5% unless check dams are used to flatten overall slope

5.2.3 Catchment area

Catchment area is another key element that determines the suitability of a stormwater management practice at a specific site. Some practices, due to treatment or hydrological factors are more appropriate to smaller or larger catchment areas. Practices that rely on vegetative or filter media filtering of runoff are more appropriate for smaller catchment areas, as large flows may overwhelm their ability to filter the runoff. Ponds, on the other hand, are more appropriate for larger catchment areas.

(ARC, 2003). The following Table 5-4 provides guidance for stormwater management practices and catchment areas that they are suitable for.

Table 5-4 Stormwater Management Practices Related to Appropriate Catchment Areas

Stormwater Management Practice										Controlling factor for use
Ponds					Suitable for use	Suitable for use	Suitable for use	Suitable for use	Suitable for use	Catchment area to maintain normal pool of water
Wetlands			Marginal for use	Suitable for use	Suitable for use	Suitable for use	Suitable for use	Suitable for use	Suitable for use	Catchment area to maintain hydric soils
Sand filters	Suitable for use	Suitable for use	Marginal for use							Volume of runoff
Rain gardens	Suitable for use	Marginal for use								Volume of runoff
Infiltration	Suitable for use									Soils, slope, stability, etc.
Swales and filter strips	Suitable for use	Suitable for use								Rate of runoff and slope
	0	2	4	6	8	10	12	14	20	40 (in hectares)
Suitable for use		Marginal for use								

5.2.4 General constraints

There are a number of other constraints that may limit a given practice from being used on a specific site. Those items can include, but not be limited to, the following issues:

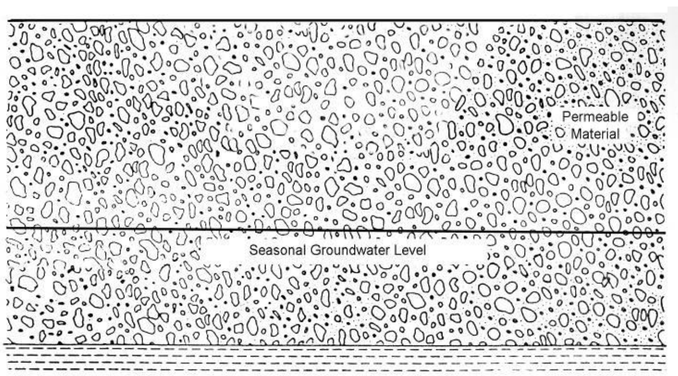
- High groundwater table and potential mounding
- Proximity to bedrock
- Slope stability
- Space availability
- Maximum depth limits
- High sediment input
- Thermal effects
- Cost

Each of these issues is discussed in the following subsections.

5.2.4.1 High groundwater table and potential mounding

Having a high groundwater table can preclude the use of a number of practices. Figure 5-2 (Department of Natural Resources, 1984) shows a typical schematic of ground surface and groundwater level. Seasonally there can be a wide variation in groundwater levels and that difference can be in excess of a metre depending on the time of year.

Figure 5-2 Groundwater Schematic



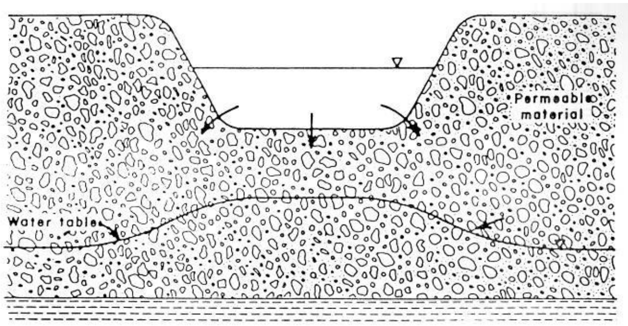
Practices that need to be cognisant of groundwater levels in terms of their location and applicability include:

- Ponds, both deeper and wetlands
- Infiltration practices
- Swales

Filter systems can generally be designed around site conditions as long as there is a positive outfall.

Groundwater mounding can also be a concern. This is particularly true for infiltration practices, where significant surface runoff is concentrated in one area, soaks into the ground and then elevates local groundwater levels as shown in Figure 5-3. Even though predevelopment groundwater levels may be low enough that problems shouldn't result, the artificial raising of local groundwater levels could cause performance problems.

Figure 5-3 Groundwater mounding under an infiltration practice



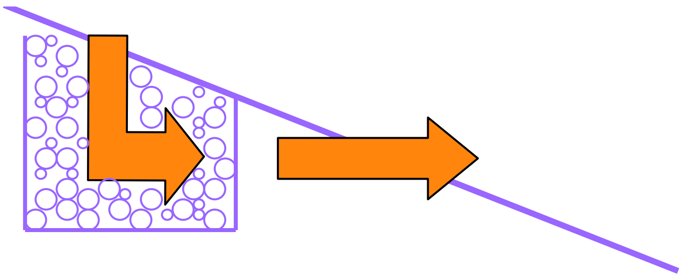
5.2.4.2 Proximity to Bedrock

Proximity to bedrock has two major issues: drainage in a similar fashion that infiltration practices have with groundwater levels, and cost to construct a practice whose invert requires excavation in bedrock. Either of these two issues could fatally impact on use of a given practice and it could apply to any practice depending on the depth to bedrock.

5.2.4.3 Slope Stability

Having a practice on a slope could increase instability issues related to the slope. Practices that discharge to ground on a slope could have that discharge exist the slope above the toe of the slope, as shown in Figure 5-4 (figure by author), and increase saturation of the slope or have overland flow across the slope where it did not exist prior to development. This could apply to infiltration practices primarily but could also apply to rain gardens and swales if they discharge to the top of a steep slope.

Figure 5-4 Slope and Seepage from an Infiltration Trench



If local stability codes are considered from around the country, a uniform requirement relates to consideration of springs and groundwater conditions. The Greater Wellington Regional Council has a Landslide Hazard Fact Sheet (undated) with one heading being “Too Much Water”. The text states, “A small amount of rain dampens the soil and helps particles stick to each other. Too much rain can cause the soil particles to lose contact with each other. Then the heavy waterlogged soil starts to move. Small surface slips after rainstorms are common in the Wellington Region”. Using a practice on a slope that may not have existing stability issues and then artificially putting water in the soil could cause stability concerns. Geotechnical reports should be done if there is an intention to place stormwater flows in the ground on slopes.

5.2.4.4 Space Availability

In general, space allocated for stormwater management is always going to be limited. There may be situations where regulatory requirements and downstream impacts may necessitate acquiring additional land but, for the most part, practices will have to fit within a given limited site area that is available for stormwater management practices.

From a water quantity perspective there may be opportunities to be creative, such as under sizing pipes conveying catchment drainage and using road embankments to control water quantity discharges downstream. Water quality will still need to be provided for the highway itself, as catchment flows would necessitate large treatment practices.

Those practices having the greatest area requirements are ponds and wetlands. These practices are generally more appropriate for larger catchments (as detailed in the catchment discussion in Section 5.2.3) and larger footprints.

5.2.4.5 Maximum Depth Limits

There will be situations, especially where there are reticulation systems, when the invert of the receiving system pipe will determine the invert of the stormwater management practice. If the invert of the receiving pipe is above the invert of the stormwater management practice then the practice won’t drain and could cause localised flooding or system bypass.

There has to be a positive outfall from stormwater management practices if they are to function for peak control or water quality treatment.

Prematurely Clogged Rain Garden from Unstabilised Adjacent Areas

5.2.4.6 High Sediment Inputs

A number of stormwater management practices are sensitive to excess sediment loadings and will incur maintenance problems if catchment sediment loads draining to the individual practice are high. Examples of this situation would be areas adjacent to a treatment practice undergoing earthworks and having high sediment loads entering the treatment system. Another situation could involve horticultural activities where seasonal land clearing and planting could increase sediment runoff to treatment practices.

Practices that are sensitive to high sediment loadings that will have a fairly rapid decline in water quality treatment performance include the following:

- Infiltration practices
- Sand filters
- Rain gardens
- Swales
- Filter strips

Ponds and wetlands, having sediment forebays, have the ability to store larger sediment loads than the other practices, although they still would require more frequent maintenance to maintain performance.

5.2.4.7 Thermal Effects

Water temperature affects water chemistry and quality, and has a pervasive, over-riding influence on the biota through its control of enzyme systems and the physiology of cold-blooded animals. Water temperature is therefore a key factor influencing the ecological performance of streams. Summer is the main problem time for stream temperatures.

Pavement modifies stormwater temperatures, raising it during the summer, but cooling it in cold winter months. A study done in the U.S. (Black, 1980) observed during one summer storm that the temperature of the stormwater from a parking lot was 5^{o}C higher than the rainwater.

From a New Zealand context, acute mortality for most native NZ fauna tested to date occurred above 25^{o}C. LT_{50} values (lethal temperatures that killed 50% of the test organisms over a 10 minute duration) for nine species of native fish ranged from 27.0-31.9^{o}C (Richardson et al, 1994). Juvenile and adult eels were considerably more tolerant than other fish species (LT_{50} ranges from 34.8-39.7^{o}C), and thus were not used in setting assessment criteria. Native invertebrate species were more sensitive than fish, where LT_{50} values (24-hour exposure) for 12 species ranged from 25.9-32.4^{o}C (Quinn et al., 1994). Simons (1986) recommended that a maximum value of 3^{o}C below the lowest LT_{50} would allow for a margin of safety. Based upon the test data and interpretations, slight, moderate and severe adverse effects were estimated to occur above 22^{o}C, 24^{o}C and 26^{o}C, respectively.

There are two possible sources of temperature increase from urban land uses: impervious surfaces and stormwater management ponds. Temperature increases from pavements were mentioned above but stormwater ponds could increase thermal loadings to receiving systems. Ponds may have degraded water quality due to temperature increases, as their surface area tends to be exposed to direct sunlight and heat up. They could cause significant adverse effects on downstream macroinvertebrate communities (Maxted et al, 2005). There are ways to reduce those impacts including if the ponds:

- Are not located in stream channels,
- Have below surface outfalls (temperatures are greatest at the surface), and

- Are small enough that riparian vegetation could provide shading of the pond surface.

Thermal impacts from wetlands are reduced from those caused by ponds due to increased surface area coverage by wetlands vegetation.

Other stormwater management practices may mitigate the effects of impervious surface temperature increases by moderating temperature as the water passes through the practice.

5.2.4.8 Cost

Stormwater management costs can relate to several factors including:

- Property acquisition
- Practice construction
- Whole of life costs relating to subsequent operational expenses

All of these factors can, and should, enter into decisions regarding practice selection and implementation. Costs may be difficult to predict on a nationwide basis depending on regulatory requirements from various consenting authorities, and site acquisition costs will be highly variable.

Landcare Research is developing a whole-of-life costing model for a number of stormwater management practices. It is very preliminary at this time but it should provide good value on stormwater cost considerations when it is completed.

An example of highway expected costs comes from the State of Washington (Hoey and Girts, 2000) where the Washington State Department of Transportation estimates $7 million/year for maintenance of stormwater practices and capital costs raging from 8 - 20 percent of total project costs depending on project type and location.

Experience by this author of stormwater management practices over the years has indicated operational costs would approximate 5% of construction costs on an annual basis to ensure adequate funding for maintenance activities. There will be years where that funding is not completely used but there will be other years where significant maintenance is required, which averages the long-term costs out.

One element of costs that may not normally be considered is the benefits of stormwater management relating to the following:

- Flooding and property damage
- Degradation of water quality
- Loss of fish and wildlife habitat
- Loss of marine habitat

Ward and Scrimgeour (1991) utilised a non-market valuation technique to quantify some of the valued aspects of Auckland's marine environment. They considered that the total benefits derived, based on the level of water quality at that time, were estimated to be $442 million (in 1991 dollars) per annum. In addition, scenarios were considered to calculate future benefits and losses as a result of deterioration in water quality. This work was only an estimation of values associated with the marine environment and excluded freshwater environmental values and the avoided property and safety implications of flooding events in developed areas. While this was only one study it is indicative that there are financial benefits to implementation of stormwater management on highways.

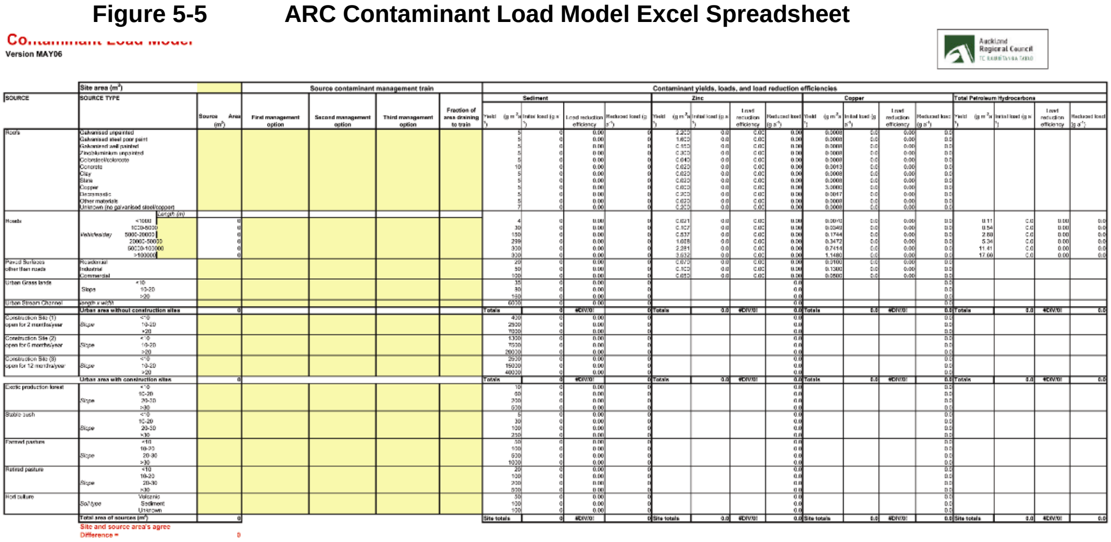
5.3 Contaminant Generation

Addressing contaminants should be done on the basis of the receiving system and the potential contaminants generated by the activity. For years, most stormwater management programmes were focused towards removal of suspended solids, but that may not be appropriate for activity-derived contaminants or for various receiving systems. When looking at contaminant generation potential, New Zealand data is similar to water quality data collected overseas.

In terms of practice selection, New Zealand data indicates that as with overseas studies, lead is the least soluble of the key elements in stormwater (<10%) with zinc being the most soluble (about 40%). Cadmium and copper appear to be moderately soluble with about 30% in the soluble phase. If zinc is a concern on a given project, practices that rely on sedimentation will not be effective at total zinc removal. If lead were a specific concern, sedimentation would be an effective approach.

The Auckland Regional Council has developed a contaminant load model (ARC Version May 2006) that inputs the land use (source) that is generating the contaminant and then allows various stormwater management practices to be applied to determine contaminant discharge from a given area. The following figure shows the contaminant spreadsheet, which can be downloaded from the ARC

Figure 5-5 ARC Contaminant Load Model Excel Spreadsheet



website at *www.arc.govt.nz/fms/stormwater/contaminantLoadModelMAY06.xls*The above figure is not provided to view various inputs but rather to show the general appearance of the spreadsheet. For roads, the contaminant model considers various vehicles/day and applies contaminant loads for that situation as shown in the following table 5-5.

Table 5-5 Contaminant Loads for Various Daily Traffic Counts

Vehicles/day	Contaminant Unit Loadings for Various Contaminants			
	Sediment ($g/m^2/yr.$)	Zinc ($g/m^2/yr.$)	Copper ($g/m^2/yr.$)	Total Petroleum Hydrocarbons ($g/m^2/yr.$)
<1,000	4	0.021	0.0070	0.11
1,000-5,000	30	0.107	0.0349	0.54
5,000-20,000	150	0.537	0.1744	2.68
20,000-50,000	299	1.068	0.3472	5.34
50,000-100,000	300	2.281	0.7414	11.41
>100,000	300	3.532	1.1480	17.66

As can be seen, the contaminant loads increase geometrically rather than linearly. Very high traffic roadways can have a significant impact on contaminant delivery to a receiving system. The daily traffic count could very well determine whether stormwater management needs to be provided for a given highway. This information would also help to prioritise where stormwater treatment needs to be provided.

5.4 Contaminant Removal Processes

Once the contaminants of greatest concern are identified, it is important to understand the processes that may be used to reduce contaminant discharge downstream. The following Table 5-6 lists all of the principal mechanisms that can capture, hold and transform various classes of contaminants in stormwater runoff and the factors that promote the operation of each mechanism to improve water quality.

Table 5-6 Summary of Contaminant Removal Mechanisms

Mechanism	Contaminants Affected	Removal Promoted By
Physical sedimentation	Solids, BOD, pathogens, particulate COD, P, N, metals, synthetic organics	Low turbulence
Filtration	Same as sedimentation	Fine, dense herbaceous plants, constructed filters
Soil incorporation	All	Medium-fine texture
Chemical precipitation	Dissolved P, metals	High alkalinity
Adsorption	Dissolved P, metals, synthetic organics	High soil Al, Fe, high soil organics, neutral pH
Ion exchange	Dissolved metals	High soil cation exchange capacity
Oxidation	COD, petroleum hydrocarbons, synthetic organics	Aerobic conditions
Photolysis	Same as oxidation	High light
Volatilisation	Volatile petroleum hydrocarbons and synthetic organics	High temperature and air movement
Biological microbial decomposition	BOD, COD, petroleum hydrocarbons, synthetic organics	High plant surface area and soil organics
Plant uptake and metabolism	P, N, metals	High plant activity and surface area
Natural die-off	Pathogens	Plant excretions
Nitrification	NH_3-N	Dissolved oxygen>2mg/l, low toxicants, temperature>5-7^oC, neutral pH
Denitrification	NO_3+NO_2-N	Anaerobic, low toxicants, temperature>15^oC

A key factor to consider in the functioning of all mechanisms is time. The effectiveness of settling a solid particle is directly related to the time provided to complete sedimentation at the particle's characteristic settling velocity (shown in Table 4-3). Time is also a crucial variable to determine the degree that chemical and biological mechanisms operate. Characteristic rates of chemical reactions and biologically mediated processes must be recognised to obtain treatment benefits. For all of these reasons, water residence time is the most basic variable to apply as an effective treatment practice technology.

The information in Table 5-6 can also be arranged by features that promote specific contaminant control objectives. The following features provide for the most common objectives.

- Features that assist in achieving any objective
  - Increasing hydraulic residence time
  - Low turbulence
  - Fine, dense herbaceous plants
  - Medium-fine textured soil
- Features that assist in achieving specific objectives
  - Phosphorus control
    - High soil exchangeable aluminium and/or iron content
    - Addition of precipitating agents

- Nitrogen control
  - Alternating aerobic and anaerobic conditions
  - Low toxicants
  - Neutral pH
- Metals control
  - High soil organic content
  - High soil cation exchange capacity
  - Neutral pH
- Organic control
  - Aerobic conditions
  - High light
  - High soil organic content
  - Low toxicants
  - Neutral pH

5.5 Device Selection

Section 6 is going to provide detailed discussion on choosing stormwater management practices. This section is providing a more generic discussion of practices and their ability to remove various contaminants and function for water quantity control. In a number of situations, stormwater management practices can provide both water quantity and water quality control for a given site. In other situations, this may not be possible and multiple practices may have to be used to achieve desired outcomes. The following Table 5-7 provides some discussion of various practices and their ability to address water quantity and water quality for various contaminants.

Table 5-7	**Stormwater Management Practices and Water Quantity/Quality Control**				
Practice	Water quantity Peak control Capability	Water Quality Capability			
		Sediment	Metals	TPH	Nutrients
Extended detention dry pond	High	Moderate	Pb - Moderate Cu - Low Zn - Low	Low	P - Low N - Low
Extended detention wet pond	High	High	Pb - High Cu - Moderate Zn - Low	Low	P - Moderate N - Low
Wet pond	High	High	Pb - High Cu - Moderate Zn - Low	Low	P - Moderate N - Low
Wetland	High	High	Pb - High Cu - High Zn - High	High	P - High N - High
Filter systems	Low	High	Pb - High Cu - Moderate Zn - Low	High	P - Moderate N - Low
Rain gardens	Low	High	Pb - High Cu - High Zn - High	High	P - High N - Moderate
Infiltration	Moderate	High	Pb - High Cu - High Zn - High	High	P - High N - Moderate
Swales and filter strips	Low	High	Pb - High Cu - Moderate Zn - Moderate	Moderate	P - Moderate N - Low

As can be seen from Table 5-7, selection of a stormwater management practice or practices will depend on the contaminants of concern and whether peak discharge control is a requirement. Other than wetlands, water quality practices have limited peak flow control capability and must be used in conjunction with another practice if overall project control is to be achieved.

5.6 Treatment Train Approach

As mentioned briefly in the previous paragraph, water quality treatment practices have limited peak flow control capability and must be used in conjunction with a water quantity control practice if both issues (water quantity/water quality) are to be addressed. It may be difficult for one practice to provide for multiple benefits and increasingly, on an international basis, more emphasis is being placed on a stormwater “treatment train” approach to stormwater management where several different types of stormwater practices are used together and integrated into a comprehensive stormwater management system.

A treatment train approach ideally considers both source control and treatment as part of the overall approach. Cleaning catchpits, street vacuum sweeping, substitution of various less contaminating building materials would be the first car in the treatment train. Source control can have value and should be considered.

Once source control has been implemented to the degree that it can, contaminant removal and peak flow control would then be pursued. A word of caution should be mentioned though with respect to using practices that complement each other and don't serve the same function. A basic question has to also be asked, "Is the incremental improvement of using two or more practices worth the additional expenditure of funds. It may be that two practices do overlap in function but the first practice is easier and less expensive to maintain than the second one and the cost savings would offset the additional construction cost.

Minton (2006) provides a number of recommendations for a treatment train approach that have been adapted in the following Table 5-8 to discuss how various practices may work in conjunction with one another.

Table 5-8 Various Desired Functions with Examples of Various Stormwater Management Practices

Function	Examples
Removal of coarse solids to reduce maintenance costs	Forebay in a wet pond or extended detention dry pond followed by a sand filter
Removal of fine sediments to meet a treatment performance goal	Sand filter followed by a wet pond or wetland
Removal of dissolved contaminants	Sorptive media filter followed by wet pond, wetland or rain garden
Reduction of petroleum hydrocarbons to prevent clogging of a second treatment practice	API unit followed by a sand filter or rain garden
Removal of litter to prevent clogging or fouling a second treatment practice	Continuous deflection separation followed by a wetland
Infiltration	Swale followed by an Infiltration practice
Aesthetics	Rain garden followed by a wetland
Wildlife habitat	Rain gardens followed by a wetland
Reliability of long term performance	Wet pond followed by a wetland

Recommendations also adapted from his overall list include the following:

- Follow the golden rule: Don't place in a treatment train two practices that have the same function.
- Conversely, follow the second golden rule, which is to have a different function for each element of the treatment train.
- When considering a specific system component, the specific contaminant to be removed should be identified, rather than thinking in terms of a general removal of multiple contaminants.
- Any two elements of the system should be considered separately.
- Recognise that including a second element may provide minor benefit.
- The additional expected benefit of an additional element should be compared to the incremental cost of the added element operation.
- Care should be taken when calculating efficiency of the overall treatment train.

An example of a treatment train approach could be the use of swales adjacent to a roadway. The swales would then discharge into a wet pond or a wetland. The combination of practices would provide water quantity control and water quality control for sediments and dissolved metals. Depending on the outlet design of the wetland, hydrocarbons would be volatilised and evaporate. The combination of practices would provide excellent water quality control.

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6 Hydrology and Water Quality

When considering hydrologic design criteria recommendations, the recommendations have to be considered in light of the issues discussed in Chapter 3 regarding receiving systems. These issues include:

- Water quantity design criteria,
- Stream channel erosion mitigation criteria, and
- Water quality design criteria.

The following sections discuss the three issues.

6.1 Water Quantity Design

There are two purposes for implementation of water quantity control:

- Preventing existing flooding problems from getting worse, and
- Controlling intermediate storms to minimise potential increases in out-of-bank flows downstream

Both of these situations may be encountered on a specific project on a case-by-case basis. It is important to define the source of flooding problems and situations where flooding issues need to be considered.

The situation considered in this Guideline is flooding in the context of being caused or exacerbated by impervious surfaces. These surfaces increase stormwater runoff from a pre-development condition that may have been pasture or bush. It is not the intent of this Guideline to consider flooding from a tidal surge context. Thus, flooding issues are considered on streams or reticulation systems located within catchments that drain rainfall-generated runoff and are not tidally induced flooding.

6.1.1 **Preventing existing flooding problems from getting worse**

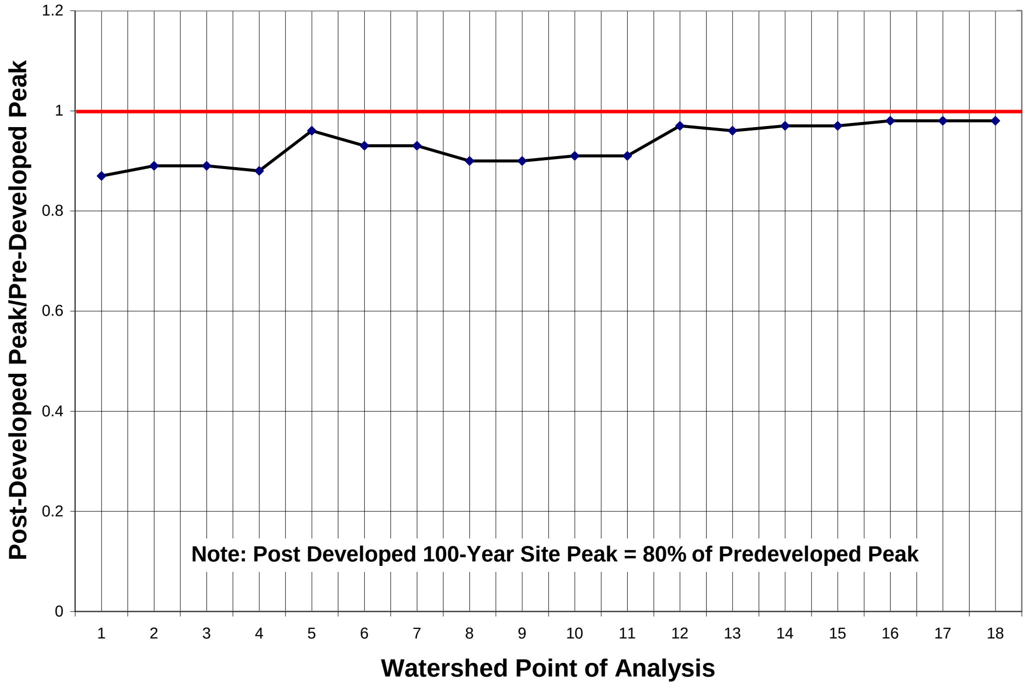
It is imperative that projects not increase the risk of downstream flooding where there is flooding potential for existing structures. Structures, in this context could be habitable buildings or highways.

Where there are downstream flooding problems, peak discharges for the post-development 100-year (1% AEP) storm may need to be managed to ensure that downstream flood levels are not increased. Depending on the catchment, the number of tributaries and the location of the project in a catchment, timing of stormwater discharges may be an issue.

Two bodies of work have been done related to preventing increases in downstream flood potential when hydrologic analyses have not been done on a catchment-wide basis. In a study on the Flat Bush catchment in Manukau City (MCC, 2004) MCC limits post-development peak flow at 80% of the pre-development flow rate. The 80% flow rate was based on a catchment hydrological model. This is to compensate for the increased volume of runoff as a result of development in the catchment. Normal attenuation of this runoff in ponds considerably extends the duration of sub-catchment peak flows, resulting in a greater coincidence of peaks and therefore a greater combined downstream discharge than occurs in the pre-development situation. The indicative target of 80% is necessary to avoid any cumulative hydrological effects that could increase the peak flow downstream.

In another study, catchment studies in New Jersey, U.S. (Shaver et.al, 2007) The following Figure 6-1 details identical criteria to the Manukau City 80% figure for catchments there.

Figure 6-1 Comparison of Pre- and Post- 100-year Peak Rates for Middle Brook Catchment



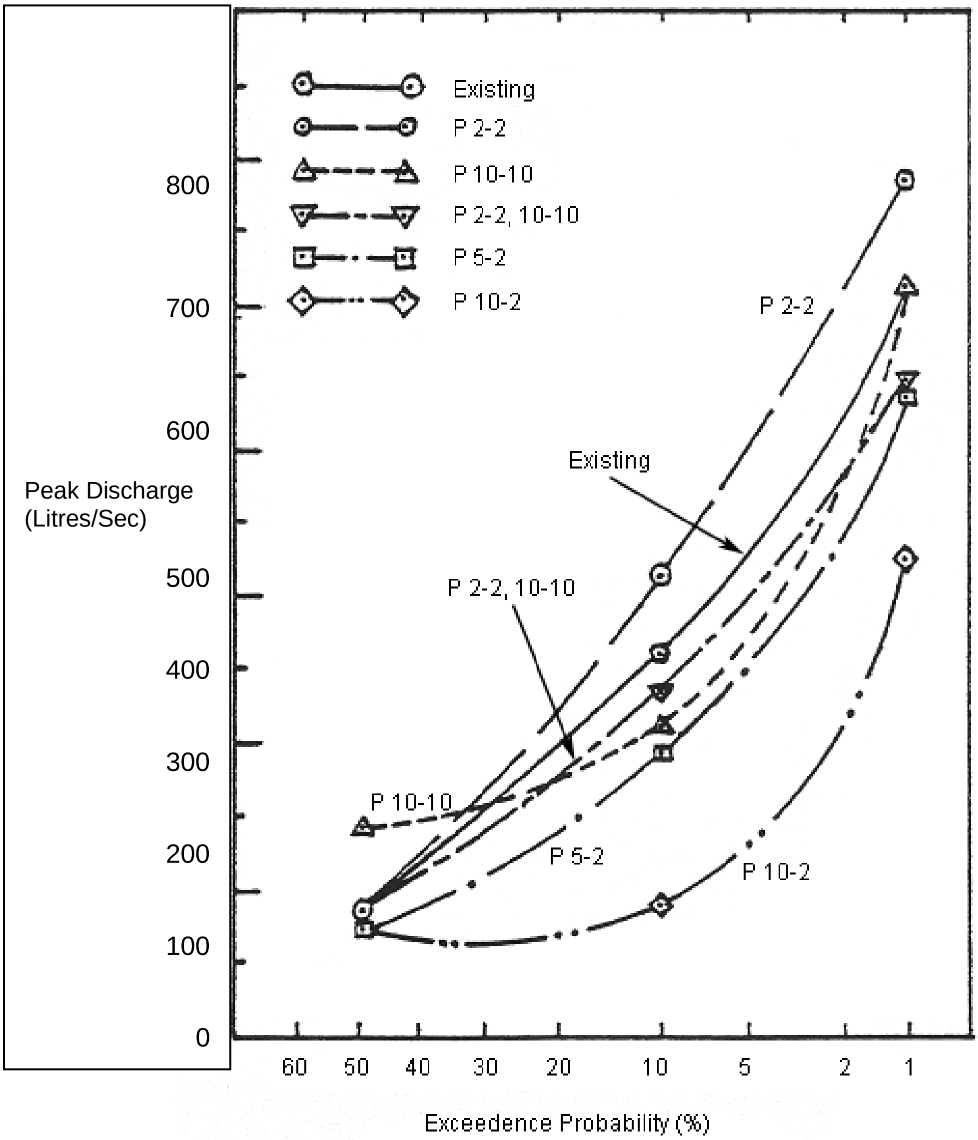
An examination of this comparison shows that, under this level of peak rate control, post-developed runoff rates are less than pre-developed for the entire storm. This increased time period offers greater opportunity for this and other post-developed site hydrographs with similar levels of control to combine downstream in such a way as to produce a total downstream peak that is no greater than the pre-developed peak at that location.

In the absence of a catchment study that evaluates a potential project in a given location, it is important to err on the side of conservatism, especially where human safety or structure damage is concerned. As such, in catchments where flooding problems do exist, it is recommended that the post-development peak discharge for the 100-year storm for a new development be limited to 80% of the pre-development peak discharge.

6.1.2 Controlling intermediate storms

The intent of peak discharge control of storms is to limit downstream increases in larger storm frequencies from the 2-year storm and larger. The issue of which storms to control has been considered (Department of Natural Resources, 1982) through an analysis of a number of different policies for peak flow control. By considering a wide range of policies in conjunction with their peak flows, volumes and timing the effects of the various policies can be visually represented through flow duration curves and hydrographs. Figure 6-2 shows a comparison of flood frequency curves for various stormwater management policies.

Figure 6-2 Comparison of Flood Frequency Curves for Various Stormwater Policies

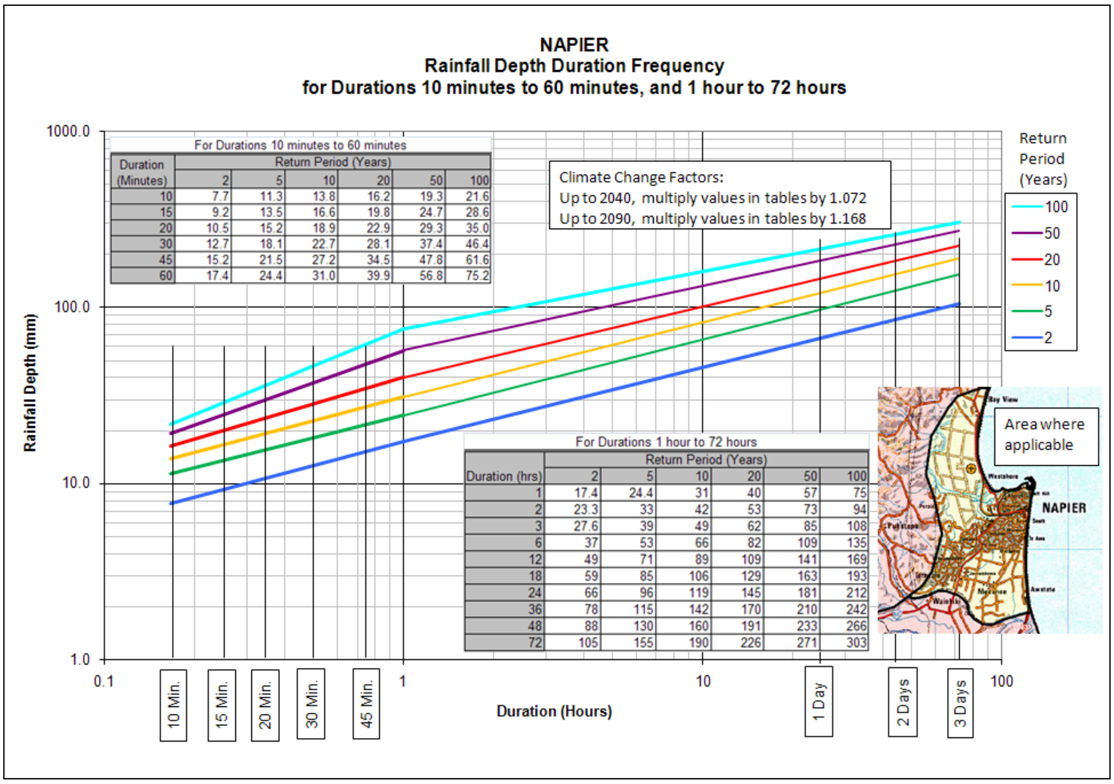


In the above figure, P stands for policy while the first number after the P stands for the post-development storm frequency and the second number stands for the pre-development storm frequency. A P 2-2 reflects a policy where the post-development peak discharge for the 2-year storm cannot exceed the pre-development peak discharge for the 2-year storm. A P 5-2 policy means that the post-development 5-year peak discharge cannot exceed the 2-year pre-development peak discharge.

What can be seen from the figure is that the 2-2, 10-10 (post-development 2 year storm cannot exceed the pre-development 2-year storm and the 10-year post-development storm cannot exceed the pre-development 10-year storm) comes closest to matching the existing frequency curve. By providing multiple storm control the post-development frequency curve comes closest to the pre-development frequency curve. Matching the 2- and 10-year post-development storms to their pre-development level is a common way of minimising downstream intermediate storm peak discharge increases.

6.1.3 Design rainfall

Detailed design rainfall calculations involve the collection of many years of rainfall over large enough area. The HBRC have completed rainfall frequency analyses for several urban areas (Napier, Hastings, Wairoa, Waipukurau/Waipawa). The results from these studies are presented below. The values may be used in the design process.

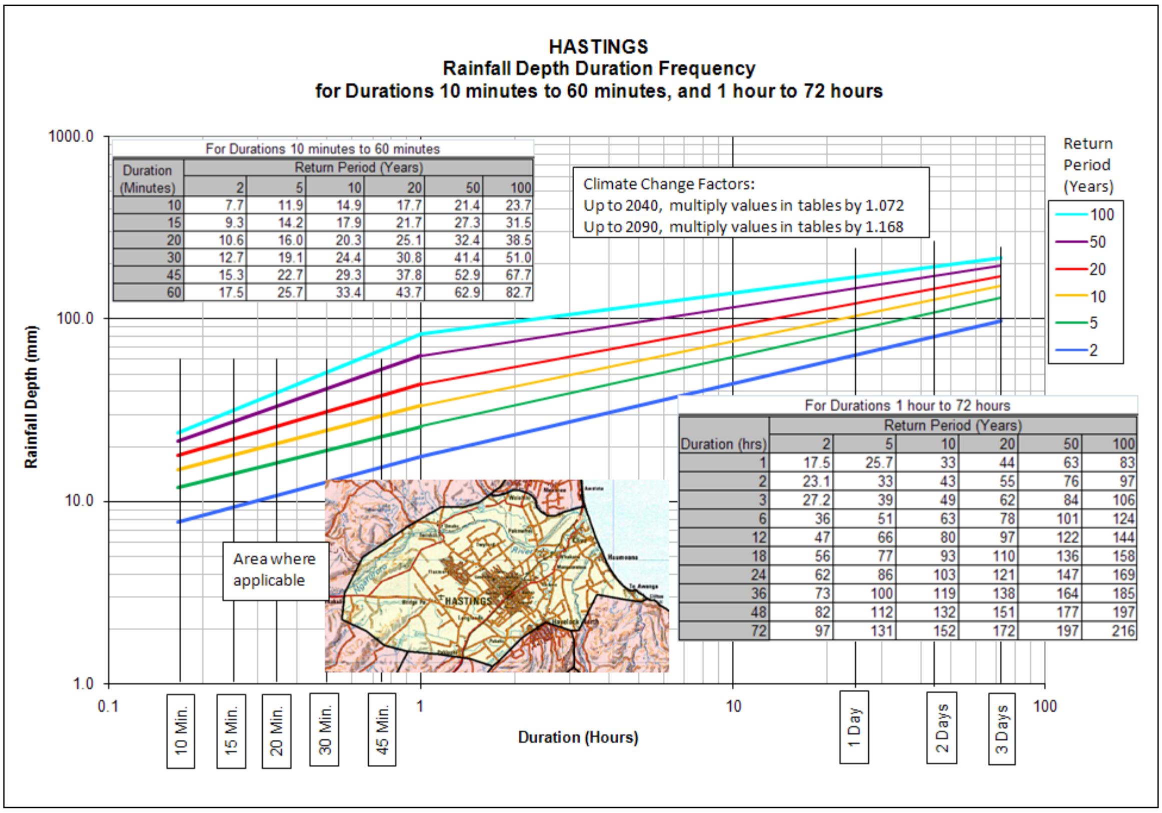


For Durations 10 minutes to 60 minutes

Duration (Minutes)	Return Period (Years) 2	5	10	20	50	100
10	7.7	11.3	13.8	16.2	19.3	21.6
15	9.2	13.5	16.6	19.8	24.7	28.6
20	10.5	15.2	18.9	22.9	29.3	35.0
30	12.7	18.1	22.7	28.1	37.4	46.4
45	15.2	21.5	27.2	34.5	47.8	61.6
60	17.4	24.4	31.0	39.9	56.8	75.2

For Durations 1 hour to 72 hours

Duration (hrs)	Return Period (Years) 2	5	10	20	50	100
1	17.4	24.4	31	40	57	75
2	23.3	33	42	53	73	94
3	27.6	39	49	62	85	108
6	37	53	66	82	109	135
12	49	71	89	109	141	169
18	59	85	106	129	163	193
24	66	96	119	145	181	212
36	78	115	142	170	210	242
48	88	130	160	191	233	266
72	105	155	190	226	271	303

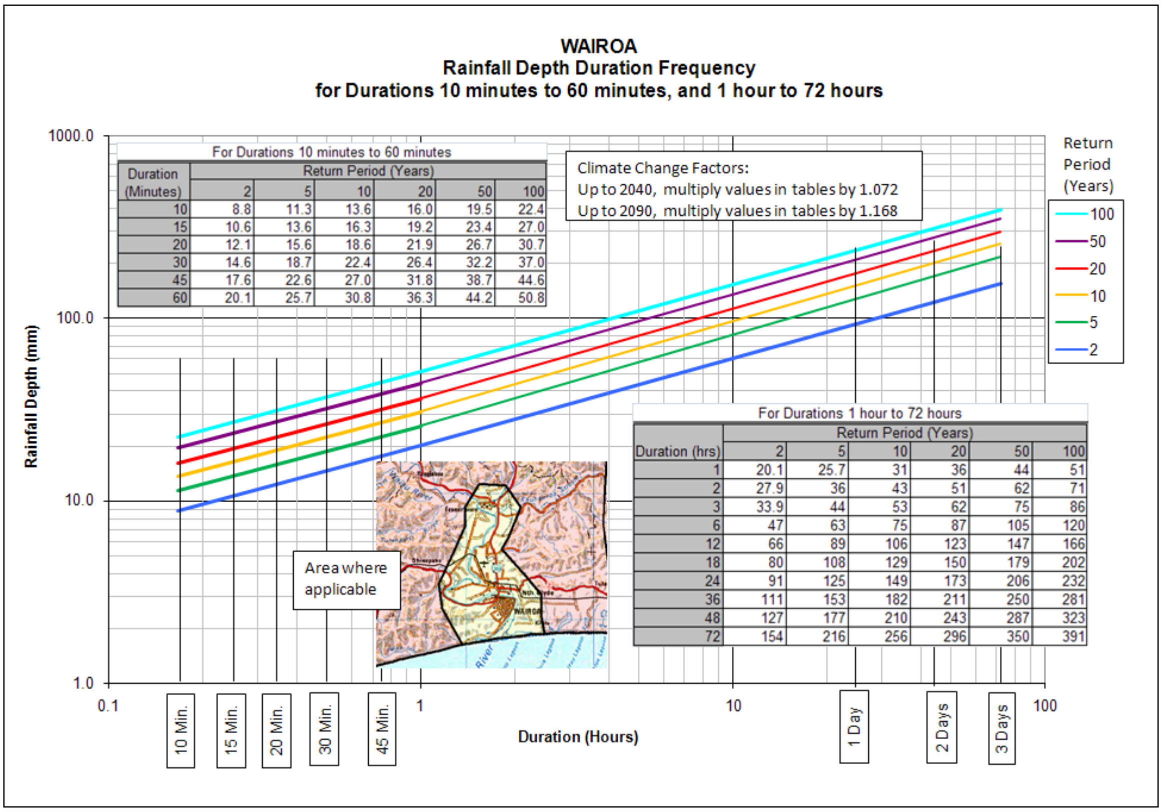


For Durations 10 minutes to 60 minutes

Duration (Minutes)	Return Period (Years) 2	5	10	20	50	100
10	7.7	11.9	14.9	17.7	21.4	23.7
15	9.3	14.2	17.9	21.7	27.3	31.5
20	10.6	16.0	20.3	25.1	32.4	38.5
30	12.7	19.1	24.4	30.8	41.4	51.0
45	15.3	22.7	29.3	37.8	52.9	67.7
60	17.5	25.7	33.4	43.7	62.9	82.7

For Durations 1 hour to 72 hours

Duration (hrs)	Return Period (Years) 2	5	10	20	50	100
1	17.5	25.7	33	44	63	83
2	23.1	33	43	55	76	97
3	27.2	39	49	62	84	106
6	36	51	63	78	101	124
12	47	66	80	97	122	144
18	56	77	93	110	136	158
24	62	86	103	121	147	169
36	73	100	119	138	164	185
48	82	112	132	151	177	197
72	97	131	152	172	197	216

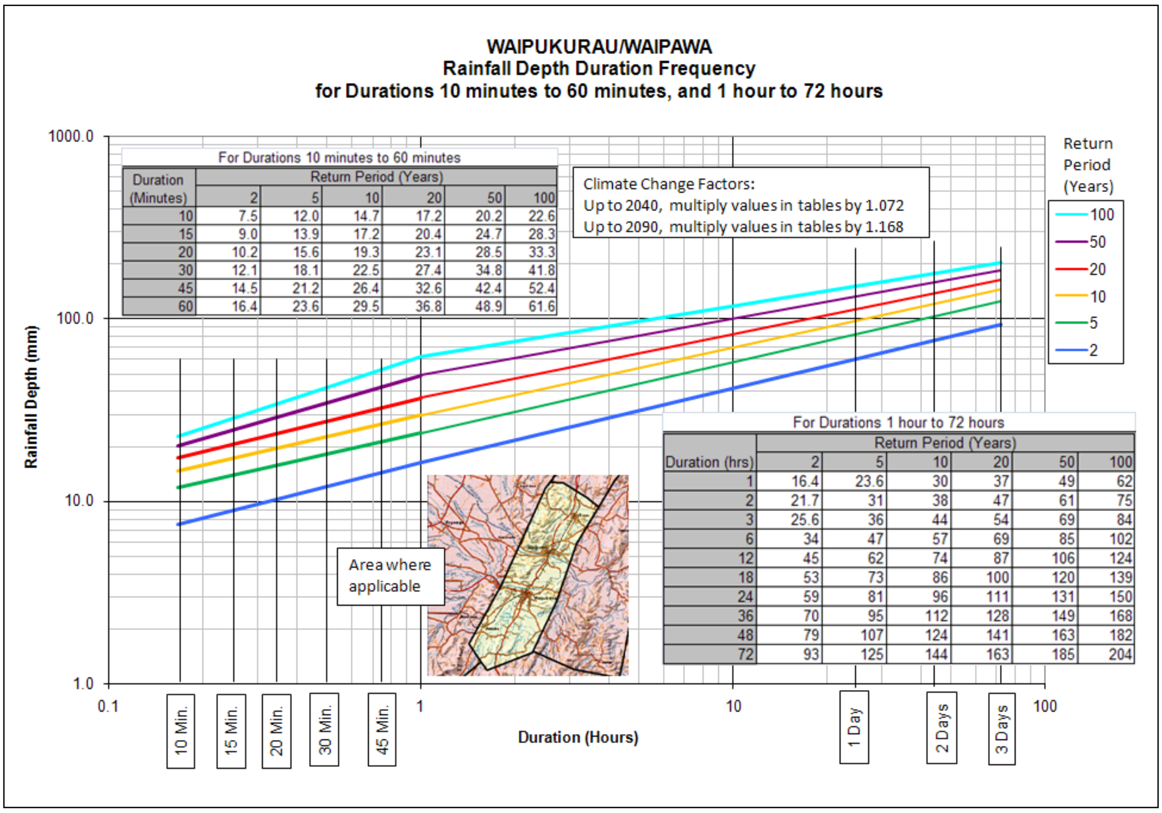


For Durations 10 minutes to 60 minutes

Duration (Minutes)	Return Period (Years) 2	5	10	20	50	100
10	8.8	11.3	13.6	16.0	19.5	22.4
15	10.6	13.6	16.3	19.2	23.4	27.0
20	12.1	15.6	18.6	21.9	26.7	30.7
30	14.6	18.7	22.4	26.4	32.2	37.0
45	17.6	22.6	27.0	31.8	38.7	44.6
60	20.1	25.7	30.8	36.3	44.2	50.8

For Durations 1 hour to 72 hours

Duration (hrs)	Return Period (Years) 2	5	10	20	50	100
1	20.1	25.7	31	36	44	51
2	27.9	36	43	51	62	71
3	33.9	44	53	62	75	86
6	47	63	75	87	105	120
12	66	89	106	123	147	166
18	80	108	129	150	179	202
24	91	125	149	173	206	232
36	111	153	182	211	250	281
48	127	177	210	243	287	323
72	154	216	256	296	350	391



For Durations 10 minutes to 60 minutes

Duration (Minutes)	Return Period (Years) 2	5	10	20	50	100
10	7.5	12.0	14.7	17.2	20.2	22.6
15	9.0	13.9	17.2	20.4	24.7	28.3
20	10.2	15.6	19.3	23.1	28.5	33.3
30	12.1	18.1	22.5	27.4	34.8	41.8
45	14.5	21.2	26.4	32.6	42.4	52.4
60	16.4	23.6	29.5	36.8	48.9	61.6

For Durations 1 hour to 72 hours

Duration (hrs)	Return Period (Years) 2	5	10	20	50	100
1	16.4	23.6	30	37	49	62
2	21.7	31	38	47	61	75
3	25.6	36	44	54	69	84
6	34	47	57	69	85	102
12	45	62	74	87	106	124
18	53	73	86	100	120	139
24	59	81	96	111	131	150
36	70	95	112	128	149	168
48	79	107	124	141	163	182
72	93	125	144	163	185	204

In areas outside the urban centres shown in the above figures, design rainfall values may be generated from the HIRDS (High Intensity Rainfall Design) software available from Niwa. The HIRDS software calculates the values for 10-minute to 72-hour rainfall intensities and for the 2 to 150 year return period intervals.

For large catchments, the areal rainfall distribution will vary; therefore some judgement is required in developing an average for the catchment by weighting similar areas within the catchment or by comparing similar catchments that may be gauged nearby. Likewise, in areas where there is steep topography serving to enhance orographic effects (intensification of rainfall in high country areas), adjustments may be required. Consultation with the HBRC will ensure appropriate design rainfall values are used.

6.1.4 Hydrologic design method

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

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

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

The Rational Formula is the following:

Q_{wq} = CIA/360

Q_{wq} = peak discharge
C = Runoff coefficient (see Tables 6-1a and 6-1b)
I = Rainfall intensity (mm/hr)
A = catchment area in hectares

The peak discharge should be calculated for pre-development and post development conditions.

Table 6-1a Rational Formula Runoff Coefficients (C Factors)
(from Compliance Document for New Zealand Building Code, Clause E1, Surface Water, Department of Building and Housing 2006)

Description of Surface	**Runoff Coefficient (C)**
Natural Surface Types	
Bare impermeable clay with no interception channels or runoff control	0.70
Bare uncultivated soil of medium soakage	0.60
Heavy Clay soil types:	
- Pasture and grass cover	0.40
- Bush and scrub cover	0.35
- Cultivated	0.30
Medium soakage soil types:	
- Pasture and scrub cover	0.30
- Bush and scrub cover	0.25
- Cultivated	0.20
High soakage gravel, sand and volcanic soil types:	
- Pasture and scrub cover	0.20
- Bush and scrub cover	0.15
- Cultivated	0.10
Parks Playground and reserves	
- Mainly grassed	0.30
- Predominately bush	0.25
Gardens, lawns etc.	0.25

Table 6-1a Rational Formula Runoff Coefficients (C Factors) (from Compliance Document for New Zealand Building Code, Clause E1, Surface Water, Department of Building and Housing 2006)	
Developed Surface Types	
Fully roofed and/or sealed developments	0.90
Steel and non-absorbent roof surfaces	0.90
Asphalt and concrete paved surfaces	0.85
Near flat and slightly absorbent roof surfaces	0.80
Stone, brick and precast concrete paving panels	
- with sealed joints	0.80
- with open joints	0.60
Unsealed roads	0.50
Railway and unsealed yards and similar surfaces	0.35
Land Use Types	
Industrial, commercial, shopping areas and townhouse developments	0.65
Residential areas in which the impervious area is less than 36% of gross area	0.45
Residential areas in which impervious area is 36% to 50% of gross area	0.55

For catchments having a variety of different surface types, the runoff coefficient shall be determined by averaging the value for the individual parts of the catchment by using the formula:

$$C = \sum_{i=1}^{n} \frac{C_i A_i}{A_c}$$

C = the runoff coefficient for the catchment
C_i = the runoff coefficient for a particular surface type
A_i = the area of land to which C_i applies
A_c = the catchment area
n = the number of individual surface types

The values of runoff coefficient given in Table 6-1a shall be adjusted for slope in accordance with values shown in Table 6-1b.

Table 6-1b Slope Correction for Runoff Coefficients (from Compliance Document for New Zealand Building Code, Clause E1, Surface Water, Department of Building and Housing 2006)		
Ground Slope	**Adjust C by:**	**Amount**
0-5%	Subtracting	0.05
5-10%	No adjustment	
10-20%	Adding	0.05
20% or steeper	Adding	0.10

6.1.4.1 Storm duration and time of concentration

In calculating the peak discharge, the storm duration is normally taken as equal to the time of concentration (T_c) of the catchment.
For the purposes of these guidelines the **Time of Concentration, T_c** can have two related definitions. These are:

1. The time taken for equilibrium of the catchment to be reached under steady rainfall excess (where outflow from the catchment is equal to the rainfall excess onto the catchment).
2. The time for a kinematic wave to propagate from the hydraulically most distant point in the catchment to the outlet.

The important distinction is that it is the time for a kinematic wave to travel the catchment length, not the travel time for a parcel of water, which is slower.

Kinematic refers to steady uniform flow where the friction or energy slope equals the slope of the bed and the discharge is a function of the depth only. A wave is a variation in flow such as a change in flow rate or water surface elevation. The kinematic wave methodology is a recommended procedure for larger projects, which, if correctly carried out, will provide a suitable estimate for T_c. This method is explained in some hydrological textbooks and not covered in these guidelines.

For the purposes of this toolbox T_c is calculated as the following:

Time of Concentration Formulae

1. **Ramser – Kirpich**

$T_c = 0.0195\ L^{0.77} S_a^{-0.385}$

Where T_c = time of concentration (minutes)
S_a = average channel slope (m/m)
L = flow length from the study location to the farthest point in the catchment (m)

2. **Bransby - Williams**

$T_c = (0.953\ L^{1.2}) / (A^{0.1}\ H^{0.2})$

Where T_c = time of concentration (hours)

A = catchment area (km^2)
L = maximum flow length (km)
H = the difference in elevation between the highest and lowest points in the study area (m)

3. **U.S. Soil Conservation Service** (less than 800 ha)

$$T_c = (0.87\ L^3 / H)^{0.385}$$

Where T_c = time of concentration (hours)
L = maximum flow length (km)
H = the difference in elevation between the highest and lowest points in the study area (m)

These three formulae are empirical and often one will differ markedly from the others. They should not be averaged. The procedure is to examine the average flow velocities (flow length / T_c) and select one that appears reasonable for the catchment and the design storm. The selection may require some previous experience. If in doubt, for small projects the use of the Ramser-Kirpich is preferred.

For small catchments the NZ Building Code: Surface Water Clause E1 determines the time of concentration by a method that considers time of entry and time of network flow and it may be appropriate to use this method in some designs.

Unless a greater duration is indicated by the site analysis, the storm duration to use for peak control purposes is the 1-hour storm.

6.1.4.2 Volume needs for storage

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

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

Where $V_{estimated}$ = required storage volume (m^3)
Q_{post} = Post-development peak discharge rate (m^3/s)
D = Duration of storm (sec)

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

The calculation should be done for 2- and 10-year storms when peak control is required for those intermediate storms.

6.1.5 Effects of climate change

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

In terms of rainfall around the country (Ministry for the Environment, 2008), mean rainfall will vary around the country, and with season. Decreases in annual mean are expected in Hawke's Bay. In terms of extreme rainfall, heavier and/or more frequent extreme rainfalls are expected; especially where mean rainfall increase is predicted.

The 2- and 10-year ARI daily storm events are used to confirm a device's ability to convey peak flows under moderately severe conditions. For device components with a design life greater than 25 years the storm event precipitation values (2- and 10-year) should be adjusted to account for climate change. The values obtained from the 2- and 10-year rainfall maps in Appendix A should be increased by the percentages listed in Table 6-2 unless locally, more detailed data provides more accurate recommendations.

Table 6-2 Factors (percentage adjustments) for Use in Deriving Extreme Rainfall Information for Screening Assessments (Table 5.2 from MfE, 2008)

ARI (years) → **Duration ↓**	**2**	**5**	**10**	**20**	**30**	**50**	**100**
< 10 minutes	8.0	8.0	8.0	8.0	8.0	8.0	8.0
10 minutes	8.0	8.0	8.0	8.0	8.0	8.0	8.0
30 minutes	7.2	7.4	7.6	7.8	8.0	8.0	8.0
1 hour	6.7	7.1	7.4	7.7	8.0	8.0	8.0
2 hours	6.2	6.7	7.2	7.6	8.0	8.0	8.0
3 hours	5.9	6.5	7.0	7.5	8.0	8.0	8.0
6 hours	5.3	6.1	6.8	7.4	8.0	8.0	8.0
12 hours	4.8	5.8	6.3	7.3	8.0	8.0	8.0
24 hours	4.3	5.4	6.3	7.2	8.0	8.0	8.0
48 hours	3.8	5.0	6.1	7.1	7.8	8.0	8.0
72 hours	3.5	4.8	5.9	7.0	7.7	8.0	8.0

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

In Hawke's Bay the increase in annual mean temperature up to the year 2090 is expected to be 2.1^{o}C. While the annual average rainfall is expected to decrease slightly the intensity of storms is expected to increase. The values in Table 6-2 should be multiplied by 2.1 to provide an expectation of rainfall for a given storm. As an example, for a 10-year 1-hour rainfall, the rainfall taken from HIRDS should be increased by 15.5% (7.4x2.1) to account for global warming.

6.1.6 Recommendation for storm peak discharge control

There are three recommendations related to peak discharge control:

- Where there are existing flooding problems downstream and in the absence of a catchment study that evaluates a potential project in a given location and depending on the location of a project within a catchment (per Section 7.1.2), it is recommended that the post-development peak discharge for the 100-year storm for a new project be limited to 80% of the pre-development peak discharge.
- In terms of intermediate storm control, it is recommended that the 2- and 10-year post-development peak discharges not exceed the 2- and 10-year pre-development peak discharges. Section 6.1.3 has 2- and 10-year 24-hour storm data that can be used for design purposes.
- In addition, the rainfall data for the 2- and 10-year storms should be increased by the percentages shown in Table 6-2 unless locally generated data provides more specific information.

6.2 Stream Channel Erosion

Urban development has the effect of increasing the frequency and magnitude of stormwater flows, particularly during frequent, small storm events. As a consequence, streams suffer stability problems.

The composition of the stream banks and bed are the key factors in stream erodibility. Erosion occurs when the shear stress (the “force” of water flowing along the bed and banks) exceeds the ability of the banks or bed to withstand it. Stream erosion is sensitive to changes in the magnitude of flood flows (Beca, 2001).

Scientists engaged in the study of stream erosion for the most part agree on the primary cause of stream erosion. One study out of the U.S. (Julian and Torres, 2005) concludes that hydraulic bank erosion is dictated by flow peak intensities. A more accurate approach to stream erosion is based on shear stress. In principle, the total shear stress on the bed of a stream is the average stress over the bed of a stream (τ - N/m^2) that resists the gravitational forces on the water under uniform conditions (Jowett, Elliott, 2006). In practice, shear stress is difficult to calculate because the water surface slope or energy slope varies across and along the reach of a river.

That being the case permissible velocities can be established to control stream erosion. Table 6-3 provides information on permissible velocities that limit stream channel erosion concerns.

Table 6-3 Maximum Permissible Velocities (Fortier and Scobey (1926)	
Material	Velocity (m/s)
Fine sand (colloidal)	0.46
Sandy loam (noncolloidal)	0.53
Silt loam (noncolloidal)	0.61
Alluvial silt (noncolloidal)	0.61
Ordinary firm loam	0.76
Volcanic ash	0.76
Fine gravel	0.76
Stiff clay	1.14
Graded loam to cobbles (noncolloidal)	1.14
Alluvial silt (colloidal)	1.14
Graded silt to cobbles (colloidal)	1.22
Coarse gravel (noncolloidal)	1.22
Cobbles and shingles	1.52
Shales and hard pans	1.83

A compounding factor relating to stream erosion depends on whether the stream has a floodplain or is an incised gully with channel flow whose depth depends on the amount of water being transported. In situations where there is a floodplain, the erosion potential does not increase significantly once the flow spreads out over the floodplain. As flows increase, the flow spreads out on the floodplain and the depth of flow and velocity do not significantly increase. On the other hand, flow in incised channels progressively increases in velocity and depth as flow increases and leads to further increases in erosion potential.

When addressing stream erosion concerns, there are two methods for meeting erosion control objectives:

- Runoff volume control
- Detention time control

The approach to addressing stormwater criteria for each of these situations is considered individually.

6.2.1 Runoff volume control

The volume of runoff can be used as a criterion for developing an erosion control recommendation. It is necessary to specify both the volume (or depth) of runoff to be stored and the duration over which this volume may generally be infiltrated into the ground. A given volume of runoff might be specified for retention and that runoff must pass through the retention system and infiltrate in a given period of time, which would depend on the inter-event time period during that time of year when the average inter-event dry period is least. An example of this is that storms in Auckland during winter months occur approximately every two days. In that scenario, the retained volume must be drained within 48 hours to ensure that the storage volume is available for the next storm.

6.2.2 Detention time control

An alternative to runoff volume control is to establish an extended detention time, which is the time interval between the times of the inflow and outflow hydrographs when a defined percent of the volume has been discharged. In this situation duration of flow is recommended that effectively separate the detained flow from the storm hydrograph. A general recommendation of 24 hours is recommended to achieve this separation.

6.2.3 General Discussion

The intent of volume control or extended detention is to not initiate or aggravate existing stream channel erosion. By reducing the total volume of water running off the land or extending the time that flows takes to travel through the catchment, channel erosion potential is reduced. Figure 6-3 (McCuen, 1987) provides a visual representation of that intent.

In general, the figure relates flow discharge with flow duration. As discussed prior in Section 6.2, peak rates of flow and higher velocities potentially cause channel erosion. Figure 6-3 shows three lines and those lines represent: pre-development flows without extended detention, post-development flows without extended detention, and the post-development condition with extended detention. If channel erosion were at a given flow rate (say 30 m^3/sec) the red line would indicate where the flow becomes erosive. Both pre- and post-development conditions cause stream erosion while the extended detention discharge is below the erosion threshold.

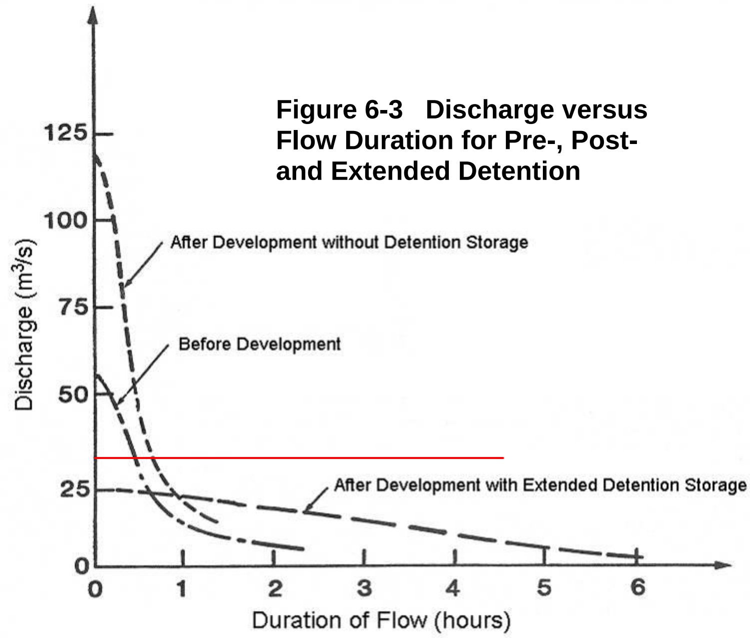


Figure 6-3 Discharge versus Flow Duration for Pre-, Post- and Extended Detention

While it is recognised that erosion is a natural process, the intent of volume control or an extended detention criteria is to prevent accelerated level of erosion as a result of increased catchment imperviousness.

There are two questions that need to be addressed:

- What criteria should be established, and
- Where should the criteria be applied

6.2.4 What criteria should be established?

An overseas study (McCuen, 1987) for the case of noncohesive sediments suggested that the runoff discharged from a detention basin for the post-development conditions and a 2-year, 24-hour rainfall should not exceed 25 mm over the 24-hour duration of the design storm. The discharge approximates that of a water quality storm. Work done by Beca, (2001) indicated that for cohesive soils the discharge from a detention basin should not exceed 30 mm over the 24-hour duration storm or within a maximum peak outflow of 7.5 L/s/ha. Beca also recommended having an active storage requirement of up to 130% of the water quality volume as being required to achieve erosion control in cohesive soils.

Another option to specific criteria would be for the project designer to calculate the receiving stream shear stress in the pre- and post-construction condition. If the stream is stable then maintain the pre-development peak flow rate and shear stress. If this analysis becomes too complicated then a generalised level of control is recommended.

For the purposes of this Guideline, erosion control criteria is regarded as 1.2 times the water quality volume that should be live storage provided within the stormwater management practice to be infiltrated or released over a 24-hour period.

6.2.5 Where the criteria should be applied?

The criterion applies to natural (earthen) streams only. It does not have the same limitations or restrictions as peak flow control (top half of catchment), so will be generally recommended throughout a catchment. At the very bottom end of a catchment it is recommended that shear stress analyses be done to determine whether volume control or extended detention is required.

Once tidal limits are reached, there is no need to consider extended detention.

Another situation is where catchment slopes are very slight and velocities of flow are under those provided in Table 6-2. An example of this situation is around Napier or Hastings. In those areas, getting the water off the land is the problem and stream velocities for the 2-year storm may be below the permissible velocities.

6.2.6 Water quality credit for extended detention

One benefit of providing extended detention for stream channel erosion control is that storing and releasing of stormwater over a 24-hour period will provide improved sedimentation due to gravitational sedimentation over that time period. As a result, when used in conjunction with a wet pond or wetland the permanently stored volume calculated for water quality control can be reduced by 50% due to a water quality credit provided by the extended detention. This credit is provided if the criteria provided in Section 6.2.4 are followed.

6.2.7 Recommendations for stream erosion control

The following recommendations are made to address stream channel erosion.

6.2.7.1 Erosion control criteria

There are three different approaches that can be taken to address stream channel erosion:

1. Check the 2-year stream velocities against Table 6-2 to ensure that velocities are non-erosive. If they are non-erosive in the post-development condition assuming ultimate development of the catchment under the appropriate district plan land use, then no extended detention is required.
2. Implement extended detention or volume control according to the following:
   - If the stream is stable under the existing development condition, design detention or retention storage for a 24-hour release of an equivalent volume to the water quality storm.
   - If the stream is not stable, multiply the water quality volume by 1.2 to determine the extended detention volume. That volume is then stored and released over a 24-hour period.
3. Conduct a shear stress analysis for a specific site doing the following:
   - Conduct catchment modelling, i.e. continuous simulation, using land use, initial losses and time of concentration for the catchment in the pre-development condition without the proposed project. Another simulation will then have to be done for the catchment with the development in place.
   - Input climate information including evaporation data and long-term rainfall.
   - Identify a typical downstream cross-section, slope bed material and channel roughness.
   - Apply standard channel hydraulics to the cross-section to get a relationship between the discharge and shear stress.
   - Develop the relationship between shear stress and erosion rate.
   - Combine this with the discharge/shear stress relationship to get a discharge/erosion relationship.
   - Apply the output hydrographs from the hydrological simulations to get the discharge/erosion curve to get the long-term time series of erosion rate.
   - Calculate the long-term erosion with and without the new development to determine whether the project will make erosion worse.

Volume control uses the same volumes as recommended for detention but then infiltrates or otherwise uses (water tanks, designed evapotranspiration) the runoff.

6.2.7.2 Where applicable

Stream erosion issues are applicable where:

- There is a new project, and
- There is a natural stream, and
- Catchment imperviousness exceeds 3%, and
- There is potential for future development to increase stream channel instability, and
- There is no tidal influence to the stream where the new development discharges to it

6.3 Water Quality Design

There are several items that need to be considered when discussing stormwater quality design. These items include:

- General sizing requirements
- Effluent limits versus best practicable option (BPO)

6.3.1 General sizing requirements

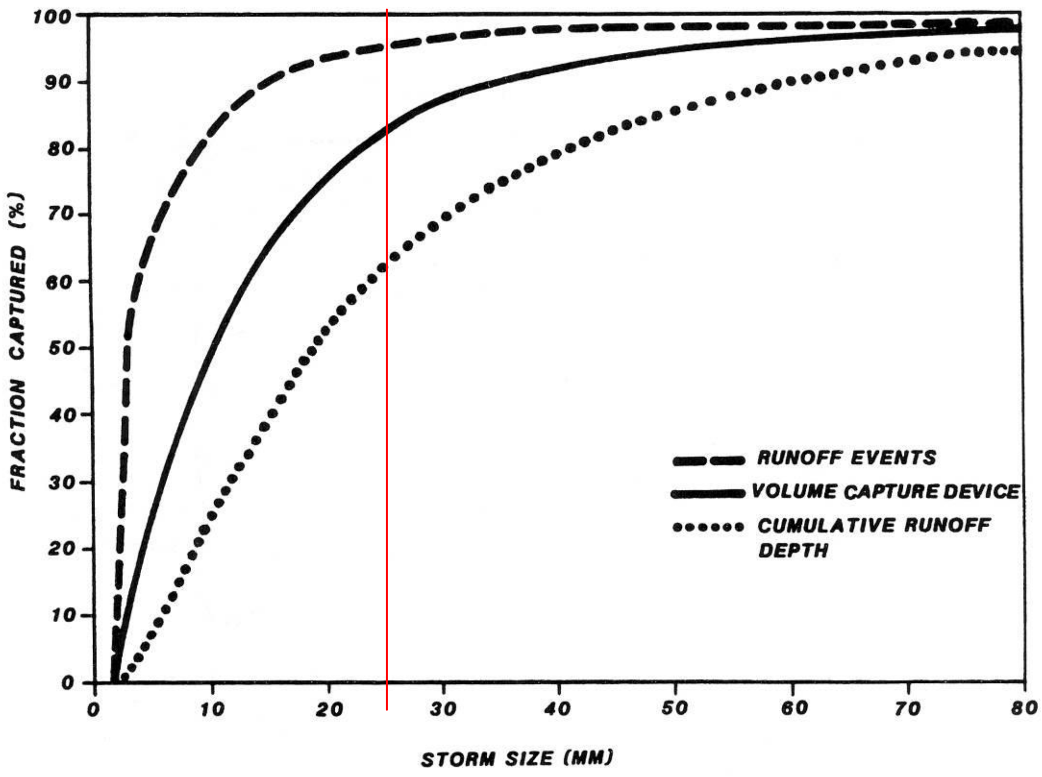
The size of stormwater runoff event to be captured and treated is a critical factor in the design of stormwater quality treatment practices. If the design runoff event is too small, the effectiveness of the practice will be reduced because too many storms will exceed the capacity of the practice. If the design event is too large, the smaller runoff events will tend to empty faster than desired or the cost of the practice will be greater than the benefit that it provides.

Analytical work to determine optimal policies for rainfall capture (Clar and Barfield, 2004) has indicated that there is a maximised point of runoff volume capture at approximately the 90-percentile storm. The 90-percentile storm is that storm where 90% of all storms on an annual basis are less than. The use of the 90% storm has become widespread throughout the U.S.

In the Auckland Region, similar work was done based on rainfall information taken from the Botanic Gardens at Manurewa (1983 - 1990) (ARC, 1992). The frequency distribution of rainfall for events greater than 2 mm is shown in Figure 6-4. As an example of the information gained by the use of the Figure, the distribution indicates that for a storm depth of 25 mm:

- 95% of events would have a lesser depth,
- 80% of the storm volume would be captured if a device could capture up to 25 mm of rainfall
- Events with a total rainfall depth less than 25 mm have a cumulative rainfall depth of 60% of total rainfall.

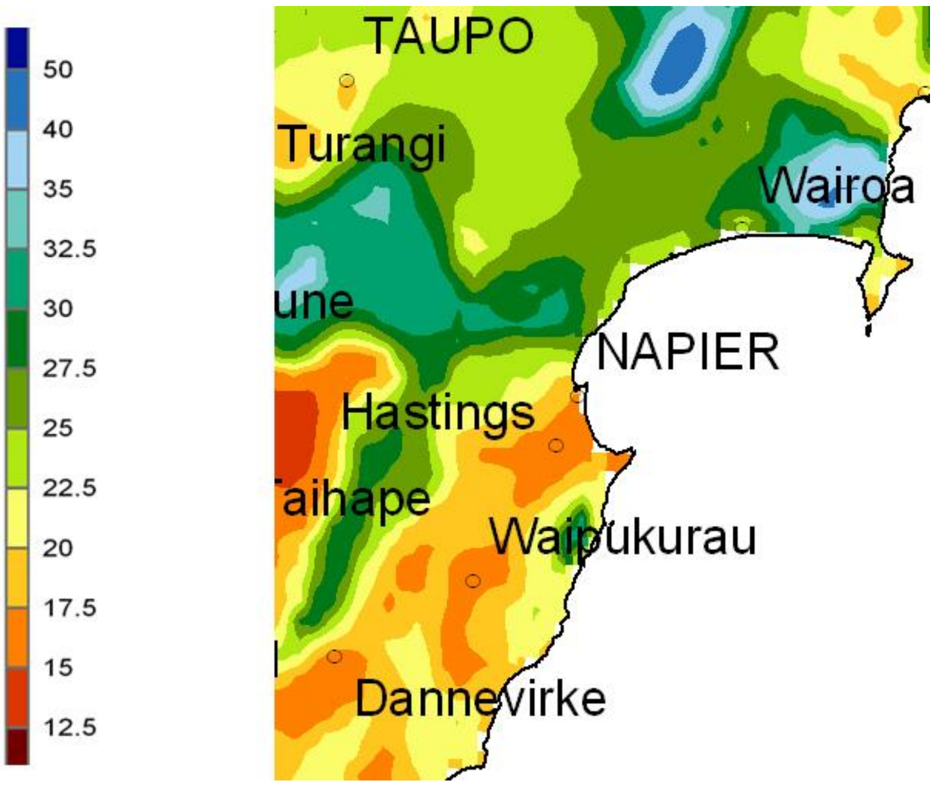
Figure 6-4 Frequency Distribution of Runoff events



As shown with the red line, the ARC used the 95% storm, or 25 mm, as the water quality storm based on the Manurewa site. Due to rainfall differences throughout the Region, the water quality storm varies somewhat from the Manurewa site depending on the site design location. The ARC has addressed this issue by developing rainfall maps for the Region and extrapolating the water quality storm from the 2-year frequency rainfall map. The 2-year rainfall event at Manurewa is 75 mm of rainfall over a 24-hour period. As the 95% storm is 25 mm of rainfall, the water quality storm is 1/3 of the 2-year storm rainfall. That ratio is used throughout the Region and the localised water quality storm is 1/3 of the mapped 2-year event.

NZWERF provides indicative values of 1/3 of the 2-year rainfall for a number of locations around the Country (NZWERF, 2004) as a suggestion of possible magnitudes for a water quality storm. When those values are considered in conjunction with the 90% storm values provided in Figure 6-5, the values are consistent. There is a slight difference from the ARC 95% values but there is no inconsistency with 1/3 of the 2-year storm value.

Figure 6-5
90% rainfall event depth for the Hawke's Bay Region



The benefit of the 90% map over the NZWERF information is that the map provides overall coverage, where the NZWERF values are for specific locations. As such, using the 90% map would be a more accurate approach.

6.3.1.1 High rainfall areas

There are a number of areas (as shown in Figure 6-5) where the 90% storms are very high. If the 90% storm were used for water quality treatment, the sizing of the water quality practices would be extremely large. Recognising this issue, the water quality storm for all of those areas having a 90% storm greater than 30 mm of rainfall is 30 mm of rainfall for design purposes.

The reason for selection of this storm depth is that it is a moderate requirement and should not represent a significant burden for implementation. The higher rainfall will add a dilution factor that limits adverse effects, which offsets the smaller water quality storm treatment requirement.

It is important in those areas that flow in excess of the water quality storm should not be directed to the water quality practice and should bypass the practice to prevent resuspension of captured contaminants. Where swales or filter strips are the practice of choice, it may be impossible to bypass larger flows. In those situations the velocity during larger storms must not exceed 1.5 m/s to prevent resuspension.

6.3.2 Effluent limits versus BPO

There is always discussion when stormwater technical people gather as to whether effluent limits should be used to determine design or whether a BPO approach is more appropriate.

6.3.2.1 Effluent limits

There is little doubt that the time is approaching when effluent limits can be used to ensure resource protection is achieved but that day depends on having good information related to receiving systems and good stormwater practice performance information. In a very localised situation, that information may be available but even then there may be difficulty in targeting contaminant sources and their relative contribution.

There may also be difficulty in assigning a storm frequency for which the effluent limit cannot be exceeded.

A variation to using effluent limits that may have more promise relates to measuring receiving system sediment contaminant concentrations. With this approach, each land use would have assigned annual loads that should not be exceeded to ensure that receiving system sediment contaminant concentrations do not increase. This would then need to relate to implementation of stormwater management practices and their ability to reduce annual loads. More information is needed on stormwater practice performance to predict annual loads.

More background information would also need to be generated for the sediment concentration approach to be implemented.

6.3.2.2 BPO approach

As defined in the RMA, best practicable option means the best method for preventing or minimising the adverse effects on the environment having regard, among other things, to:

- The nature of the discharge or emission and the sensitivity of the receiving environment to adverse effects, and
- The financial implications, and the effects on the environment, of that option when compared with other options, and
- The current state of technical knowledge and the likelihood that the option can be successfully applied.

The BPO approach, while being somewhat vague in providing a water quality target, provides the flexibility for a given approach to be used and, at a minimum, increase the time frame before sediment contamination levels approach the Australian and New Zealand Environment and Conservational Council (ANZECC, 2000) Interim Sediment Quality Guidelines (ISQG) low or high values.

It is difficult to go beyond the BPO approach for stormwater treatment until more information is gained on stormwater practice performance and more information is gained on receiving system sediment contaminant levels and accumulation.

6.3.3 **Recommendations for water quality control**

The following recommendations are made:

- The 90% storm map provided above be used for determining water quality treatment volumes and flow rates in sizing stormwater management practices,
- In regions where the 90% storm is greater than 30 mm, water quality treatment will use 30 mm of rainfall for design purposes.
- The BPO approach be used for stormwater management practice design.

6.3.4 **Calculating water quality volumes**

The Rational Formula does not calculate volumes of runoff but rather calculates peak discharges for various storm intensities. Calculate the water quality volume to be treated by using the 90% storm as shown in Figure 6-5. The City of Christchurch has a simple method of determining the first flush volume in their Waterways, Wetlands and Drainage Guide (2003) where the water quality volume (their first flush volume) is based on the following:

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

The first flush volume $V_{wq} = 10 \times A_{eff} \times d_{ff}$ (m^3)
Where d_{ff} = first flush water quality depth (water quality storm)

Use this method to calculate the water quality volume storage.

6.4 Modelling Methods

There are a number of different approaches that can be used for design of stormwater management practices and several of the more common approaches are the following:

- The Rational Method
- The Regional Flood Frequency Formula, also known as the Regional Method, and
- The Unit Hydrograph Method

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

The Regional Method was derived from data for medium to large inland hill catchments and its accuracy is limited to those catchments. It is not recommended in small catchments, coastal areas and urban areas.

The Unit Hydrograph Method can be applied to both urban and rural catchments of various sizes. The Unit Hydrograph method is very appropriate for the following:

- Design of retention and detention structures including low impact design systems such as swales and rain gardens,
- Routine design of minor to medium works in small to medium sized catchments,
- Calculation of flow rates, volumes and times of concentration.

Probably the most common form of a Unit Hydrograph design approach is the U.S. Natural Resources Conservation Service method. This method has been adapted for use in the Auckland Region and by the Kapiti Coast District Council.

The difficulty in using the Unit Hydrograph approach is that the unit hydrograph has to be based on local data. A 24-hour storm has to be developed and normalised based on long-term rainfall records representative of the area where the analyses are being done. Not developing such normalised rainfall intensity will result in incorrect results. Normalised rainfall intensity unit hydrographs have not been developed for most of New Zealand.

There are a number of other design approaches that can be used, a number of which will give reasonable results. An important distinction with this analysis is that the main emphasis is on determining the relative difference between pre- versus post-development runoff. It is not the intention to provide a hydrological method for overall catchment analyses. The method must provide for peak discharges, intermediate storm volumes and water quality volumes.

6.5 Summation of Recommendations

6.5.1 **Peak flow control**

There are three recommendations related to peak discharge control:

- Where there are existing flooding problems downstream and in the absence of a catchment study that evaluates a potential site in a given location and depending on the location of a project within a catchment, it is recommended that the post-development peak discharge for the 100-year storm for a new site be limited to 80% of the pre-development peak discharge.

- In terms of intermediate storm control, it is recommended that the 2- and 10-year post-development peak discharges not exceed the 2- and 10-year pre-development peak discharges.
- In addition, the rainfall data for the 2- and 10-year storms should be increased by the percentages shown in Table 6-2 unless locally generated data provides more specific information in a given region.

6.5.2 Stream erosion control

The following recommendations are made to address stream channel erosion.

6.5.2.1 Erosion control Criteria

There are three different approaches that can be taken to address stream channel erosion:

3. Check the 2-year stream velocities against Table 6-2 to ensure that velocities are non-erosive. If they are non-erosive in the post-development condition assuming ultimate development of the catchment under the appropriate district plan land use, then no extended detention is required.
4. Implement extended detention or volume control according to the following:
    - If the stream is stable under the existing development condition, design detention or retention storage for a 24-hour release of an equivalent volume to the water quality storm.
    - If the stream is not stable, multiply the water quality volume by 1.2 to determine the extended detention volume. That volume is then stored and released over a 24-hour period.
3. Conduct a shear stress analysis for a specific site doing the following:
    - Conduct catchment modelling, i.e. continuous simulation, using land use, initial losses and time of concentration for the catchment in the pre-development condition without the proposed project. Another simulation will then have to be done for the catchment with the development in place.
    - Input climate information including evaporation data and long-term rainfall.
    - Identify a typical downstream cross-section, slope bed material and channel roughness.
    - Apply standard channel hydraulics to the cross-section to get a relationship between the discharge and shear stress.
    - Develop the relationship between shear stress and erosion rate.
    - Combine this with the discharge/shear stress relationship to get a discharge/erosion relationship.
    - Apply the output hydrographs from the hydrological simulations to get the discharge/erosion curve to get the long-term time series of erosion rate.
    - Calculate the long-term erosion with and without the new project to determine whether the highway will make erosion worse.

Volume control uses the same volumes as recommended for detention but then infiltrates or otherwise uses (water tanks, designed evapotranspiration) the runoff.

6.5.2.2 Where Applicable

Stream erosion issues are applicable where:

- There is a new development project, and
- There is a natural stream, and
- Catchment imperviousness exceeds 3%, and
- There is potential for future development to increase stream channel instability, and
- There is no tidal influence to the stream where the new development discharges to it

6.5.3 Water quality control

The following recommendations are made:

- The 90% storm map be used for determining water quality treatment volumes and flow rates in sizing stormwater management practices,
- In regions where the 90% storm is greater than 30 mm, water quality treatment will use 30 mm of rainfall for design purposes.
- The BPO approach be used for stormwater management practice design.

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7 Detailed Stormwater Management Practice Design

7.1 Introduction

The chapters up to now have laid the foundation for the need to consider stormwater management, the types of practices that can be used, analytical approaches and recommendations for the form that management should take from a flooding, erosional and water quality perspective. This chapter is devoted to detailed design approaches for stormwater quantity and quality control.

The chapter will be broken up to discuss several different areas.

- Source control,
- Design for operation and maintenance, and
- Flow and treatment control

7.2 Source control

Prior to any consideration of stormwater treatment, consideration should be given to source control and a series of questions answered.

- Have building materials been used that minimise leaching of contaminants?
- Has existing vegetation been preserved to the degree practicable or has vegetation been re-established upon project completion?
- Are flow velocities and volumes increased downstream (energy dissipation)?
- Has slope disturbance been minimised and have disturbed slopes been vegetated and slope lengths minimised through the use of cut-off drains?
- Can concentrated flow areas be minimised?
- Are any cross drains combined and considered for erosion protection?

When these types of questions have been considered and addressed, the stormwater management practice selection process then moves on to flow and treatment control.

7.3 Design for operation and maintenance

As well as water quality and water quantity control, another key element that <u>must</u> be considered during the design phase is operation and maintenance of the practice. There are several key elements that must be considered during the design phase. Asking and answering some questions or giving serious consideration to operation of the stormwater practice and system can answer them.

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

7.3.1 Spend a year at the practice

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

As these conditions are visualised, the designer should also imagine how they might affect not only the operation of the practice itself, but also the people that will maintain it or otherwise interact with it. Will the outlet structure trash rack be prone to clogging from vegetation or debris floating in the stormwater runoff? Is there a safety issue related to maintenance for maintenance employees.

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

7.3.2 Asking maintenance questions

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

7.3.2.1 Who will perform the maintenance

Does the design of the practice require operation and maintenance specialists or will someone with general maintenance equipment and training be able to accomplish it.

7.3.2.2 What needs to be maintained

A list of practice components that are part of the design may prompt a revised design with either a shorter list or one that modifies a practice component to facilitate maintenance. An example of this could be a sand filter system that has heavy covers that are not easily removed by hand or require a specialised piece of equipment to lift the covers.

7.3.2.3 When will maintenance need to be performed

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

7.3.2.4 Where will maintenance have to be performed

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

7.3.2.5 How will maintenance be performed

The simple instruction to remove sediment or harvest vegetation can become complicated if there hasn't been any provision made to allow equipment access to the practice or even to the site. Are locks used to limit public access to a practice? If security features are used then there has to be a common key to allow easy access. Stormwater practices cannot become a liability to the local community.

7.3.3 Considering the use of uniform materials or components

Specify materials that will last for as long as the life expectancy of the stormwater management practice might be. If further development is anticipated in 15 years than materials used should last 15 years. Reducing construction costs may have a significant adverse impact on long-term maintenance costs.

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

7.4 Flow and Treatment Control

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

- Swales
- Filter strips
- Sand filters
- Rain gardens
- Infiltration
- Wet ponds
- Wetlands
- Green roofs
- Water tanks
- Oil and water separators

These practices are seen as applicable for new development.

Prioritisation of this list is difficult as each practice has value, but one or more may be more appropriate in a given catchment. For example, swales, filter strips, sand filters, rain gardens and oil and water separators are primarily water quality practices with limited ability to address water quantity issues.

Wet ponds and wetlands can provide good water quantity control but wet ponds have a limited ability to remove hydrocarbons and soluble metals.

One practice that is good for both water quantity and water quality control is wetlands. Their organic substrate, density of vegetation and ability to provide live storage for water quantity control makes them suitable for both water quantity and water quality control. The major drawbacks of wetlands are their occupation of space and the need to have a catchment area large enough to support hydric soils, but they should be considered whenever peak control or stream erosion protection is a project component.

7.4.1 **Swales**

Description: Vegetated swales 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
x	Stream channel erosion protection

Swales are a very appropriate practice for roads and new development. They can easily occupy a linear corridor without taking up much additional space. They can also take the place of conventional stormwater conveyance systems. Although swales may vary in their purpose in different areas, their overall objective is to slow stormwater flows, capture contaminants and possibly reduce the total volume of stormwater runoff.

Water quality treatment can be 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.

A key factor in vegetated swale water quality performance is the residence time that the water takes to travel through the swale. Residence time depends on the following items:

- The longitudinal slope of the swale,
- The cross-sectional area of the swale, and
- 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.

$$V = R^{0.67}s^{0.5}/n$$

Where:
V = Average velocity in metres/sec.
R = the hydraulic radius of the swale in metres
s = slope of the swale in metres/metre
n = Manning coefficient of roughness

Residence time can then be determined by the following equation:

$t = L/V$

Where:

t = residence time in minutes (divide result by 60 sec/m)
V = velocity of flow at the design rate of flow in metres/sec.
L = swale length in metres

7.4.1.1 Basic design parameters

The following Table 7-1 should be used for swale design elements.

Table 7-1 Swale design elements

Design parameter	Criteria
Longitudinal slope	< 5%
Maximum velocity	0.8 m/s for water quality storm
Maximum water depth above vegetation	The water quality design water depth should not exceed design height for grass. This is a key criterion for ensuring Manning roughness coefficient is provided.
Design vegetation height	100 - 150 mm
Manning coefficient	0.25 for WQ storm, 0.03 for submerged flow (10-yr. Storm)
Maximum bottom width	2 m
Minimum hydraulic residence time	9 minutes
Minimum length	30 m
Maximum catchment area served	4 hectares
Maximum lateral slope	0%
Maximum side slope	4 H:1V (shallow as possible for mowing purposes)
Where longitudinal slope < 2%	Perforated underdrains shall be provided
Where longitudinal slope > 5%	Check dams shall be provided to ensure effective slope < 5%
Where concentrated flows enter the swale (from pipes)	Level spreaders shall be placed at the head of the swale to disperse flows
10-year storm velocities	< 1.5 m/s unless erosion protection is provided

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

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

7.4.1.2 Residence time

There have not been many studies that relate water quality performance in swale design. The most recognised work has been done in the U.S. (Metropolitan Seattle, 1992). That study recommended a residence of 9 minutes for flow to pass through the swale and provide approximately an 80^{+}% removal of total suspended solids. Most governmental agencies in the U.S. have adopted that criterion. More recently, the recommendation has been recommended upward to 22 minutes due to the uncertainty of performance (Washington State Department of Ecology, 2001). That change in residence time is a significant change from the 9-minute criterion but it is not recommended that the time be increased until further investigation of swale performance is done in New Zealand.

7.4.1.3 Manning's coefficient of roughness

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

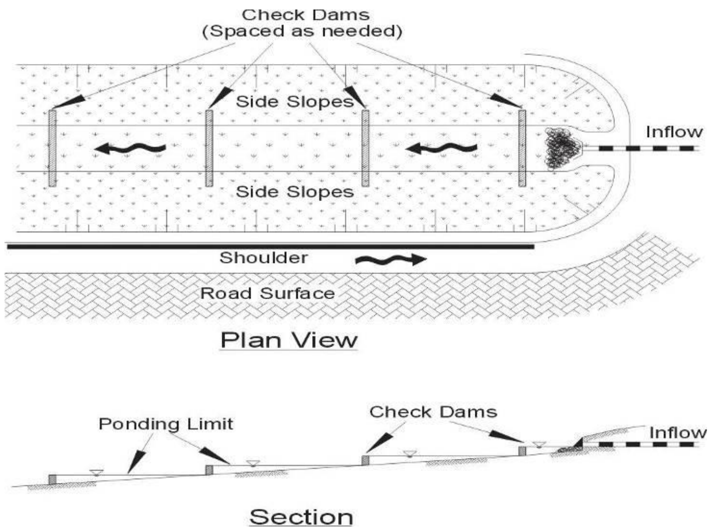
It is recommended that a standardised value for Manning's coefficient of roughness be set at 0.25. It is a mid-point in the Larcombe study and agrees favourably with The California recommendation. It is not felt that using the equations would provide necessarily a better result in design.

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

7.4.1.4 Lateral inflow

A common concern with swales is lateral inflow from site areas to a point where the flow does not achieve the 9-minute residence time. To the degree that the 9 minutes can be attained it should be. An example of this is Figure 7-1 that, in addition to check dams, shows a lateral flow diversion that directs the lateral flow to the head of the swale.

Figure 7-1 Swale with Check Dams and Diversion of Lateral Inflow



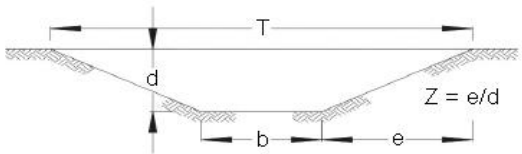
Where lateral inflow cannot meet the nine-minute residence time for part of the alignment, the normal approach is to accept that the average flow through the swale does take nine minutes. There will be areas in the upper part of the swale that may exceed the required residence time so the average is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

7.4.1.5 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. Estimate runoff flow rate from the water quality storm. Use the 90% storm in 6-5 as the water quality storm and use an appropriate hydrologic design procedure for calculation of flows. One difference between swale and filter strip design and other stormwater management practices is that they are designed by flow rate where other practices are designed by calculation of the water quality volume.
2. If using the Rational Method for design, use the entire water quality storm as i. This would be the maximum possible value for discharge calculations and thus be conservative.
3. Establish the longitudinal slope of the swale.
4. Select a vegetation cover. It should be grass and would generally be either perennial rye or fescue.
5. The value for Manning's coefficient of roughness is 0.25
6. Select a swale shape. Two shapes are proposed as they ensure distributed flow throughout the bottom of the swale. Triangular swales are not recommended as they concentrate flow at the bottom of the swale. Channel geometry and equations for calculating cross-sectional areas and hydraulic radius are provided under the individual configurations in Figure 7-2.
7. An assumption is made on the depth of flow in the swale for the water quality storm. This assumed depth is used for calculating the bottom width of the swale and cross-sectional area.
8. Use Manning's equation for calculating dimensions of the swale by using first approximations for the hydraulic radius and dimensions for selected shape.

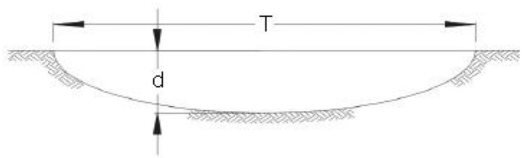
Figure 7-2 Channel Geometry



Cross-sectional area (A) = $bd + Zd^2$

Top width = $b + 2dZ$

Hydraulic Radius (R) = $\dfrac{bd + Zd^2}{b + 2d(Z+1)^{1/2}}$



Cross-sectional area (A) = 1/3 Td

Top width = 1.5A/d

Hydraulic Radius (R) = $\dfrac{T^2 d}{1.5T^2 + 4d^2}$

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

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

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

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

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

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

8. Knowing b (trapezoid) or T (parabola), the cross-sectional area can be determined by the equations in Figure 7-2.
9. Calculate the swale velocity from the following equation:

$$V = Q/A$$

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

L = Vt(60 s/minute)

Where t = residence time in minutes.

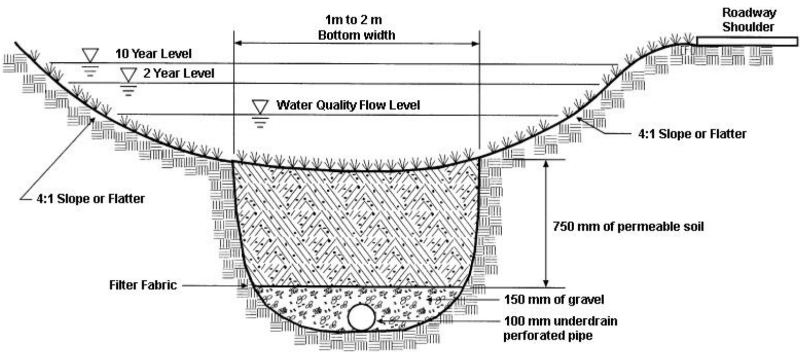
7.4.1.6 Flows in excess of the water quality storm

It is expected that runoff from events larger than the water quality design storm will go through the swale. In that situation, a stability check should be performed to ensure that the 10-year, 1-hour storm does not cause erosion. For the 10-year storm, flow velocities should not exceed 1.5 m/s, although higher velocities may be designed for with appropriate erosion protection. When considering larger storms consideration must be given to increased rainfall values as a result of climate change. Table 6-2 provides global warming design information.

7.4.1.7 Shallow slope situations

Where slopes are less than 2%, an underdrain must be used to prevent soils from becoming saturated during wet times of the year. Figure 7-3 provides a typical cross-section of what the underdrain system should be designed to ensure that water

Figure 7-3 Swale schematic showing soils and underdrain



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.

7.4.1.8 Case Study

Project description

A lane addition to an existing highway for vehicle passing purposes is proposed. The lane is 4 metres wide and 1,000 metres long. The project is located in Hastings.

Hydrology

Using the Rational Formula

Q_{wq} = CIA/360

C = 0.9 (estimate for paved surface)
I = Rainfall intensity (mm/hr.) - for Hastings the water quality storm is 15 mm.
A = catchment area in hectares

$Q_{wq} = (0.9)(15)(0.4)/360 = 0.015\ m^3/s$

For Q_{10} 1-hour storm rainfall is 22.4 mm and Mannings coefficient of roughness is 0.03. Effect of global warming on the 10-year storm is 15.5% increase in rainfall. So 22.4 mm for a 10-year storm increased by 15.5% results in 25.87 mm of rainfall.

$Q_{10} = (.9)(25.87)(0.4)/360 = 0.026\ m^3/s$

Swale Design

Slope of swale alignment = 0.015

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

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

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

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

$b = ((.015)(.25)/(.1^{1.67})(.015^{0.5})) - (4)(.1) = 1.04\ m$

Calculate the top width

$T = b + 2dZ = 1.04 + 2(.1)(4) = 1.84\ m$

Calculate the cross-sectional area

$A = bd + Zd^2 = (1.04)(.1) + 4(.1^2) = 0.144\ m^2$

Calculate the flow velocity

V = Q/A = 0.015/0.144 = 0.1 m/s which is well under than the 0.8 m/s maximum - good.

Calculate the swale length

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

As the swale will probably have larger flows pass through it, the swale design can be adjusted to account for the larger flows. In this situation the Manning coefficient of roughness will have to be decreased, as flow will be above the grass height so assume n = .03 as the vegetation is completely submerged. Solve for d and ensure that velocities are not erosive. Q_{10} = 0.026 m^3/s.

The following Table 7-2 relating flow depth to Manning's n to discharge provides information on swale flow under larger flow conditions.

Table 7-2 Flow Depth vs. Manning's n versus Discharge

Flow depth (m)	Manning's n	Discharge (m^3/s)
0.1	0.25	0.015
0.1 - 0.15	0.03	0.035
Total Discharge		0.051

Even adding only 50 mm to the swale depth provides for conveyance of the 10-year event. In terms of ensuring that the velocity is not greater than 1.5 m/s

Q = AV or Q/A = V = 0.026m^3/s/0.246 = 0.11 m/s so the velocities during the 10-year storm are non-erosive.

7.4.2 Filter Strips

Description: Filter strips are uniformly graded and densely vegetated to 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

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

They are well suited for treating runoff from small impervious surfaces and for use as a pre-treatment device for other practices such as a sand filter or wetland.

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

7.4.2.1 Basic Design Parameters

The following Table 7-3 should be adhered to in designing a filter strip.

Table 7-3 Filter Strip design elements

Design parameter	Criteria
Longitudinal slope	2% - 5%
Maximum velocity	0.4 m/s for water quality storm
Maximum water depth above vegetation	The water quality design water depth should not exceed ½ of the design height for grass. This is a key criterion for ensuring Manning roughness coefficient is provided.
Design vegetation height	100 - 150 mm
Manning coefficient	0.35 for WQ storm, 0.03 for submerged flow (10-yr. Storm)
Minimum hydraulic residence time	9 minutes
Minimum length	Sufficient to attain residence time
Maximum catchment area served	2 hectares
Maximum lateral slope	2%
Where longitudinal slope < 2%	Filter strips are not recommended for slopes less than 2% unless they are designed for infiltration of runoff
Where longitudinal slope > 5%	Level spreaders shall be provided to ensure effective slope < 5%
Maximum overland flow distance uphill of the filter strip	23 m for impervious surfaces
Where concentrated flows enter the swale (from pipes)	Flows entering a filter strip cannot be concentrated. If this is the situation, level spreaders must be used to disperse flows
10-year storm velocities	< 1.5 m/s unless erosion protection is provided

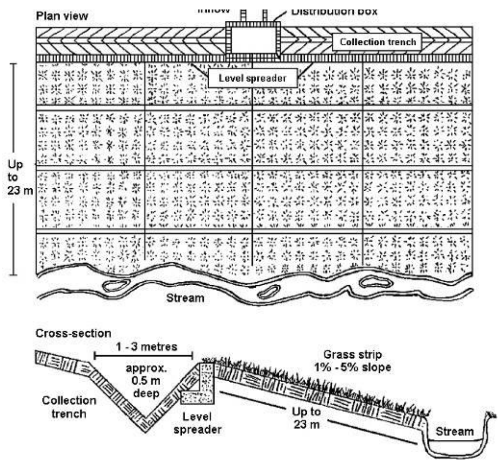
The Washington State Department of Transportation (WSDOT, 1995) recommends that filter strips treat highway runoff with a maximum of two lanes, and for an average daily traffic of less than 30,000 vehicles per day.

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

7.4.2.2 Detailed Design Procedure

A schematic of a filter strip is shown in Figure 7-4. The schematic shows a collection trench and a level spreader if the flow is from a pipe. In this situation the dispersed flow is maintained across the width of the filter strip.

Figure 7-4 Schematic of a Filter Strip



Design approach

1. The first step is to calculate the discharge (Q) for the area draining to the filter strip. If the filter strip is to take runoff only from impervious surfaces, the easiest way to determine the discharge is to use the rational equation Q = ciA/360 where c = 0.9, i = WQ storm depth and A = paved area.
2. Once the peak discharge is determined, that discharge can be entered into Manning's equation to determine the width of the filter strip.

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

   Where:

   A = width of filter strip x depth of flow (determined by design grass height)
   W = width of filter strip in metres
   R = depth of flow (due to very wide flow)(in metres)
   d = depth of flow in metres = R
   s = slope
   n = roughness coefficient (0.35)

   w is known from individual site conditions

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

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

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

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

   L = Vt

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

7.4.2.3 Case Study

<u>Project Description</u>

A residential home site is being constructed with a filter strip to treat the access road in front of it. The project is located in Wairoa so the water quality storm depth is 30 mm. The slope of land adjacent to the access road is 3% and the road is 500 metres with a crown in the centre so the portion of road draining to the filter strip is 3.6 metres wide.

Hydrology

Using the Rational Formula

Q_{wq} = CIA/360

C = 0.9 (estimate for paved surface)
I = Rainfall intensity (mm/hr.) - for Wairoa the water quality storm is 30 mm.
A = catchment area in hectares

Q_{wq} = (0.9)(30)(0.18)/360 = 0.013 m^3/s

For the 10-year storm, the effect of global warming is predicted to be 15.5% increase in rainfall. So, design rainfall for the 10-year storm = 34.6 mm.

Q_{10} = (.9)(34.6)(0.18)/360 = 0.0156 m^3/s

Filter strip design

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

   Where
   Q = water quality discharge (m^3/s)
   A = area of filter strip = (w -width in m)(depth of flow - d - in metres)
   R = 0.03 m based on water quality storm and very wide flow path
   s = .03
   n = .35

2. The width is given based on site conditions (75 m) so solve for y and ensure that it is less than 0.05 m.

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

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

   $d = (.013(.35)/75(.03)^{.5})^{.6}$

   d = 8.4 mm which is well under the maximum of 50 mm.

3. Calculate the flow velocity

   V = Q/wd = .013/75(.0084) = .02 m/s which is well under the maximum 0.4 m/s allowed.

4. Calculate the length of the filter strip.

   L = Vt = .02(540) = 10.8 metres in length.

As can be seen from this example, the filter strip width can be reduced substantially to adjust to site conditions. The two key elements are a maximum depth of flow during the water quality storm of 30 mm and a residence time of at least 9 minutes (540 seconds) to establish the length of the filter strip.

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

$Q_{10} = 0.015\ m^3/s$

Again using Manning's equation:

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

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

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

$d = (.015(.15)/75(.03)^{.5})^{.6}$

$d = 0.005\ m$

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

$V = Q/wd = 0.063/75(.005)$

$V = 0.04$ m/s which is well under an erosive velocity.

7.4.3 Sand Filters

Description: Sand filters are designed and constructed to capture and treat stormwater runoff through:

- Sedimentation
- Filtration
- Volatilisation
- Adsorption, and
- Biological processes

Stormwater Management Function

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

Sand filters use filtration for treating stormwater runoff. They are similar to biofiltration in that stormwater passes through a filtering media such as sand, gravel, compost or peat to filter out contaminants. They are especially suited for small catchment areas and are primarily water quality treatment practices having little water quantity benefit.

Sand filters have been used to treat stormwater runoff for years, mainly as a result of sand filter effectiveness at removal of hydrocarbons. They are very suitable in ultra-urban environments where space is limited but are also used where more space is available in a similar fashion to ponds.

As they are so effective at removal of finer sediments, they are prone to clogging and require maintenance on a more frequent basis than a practice such as wetlands. They are primarily used for high percentages of impervious surfaces where the majority of sediments are in the coarse fraction.

7.4.3.1 Basic design parameters

Sand filters should have a forebay (or sedimentation chamber) where coarser sediments would be captured and a filtration chamber, having an underdrain, for removal of finer sediments and hydrocarbons. A major component of a sand filter is live storage above the sediment/filtration chambers for storage of stormwater until the water can soak through the sand.

The following schematics provide a visual indication of how sand filters can be designed. They can be constructed similarly to ponds as shown in Figure 7-5, or an underground vault as shown in Figure 7-6 or as a linear filter as shown in Figure 7-7.

Figure 7-5
Sand Filter Basin

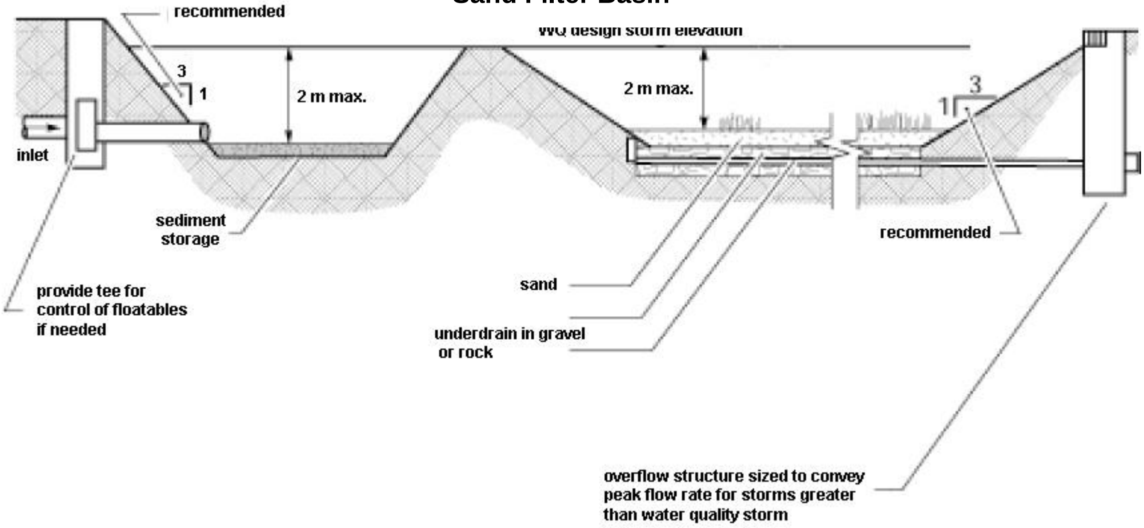


Figure 7-6
Vault Sand Filter

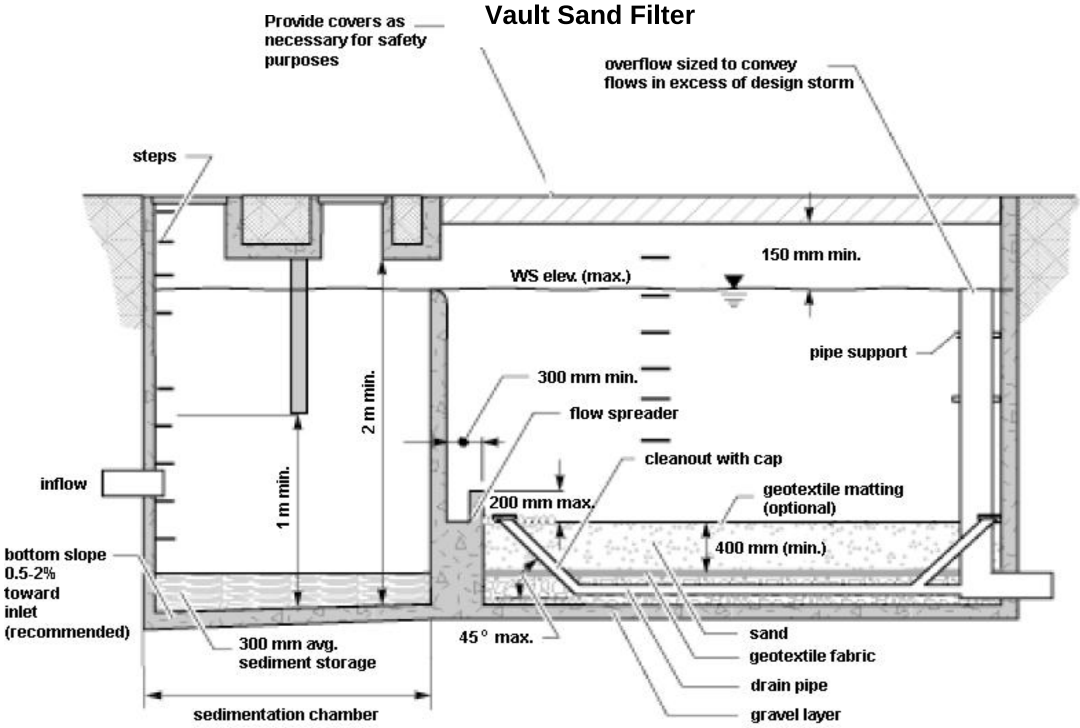
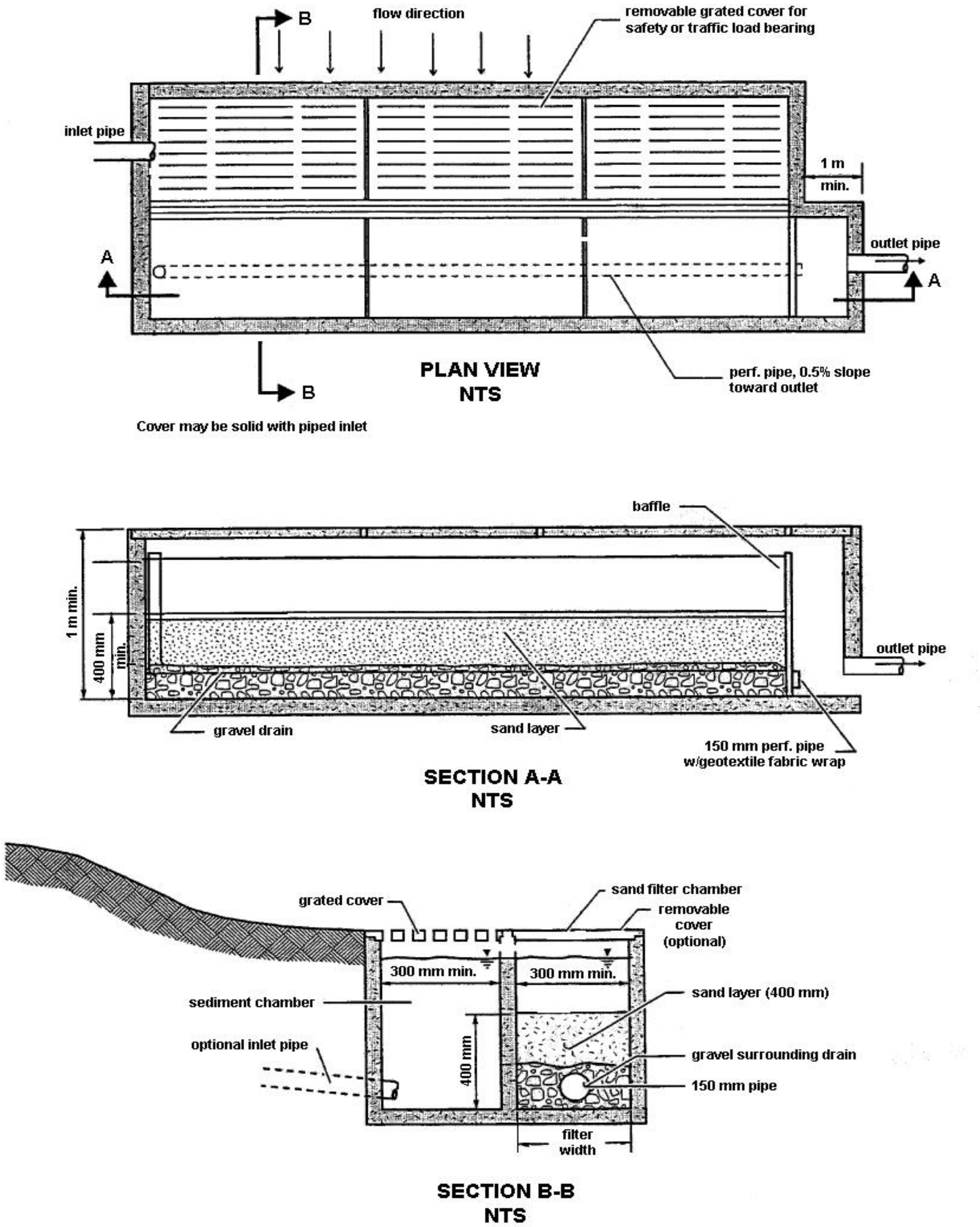


Figure 7-7
Linear Sand Filter



The treatment process is the same for all three of the practices, but Figure 7-5 allows for peak flow control in addition to water quality treatment. The other two figures provide water quality control only.

An important consideration of sand filter performance is the diversion of larger flows around the filter. Having high flows enter the filter with an overflow in the filter will significantly reduce performance, as turbulent flow will allow for finer sediments to pass over the filter bed. In a similar fashion, hydrocarbons having a specific gravity less than water will pass over the filter into the overflow pipe.

A simple way to prevent contaminants from exiting the filter is to have a flow diversion structure placed prior to the sand filter. This is a simple design, especially when the flow into the filter is through a pipe. Figure 7-8 provides a schematic of how the flow diverter can be designed so that the water quality storm passes through the sand filter and larger flows bypass it. Figure 7-9 shows how the system is placed schematically.

Most street and road particulate matter is in coarser fractions. However most stormwater contaminants are associated with fine particles. As sand filters have two chambers, the sedimentation chamber will remove the coarse sands and gravels while the filtration chamber will remove the finer silts and clays.

Figure 7-8 Diversion Weir

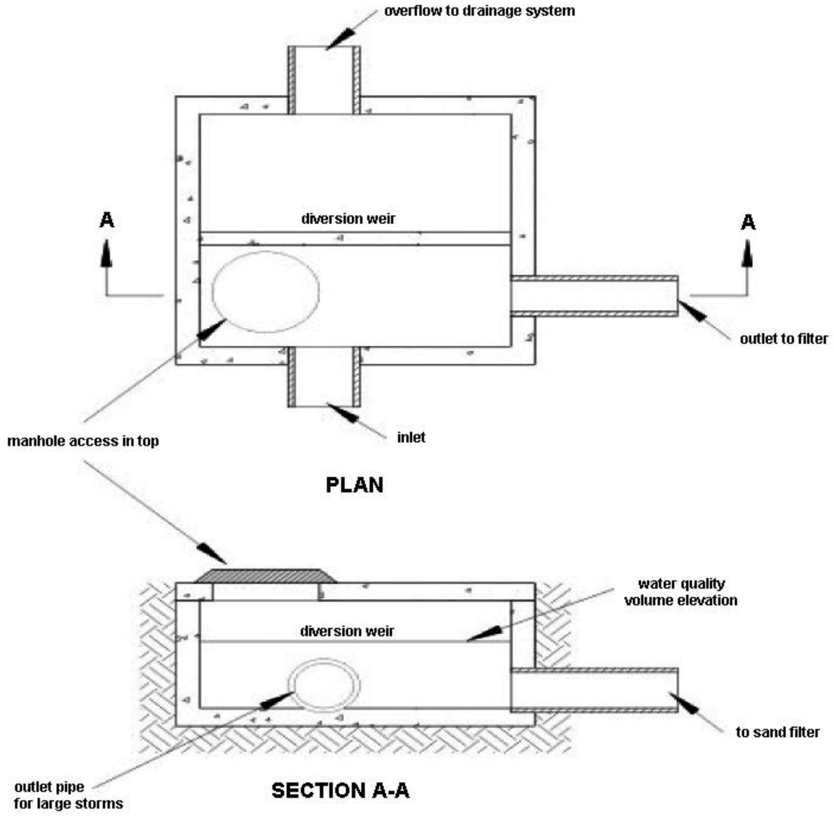
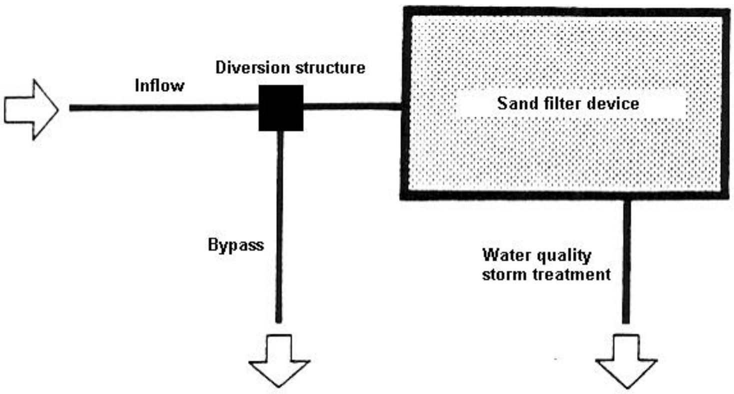


Figure 7-9
Large Flow Bypass Schematic



7.4.3.2 Detailed design procedure

Design approach:

1. Calculate the water quality volume to be treated by using the 90% storm. The City of Christchurch has a simple method of determining the first flush volume (Christchurch City Council, 2003) where the water quality volume (their first flush volume) is based on the following:

The catchment effective first flush runoff area = A_{eff} = imp.%A (ha)

The first flush volume $V_{ff} = 10A_{eff}d_{ff}$ (m^3)
Where d_{ff} = first flush water quality depth (water quality storm)

Another approach is to use a variant of the ARC's TP 108 manual ARC, 1999), which is based on the unit hydrograph method.

2. A minimum of 37% of the water quality volume must be available as live storage to ensure that the water quality volume passes through the filter without bypassing.
3. The sand filtration chamber is sized by a variation of Darcy's Law.

$A_f = V_{wq}d_f/k(h+d_f)t_f$

Where:
A_f = surface area of sand bed (m^2)
V_{wq} = water quality volume
d_f = sand bed depth (m)
k = coefficient of permeability for sand (metres/day)
h = average depth of water (WQ storm) above surface of sand (m) (1/2 max. depth
t_f = time required for runoff to pass through the filter (days)

The following values should be used.

t_f = 2 days (maximum)
k = 1 metre/day
d_f = 0.3metres (minimum)

Several points should be discussed regarding the values that should be used:

- Time required to pass the water quality storm
- The permeability rate selected

Time required passing the water quality storm

There are several reasons why this value was selected.

1. Having two days as a limiting value will ensure that the volume is available for the next storm. It should be recognised that these are averages and some fluctuation will occur.
2. Having the system drain within a two-day period will prevent the development of biofilms on the surface of the sand, which would reduce permeability rates.

Permeability rate

This is an issue that has controversy associated with it. Sand has a high permeability rate (Table 5-1) and the value selected is very low. Experience has shown that the initial high permeability rate rapidly reduces when contaminated stormwater runoff passes through the sand. The rate reduces to a level where it stabilises for a period of time before complete clogging occurs. The value generally accepted internationally is approximately one metre/day.

1. Size the sedimentation chamber with the following points in mind.
   a) Inflow into the chamber must not cause resuspension of previously deposited sediments
   b) The sedimentation chamber outlet must deliver flow to the filtration chamber as sheet flow
   c) The sedimentation chamber must be at least 25% of the filtration area
   d) Flow velocities in the sedimentation chamber are required to be below 0.25 m/s
   e) The sedimentation chamber must have a permanent pool with a minimum depth of 400 mm to reduce potential for sediment resuspension
   f) The sedimentation chamber should be configured to avoid short-circuiting of flow.

2. The sand specifications are the following as provided in Table 7-4:

Table 7-4

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

There will be some variation in sand grades from the specified grades. However, a number are close to the lower limit and can be used. It is important to meet as closely as possible the specified limits as coarser aggregate will allow for more contaminate migration and finer aggregate will clog more quickly.

A variation to using 100% sand is being used on a number of sites where dissolved metals are the contaminants of greatest concern. In those situations peat and/or activated carbon are being blended with the sand to provide for cation exchange and uptake by organic matter. The permeability rates are unchanged from those recommended in step 3 above.

3. An under drainage system shall be provided. The system will normally consist of perforated lateral pipes (150 mm diameter) that are placed in the gravel or stone layer that is under the sand. The depth of the gravel layer shall be at least 200 mm in depth with filter fabric between the gravel and sand to prevent migration out of the system.

7.4.3.3 Case study

<u>Project description</u>

It is the intention to construct a parking lot in Rissington that drains to the Mangaone River. The parking lot is approximately 3,000 square metres.

<u>Hydrology</u>

1. The water quality storm from the 90% rainfall map is 30 mm.

2. Calculate the water quality volume to be treated.

   The catchment effective first flush runoff area = A_{eff} = imp.%A (ha)

   A_{eff} = .9(0.3) = 0.27

   The first flush volume $V_{wq} = 10A_{eff}d_{ff}$ (m^3)
   Where d_{ff} = first flush water quality depth (water quality storm) = 30 mm from Figure 6-5..

   V_{wq} = 10(0.27)(30) = 81 m^3

Sand filter design

3. Live volume of storage needed V_{live} = .37(81 m^3) = 29.97 m^3

4. Sand filter surface Area - Assume that max. head, h_p = 1 metre so h = 0.5 m

   $A_f = V_{wq}d_f/k(h+d_f)t_f$

   We know the following:

V_{wq}	= 81 m^3
d_f	= 0.3 m
k	= 1 m/day
h	= 0.5 m
t_f	= 2 days

   A_f = (81)(0.3)/1(0.5+0.3)2

   A_f =15.19 m^2

5. Size sedimentation chamber has to have at least 25% of the surface area of the filter area = 3.79 m^2. In reality it has to be larger due to the need to provide 37% live storage. If the designs live depth of storage is one metre then the volume above the filtration area is 15.19 m^3.

   Since the required live storage is 29.97 m^3 then the combination of live storage in both the sediment chamber and filtration chamber must be increased by 11 m^3. The designer can determine where that additional volume must be obtained.

7.4.4 Rain gardens

Description: Rain gardens are designed and constructed to capture and treat stormwater runoff through:

- Sedimentation
- Filtration
- Infiltration (depending on soils)
- Adsorption, and
- Biological processes

Stormwater Management Function

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

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

- Bioretention
- Biodetention

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

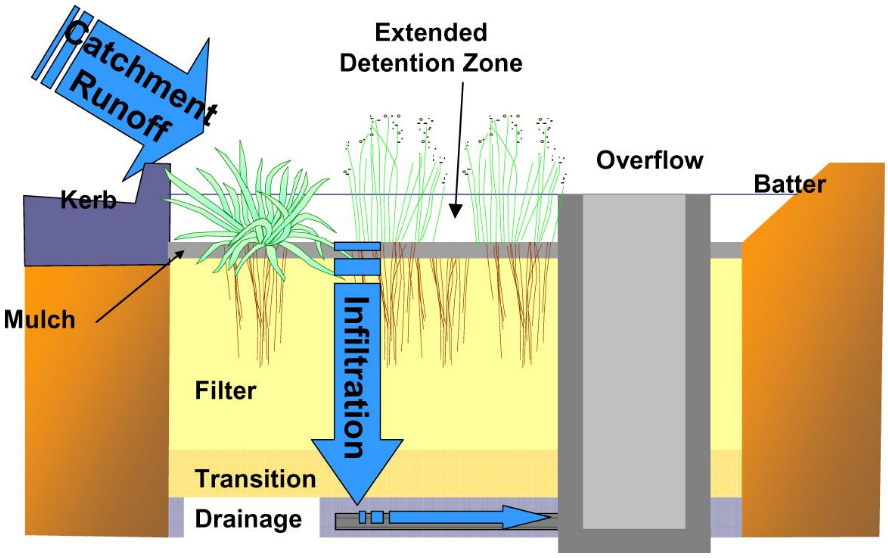
Biodetention is the passage of water through a filter media and then discharging that water downstream to surface waters.

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

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

To retain the filter media within the rain garden and aid drainage, one or more layers are used at the bottom of the filter. The surface of most rain gardens is planted with a range of vegetation. Figure 7-10 shows a schematic of a rain garden highlighting key elements and flow paths.

Figure 7-10
Schematic of Rain Garden Key Elements and Flow Paths
(Diagram courtesy of City of Kingston, Melbourne, Australia)



Rain garden design, as shown in Figure 7-10 differs only slightly from sand filter design. Where sand filtration relies on water quality treatment via passage of stormwater through sand, rain gardens incorporate plants and soils for removal of contaminants. A downside of the use of plants and soils rather than sand is a reduction in the permeability rate. This results in rain gardens being larger in surface area than sand filters.

Rain gardens have more aesthetic benefits not provided by sand filter systems and provide greater water quality benefits for a wider range of contaminants as a result of additional biological processes provided by plants and organic soils.

7.4.4.1 Basic design parameters

The main components of a rain garden include:

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

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

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

Rain gardens are designed as water quality practices and will generally not be used for water quantity control. If peak control is required and cannot otherwise be provided then consideration should be given to a constructed wetland that also provides peak control.

7.4.4.2 Detailed design procedure

Design approach:

1. Determine the water quality storage volume using the 90% rainfall level.
2. Minimum live storage provided above the soil media is 40% of the water quality volume to ensure that the entire water quality storm passes through the rain garden. Failure to provide the storage will result in system bypass and reduced water quality expectations.
3. Calculate the required surface area of the rain garden.

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

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

   The following values should be used.

   d_{rg} = 0.85 metre
   k = 0.75 m/d
   h = 0.15 m (maximum water depth 300 mm)
   t_{rg} = 1.5 days

4. General comments on rain gardens

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

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

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

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

5. Composition of planting soil

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

The bioretention filter media guidelines require three layers of media. The filter media itself (400 - 600 mm deep), a transition layer (100 mm deep) and a drainage layer (50 mm minimum under drainage pipe cover.

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

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

6. Determination of hydraulic conductivity

The hydraulic conductivity of potential filter media should be measured using the ASTM F1815-06 method. This test method uses a compaction method that best represents field conditions and so provides a more realistic assessment of hydraulic conductivity than other test methods.

7. Particle size distribution

Particle size distribution (PSD) is of secondary importance compared to hydraulic conductivity. A material whose PSD falls within the recommended range does not preclude the need for hydraulic conductivity testing. However, the following Table 7-5 provides a composition range for appropriate material specification.

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

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

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

8. Soil properties

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

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

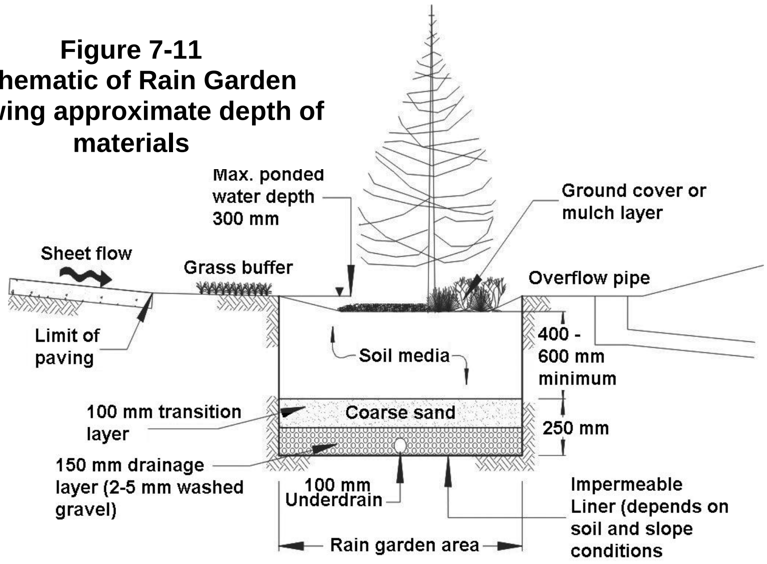
9. Transition and drainage layers

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

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

   The following Figure 7-11 provides a profile of a rain garden giving indicated distances for the various media and underdrain.

Figure 7-11
Schematic of Rain Garden showing approximate depth of materials



10. Plant material

Consider the following when making planting recommendations:

- Native plant species should be specified over exotic or foreign species
- Appropriate vegetation should be selected on its ability to thrive in wet and dry conditions.

The following two tables (Tables 7-6 and 7-7) provide some recommendations for rain garden plant species.

Table 7-6 Recommendations for Trees and Shrubs	
Trees and shrubs	Descriptions
Brachyglottis repanda **rangiora**	Coastal shrub or small tree growing to 4m+. Large attractive pale green leaves with white fuzz on underside.
Coprosma acerosa **sand coprosma**	Grows naturally in sand dunes. Yellow, interlaced stems and fine golden foliage. Forms a tangled shrubby ground cover. Tolerates drought and full exposure. Prefers full sun.
Coprosma robusta / C. lucida **karamu, shining karamu**	Shrubs or small trees growing to 3m+, with glossy green leaves. Masses of orange-red fruit in autumn are attractive to birds. Hardy plants.
Cordyline australis **ti kouka, cabbage tree**	Palm-like in appearance with large heads of linear leaves and panicles of scented flowers. Sun to semi-shade. Prefers damp to moist soil. Grows eventually to 12m+ height.
Cordyline banksii **ti ngahere, forest cabbage tree**	Branching from the base and forming a clump. Long strap-shaped leaves with red-orange coloured veins. Prefers good drainage and semi-shade.
Corokia buddleioides **korokio**	Bushy shrub to 3m, with pale green leaves with silvery underside. Many small bright yellow starry flowers are produced in spring. Prefers an open situation but will tolerate very light shade.
Entelea arborescens **whau**	Fast growing shrub or small tree (to 5m height) with large bright green heart-shaped leaves. Spiny seed capsules follow clusters of white flowers in spring. Handsome foliage plant
Geniostoma rupestre **hangehange**	Common forest shrub with pale green glossy foliage, growing to 2-3m. Tiny flowers give off strong scent in spring. Looks best in sunny position where it retains a bushy habit, and prefers well drained soil.
Hebe stricta **koromiko**	Shrub or small tree growing to 2-5m in height. Natural forms have white to bluish flowers. Many cultivars and hybrids available with other colours, but unsuitable for use near existing natural areas. Full sun.
Leptospermum scoparium **manuka**	Shrub or small tree growing to 4m+ in height. Natural forms have white to pinkish flowers. Many cultivars and hybrids available with other colours, but unsuitable for use near existing natural areas. Hardy and tolerant of difficult conditions.
Metrosideros robusta **rata**	Eventually forms a large tree. Flowers bright red in summer. Will tolerate dryness and exposure. Full sun.
Pittosporum cornifolium **tawhirikaro**	A slender branched shrub grown for its attractive fruiting capsules which are brilliant orange when split open. Sun or semi-shade.
Pittosporum kirkii	A small tree with dark green leaves and large yellow flowers in the summer. Prefers shade
Pseudopanax crassifolius **horoeka**	Very narrow rigid and leathery leaves in its juvenile form. Stunning in amongst bold leaved plants. Sun or semi-shade.
Pseudopanax lessonii **houpara**	Small tree with attractive foliage. Tolerates full exposure and drought. Sun or semi-shade

Table 7-7 Grasses, Ground Covers and Other Plants	
Grasses, ground covers, and other plants	Description
Arthropodium cirratum **Rengarenga, renga lily**	A lily with fleshy pale green – greyish leaves and white flowers. Ground cover in semi shady situation
Asplenium bulbiferum **mouku, hen and chicken fern**	A robust fern with small plantlets produced on the fronds. Tolerates dryness and prefers shade
Asplenium oblongifolium **huruhuruwhenua, shining spleenwort**	Fern with large shiny fronds. Tolerates dryness. Prefers shade
Astelia banksii **kowharawhara, coastal astelia**	Clump forming plant up to a metre high with flax-like leaves. Requires semi-shade. Tolerates full exposure. Frost tender
Astelia solandri **kowharawhara, perching astelia**	An epiphytic plant in natural situations. Long drooping bright green leaves. Tolerates dryness. Prefers shade
Carex flagellifera **manaia, Glen Murray tussock**	Sedge up to 70cm high with reddish-brown spreading foliage. Prefers damp soil and full sun. Tolerates exposure
Carex testacea **sedge**	Coastal sedge up to 40cm high with shiny orange foliage. Prefers full sun and exposure. Tolerates dry soil conditions
Cortaderia fulvida **toetoe**	Branching from the base and forming a clump to 4m high. Long strap-shaped leaves with red-orange coloured veins. Prefers good drainage and semi-shade
Dianella nigra **turutu**	Lily with reddish leaves, and striking violet-blue fruit. Ground cover; prefers open well-drained situation
Disphyma australe **glasswort**	Fleshy leaved ground cover with mauve flowers in the spring. Tolerates drought and full exposure. Frost tender
Doodia media **pukupuku, rasp fern**	Hardy fern growing to 25cm. Young fronds coloured bright red when in full sun. Sensitive to frost
Libertia grandiflora & ***L. ixioides*** **mikoikoi, native iris**	Clump forming native irises with narrow, upright leaves. Small white flowers in spring. Sun or shade
Phormium cookianum **wharariki, mountain flax**	Clump-forming flax with yellow –green drooping leaves, to 2m. Full exposure and sun
Phormium tenax **harakeke, flax**	Clump-forming flax with large stiff leaves, to 3 m. Full exposure and sun

Regarding planting, the following recommendations are made;

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

7.4.4.3 Case study

Project description

An industrial parking lot is proposed in Flaxmere with a rain garden proposed due to aesthetic reasons and for dissolved metals. The total extent of the catchment being served is 2,000 square metres of which 80% is impervious with the remainder being grassed.

Hydrology

1. The water quality storm from the 90% rainfall map is 18 mm.

2. Calculate the water quality volume to be treated.

   The catchment effective first flush runoff area = A_{eff} = imp.%A (ha)

   A_{eff} = .8(0.2) = 0.16

   The first flush volume $V_{wq} = 10A_{eff}d_{ff}$ (m^3)
   Where d_{ff} = first flush water quality depth (water quality storm) = 18 mm from Figure 6-5..

   V_{wq} = 10(0.16)(18) = 28.8 m^3

Rain garden design

1. Live volume of storage needed V_{live} = .40(28.8 m^3) = 11.52 m^3

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

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

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

   The following values should be used.

   d_{rg} = 0.85 metre
   k = 0.75 m/d
   h = 0.15 m (maximum water depth 300 mm)
   t_{rg} = 1.5 days

   A_{rg} = 28.8(.85)/0.75(0.15+0.85)(1.5)

   A_{rg} = 21.76 m^2

   Check to see that there is adequate live storage (11.52 m^3). Live storage available = surface area times maximum depth or (21.76)(.3) = 6.5 m^3 so the rain garden surface area has to be increased by 16.6 m^2 to provide the necessary live storage which gives a surface area requirement of 38.4 m^2.

7.4.5 Infiltration

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

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

Stormwater Management Function

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

Infiltration practices 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 practices (together with revegetation and rain tanks) that reduce the overall volume of stormwater being discharged.

Infiltration practices comprise a suite of different practices, including:

- Trenches
- Dry wells
- Modular block porous pavement
- To a certain extent, rain gardens, swales and filter strips that are considered separately.

Schematics for these practices are shown in Figures 7-12, 7-13 and 7-14.

Infiltration practices are used for three primary purposes:

- Reducing the total volume of stormwater runoff,
- Reducing the contaminant loadings downstream, and
- Low streamflow augmentation.

The use of infiltration practices for water quality treatment must be considered with caution. Infiltration practices are much more sensitive to clogging than are ponds or filters. As much as possible, sediment should be prevented from entering these practices.

Infiltration trenches receive runoff in a shallow excavated trench that has been backfilled with stone to form a below-grade reservoir. Water then enters the underlying subsoil according to its infiltration rate.

Figure 7-12
Schematic of an Infiltration Trench

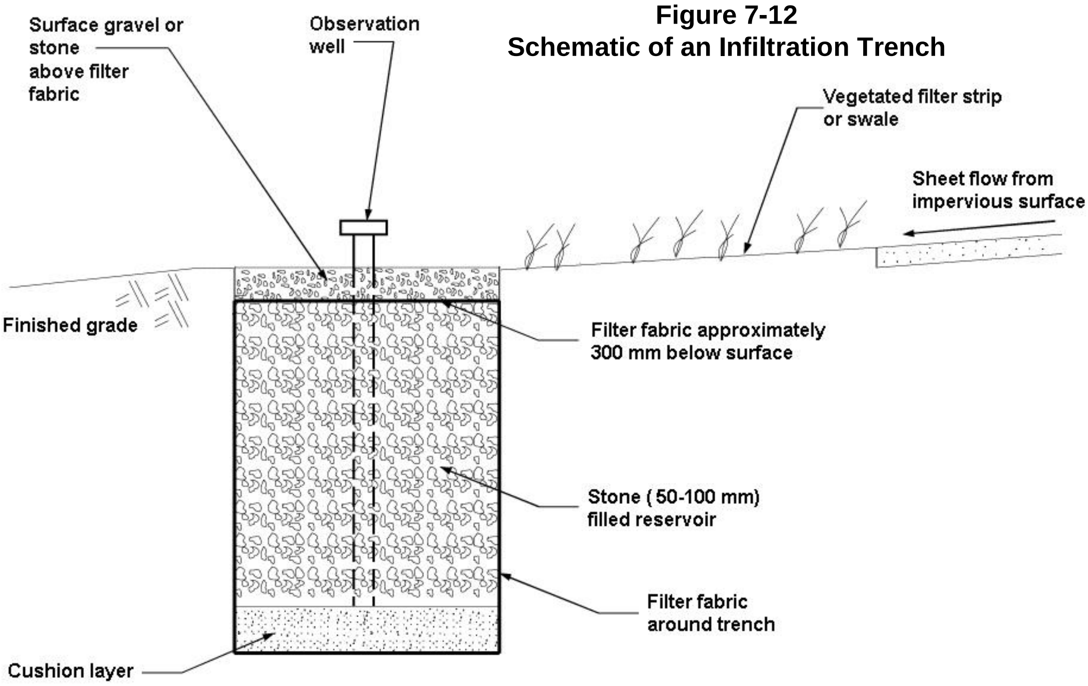
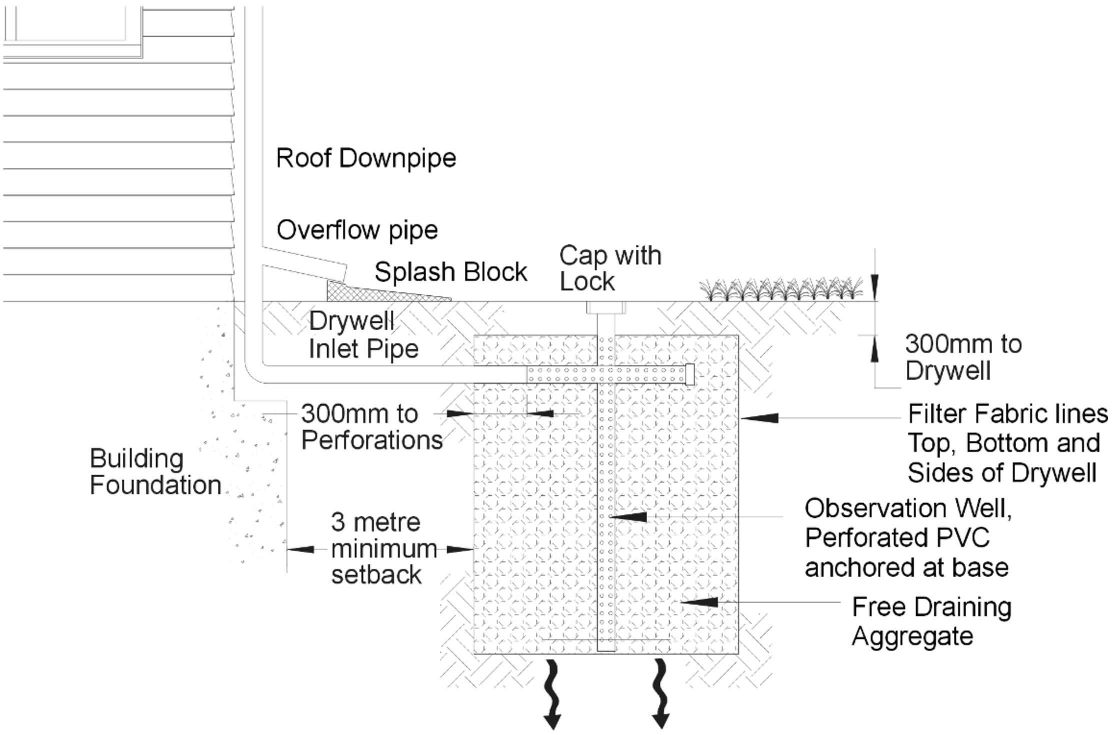


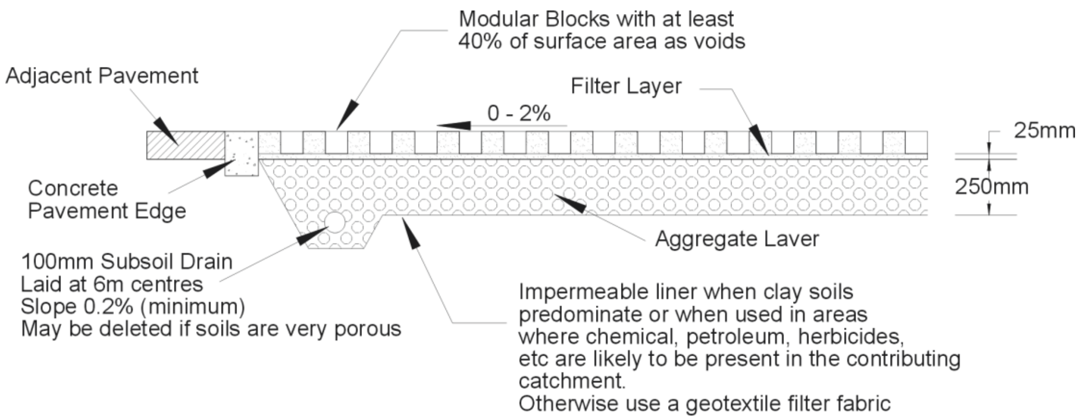
Figure 7-13
Schematic of a Dry Well



Dry wells function in a similar fashion with the excavated subgrade being filled with stone and relying upon the void spaces to provide for stormwater storage until the runoff infiltrates into the soil.

Modular block porous pavement permits precipitation to drain through paving blocks with a pervious opening. Paving blocks are appropriate only for areas with very light or no traffic or for parking pads. They are laid on a gravel subgrade and filled with sand or sandy loam turf but can also be used with grass in the voids which may require irrigation and lawn care during the summer months.

Figure 7-14
Schematic of a Permeable Pavement



7.4.5.1 Water quality performance

Infiltration systems do not have underdrains, so the design and soil characteristics determine how much runoff is captured and how efficient the treatment.

Among the various runoff treatment options, only soil infiltration systems have been reliable in removing soluble phosphorus. This result likely applies to other relatively soluble contaminants as well. Dissolved contaminant reduction is incomplete but is still higher than with any other treatment method. Table 7-8 estimates runoff contaminant removals.

Table 7-8
Long Term Contaminant Removal Rates

Contaminant	Size based on	
	Runoff from 25 mm rainfall	2-year runoff volume
Total suspended solids	90	99
Total Phosphorus	60 - 70	65 - 75
Total Nitrogen	55 - 60	60 - 70
Metals	85 - 90	95 - 99
BOD	80	90
Bacteria	90	98

7.4.5.2 Applicability

Soil permeability is the most critical consideration for the suitability of infiltration practices. Practices are generally built in native soil; but when this is inappropriate, a soil system can be constructed with media such as sand, peat, or a combination. Table 7-9 provides information on the suitability of various soils for infiltration. The red line indicates that 7 mm is the lowest infiltration rate that is considered acceptable for use of infiltration practices. Infiltration practices normally convey most runoff directly into the soil to eventually enter the groundwater. Constructed soil systems usually require underdrains.

Table 7-9 Infiltration Rate for Various Soil Textural Classes	
Texture Class	Approximate Infiltration Rate in mm/hour
Sand	210
Loamy sand	61
Sandy loam	26
Silt loam	13
Sandy clay loam	7
Clay loam	4.5
Silty clay loam	2.5
Sandy clay	1.5
Silty clay	1.3
Clay	1.0
	0.5

The next most crucial considerations for the suitability of infiltration practices are:

- Avoiding clogging
- Avoiding potential to contaminate groundwater.

Infiltration practices should be constructed in medium textured soils. They are generally unsuitable for clay because of restricted percolation and for gravel and coarse sands because of the risk of groundwater contamination (unless effective pretreatment is provided).

Any impermeable soil layer close to the surface may need to be penetrated. If the layer is too thick, underdrains may be required. As a minimum measure to prevent clogging, infiltration trenches should require a pretreatment device to settle larger solids and reject runoff from eroding construction sites. Infiltration dry wells accept only roof runoff so pretreatment is not expected, Pretreatment is not possible for modular paving either.

The following guidance is applicable to design and implementation of all infiltration practices.

<u>Site characteristics</u>

Site characteristics relate to whether the infiltration practice is intended for quantity control alone or for both quality and quantity control. While quantity control is best achieved with a rapid percolation rate, this rate could be too fast to provide sufficient contact with the soil for contaminant capture, if the groundwater table is relatively close to the surface.

Consequently, the Hawke's Bay Regional Council:

- Specifies a maximum and a minimum percolation rate to protect groundwater and attain contaminant capture objectives. Infiltration rates greater than 1 m/hr may indicate a direct link to a very permeable aquifer while slower than 7 mm/hour is too slow

- Requires runoff pretreatment to meet water quality objectives before the pretreated runoff is infiltrated for quantity control or stream baseflow augmentation

The following criteria aims to reduce the substantial risks of failure and groundwater contamination, and to achieve the desired urban stormwater management benefits:

- The invert of the infiltration practice should be at least one metre from the seasonal high water table, bedrock, or relatively impermeable soil layer
- The percolation rate should be at least 7 mm/hr.
- The soil should not have more than 30 percent clay or more than 40 percent clay and silt combined.
- If the infiltration practice is to function for primary water quality treatment, infiltration rates must not be greater than that given for sand. Injection into basalts must be preceded by water quality treatment prior to injection.
- Infiltration practices must not be constructed in fill material.
- Infiltration practices must not be constructed on slopes exceeding 15 percent.
- Catchments draining to infiltration practices must not exceed four hectares, but preferably not more than two hectares.
- Infiltration basins are not encouraged for use unless approved on a case-by-case basis because their long term historical performance has not been good, mainly as a result of surface clogging

<u>Pretreatment</u>

The use of vegetative filters as a pretreatment BMP to improve long term performance of infiltration practices cannot be stressed enough.

Of primary importance to the long-term function of infiltration practices is the need to keep all contributing catchment areas stablised. Sediment loadings into the practice must be kept to a minimum. All inspections of these practices must include inspection for site stabilisation. All areas draining to the infiltration practice must be stabilised or premature clogging of the facility will result. Infiltration practices should have annual inspections done for assessing sediment accumulation. The frequency of actual maintenance activities depends on loadings from contributing catchment areas.

7.4.5.3 Objectives

Because infiltration practices are the only traditional stormwater management practice that reduces the total volume of runoff, objectives relate to:

- Peak flow reduction
- Contaminant removal
- Low stream flow augmentation

Due to the sensitivity of infiltration practices to clogging, they are best utilised to augment low stream baseflow, with pretreatment to reduce contaminant loads so that the cleaner water infiltrates to maintain groundwater levels and maintain low stream flow.

If long-term responsible maintenance can be assured, infiltration is appropriate as a water quality treatment practice

7.4.5.4 Design approach

There are a number of items that should be considered when infiltration practices are used.

Site characteristics

A site characterisation must be done to determine the following:

- Topography within 150 metres of the proposed infiltration practice
- Site use
- Location of any water supply wells within 150 metres of the proposed infiltration practice
- Local site geology to gain understanding of soil and rock units likely to be encountered, the groundwater regime and geologic history of the site.
- For infiltration trenches, at least one test pit or test hole per 15 metres of trench length and 2.5 times deeper than the invert depth of the trench.
- For dry wells, at least one test pit for each dry well. The test pit should be 1.5 times deeper than the invert depth of the dry well.
- For modular porous pavement, there must be one test pit per 500 m^2 of infiltrating surface and the test pit should be 2.5 times deeper than the invert depth of the filter bed.
- The depth, number of test holes or test pits and sampling should be increased, if, in the judgment of the geotechnical engineer, the conditions are highly variable and increasing the depth or the number of explorations is necessary to accurately estimate the performance of the infiltration practice. In addition, the number of explorations may be decreased if, in the opinion of the geotechnical engineer, the conditions are relatively uniform and the borings/test pits omitted will not influence the design.
- Detailed logs for each test pit or test hole must be prepared along with a map showing the location of the test pits or holes. Logs must include at a minimum, depth of pit or hole, soil description, depth to water, depth to bedrock or impermeable layer, and presence of stratification.
- Install ground water monitoring wells (unless the highest ground water level is far below the infiltration practice) to monitor the seasonal ground water levels at the site.

8.5.2 Procedure for conducting an infiltration test

The required approach consists of a relatively large-scale infiltration test to better approximate infiltration rates for design of infiltration practices. This approach reduces some of the scale errors associated with relatively small-scale double ring infiltrometer or “stove pipe” infiltration tests.

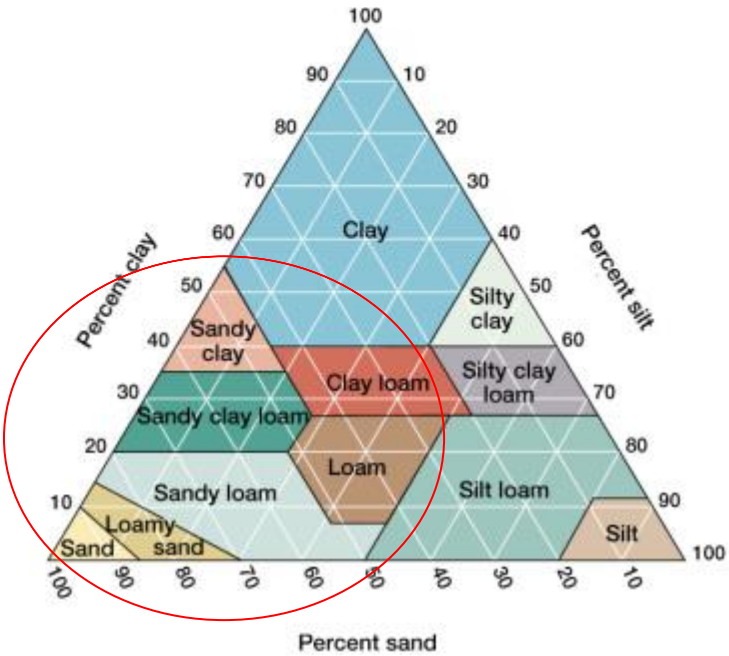
1. Excavate the test pit at least 1.5 metres below the bottom of the proposed infiltration practice. Lay back the slopes sufficiently to avoid caving and erosion during the test.
2. The surface area of the bottom of the test pit shall be at least 1 square metre.
3. Install a vertical minimum 1.5 metre long measuring rod marked in 10 mm increments in the centre of the pit bottom.
4. Use a rigid 150 mm pipe with a splash plate on the bottom to convey water to the bottom of the pit and reduce side-wall erosion or excessive disturbance of the ponded bottom.

5. Add water to the pit at a rate that will maintain a water level of between 1 - 1.25 metres above the bottom of the pit. A rotameter can be used to measure the flow rate into the pit.
6. Every 15-30 minutes, record the cumulative volume and instantaneous flow rate in litres per minute necessary to maintain the water level at the same point on the measuring rod.
7. Add water to the pit until one hour after the flow rate into the pit has stabilised (constant flow rate) while maintaining the same ponded level.
8. After one hour after the flow rate has stabilised, turn off the water and record the rate of infiltration in mm/hour from the measuring rod data, until the pit is empty.
9. Based on partial clogging, reduce the derived infiltration rate by a factor of 0.5 and reduce this reduced rate in the design calculations.

<u>Site data analysis</u>

- Determine representative site infiltration rate from soil test results and the stratification identified during the site investigation.
- Determine the textural class from the U.S. Department of Agriculture (USDA) textural triangle in Figure 7-15. Sand is defined to have a diameter between 2000 um and 50 um while clay has a diameter of less than 2 um. Once the texural class has been determined, the infiltration rates can be found.

Figure 7-15
Soil Textural Triangle (Davis, Bennett, 1927)



- Determine infiltration rates by taking direct in-situ measurements of soil infiltration rates.
- Long-term infiltration rates greater than one metre per hour (as per steps 8 and 9 above) are considered too rapid to allow significant water quality treatment to occur and pretreatment will have to be provided.

7.4.5.5 Detailed design procedure

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

In terms of the design approach:

1. Determine the water quality rainfall from the 90% storm.
2. Calculate the water quality volume.
3. Size the practice surface area to allow complete infiltration within 48 hours, including rainfall falling directly on it. Use the following equation to determine surface area:

   $A_s = WQV/((f_d)(i)(t)\text{-}p)$

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

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

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

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

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

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

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

   Where:

   V = practice volume with the aggregate added

   <u>NOTE:</u> Permeable paving does not usually have a contributing drainage area draining to it. As such the volume of storage equals the following:

   $V = pA_s/v_r$ where p is the design rainfall event (at least the water quality storm)

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

   V/A_s = depth of trench (d)

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

Infiltration practices 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 stormwater runoff to streams, infiltration practices divert runoff into groundwater.

7.4.5.6 Case study

Project description

A development road is being designed for in the vicinity of Tutira. The road will be crowned in the middle and 3000 m^2 will drain to each side of it so two trenches are sized.

Hydrology

1. Calculate the water quality volume

   The 90% storm in Tutira is 27.5 mm of rainfall.

2. Using that rainfall, the water quality volume is calculated.

   WQV = 82.5 m^3

Infiltration trench design

3. Calculate the practice surface area

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

   Where:
   A_s = surface area of the trench (m^2)
   WQV = water quality volume (m^3) = 40.5 m^3
   f_d = infiltration rate (m/hr) - rate reduced by ½ from measured =14 mm/hour reduced by ½ as a factor of safety, so f_d = 7 mm/hour = 0.007 m/hour
   i = hydraulic gradient (m/m) - assumed to be 1
   t = time to drain from full condition (hours) - maximum time 48 hours
   p = rainfall depth for water quality storm (m) = .015 m

   A_s = 82.5/((.007)(1)(48) - .0275) = 275 m^2

   Calculate the maximum trench depth

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

   Where:

   d_{max} = maximum depth of trench
   f_d = infiltration rate (m/hr) = 0.007 m/hour
   t = time to drain from full condition (hours) = 48 hours
   V_r = void ratio of reservoir stone = 0.35

   d_{max} = .007(48/.35) = 0.96 m

4. Find the trench volume

$V = 0.37(WQV + pA_s)/V_r = 0.37(82.5 + 0.0275(275)/.35 = 95.2\ m^3$

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

   V/A_s = depth of trench (d) = 95.2/275 = 0.346 m

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

7.4.6 Stormwater Management Ponds

Description: Stormwater management ponds can provide peak flow control and water quality treatment. Processes for contaminant reduction are primarily related to:

- Sedimentation

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

Stormwater management ponds have been used in local programmes for years, initially for water quantity control, but more recently also for water quality control. They have been, and are expected to remain, important components in the stormwater effort to minimise adverse impacts associated with urban land use. This Section reviews ponds that are either normally dry or normally wet. Both forms of pond can and may possibly have an extended detention component to them. This Section does not include discussion of wetland ponds. Wetland ponds, while having much in common with deeper ponds are being considered separately within Section 7.4.7, a more detailed discussion of the additional functions that they provide.

Ponds are defined as:

Dry pond - A permanent pond that temporarily stores stormwater runoff to control the peak rate of discharges and provide water quality treatment, primarily through the incorporation of extended detention. These ponds are normally dry between storm events.

Wet pond - A permanent pond that has a standing pool of water. These ponds can, through their normal storage of water, or in conjunction with extended detention, provide water quality treatment. They can, also in conjunction with extended detention, provide protection of downstream channels from frequent storms.

Stormwater ponds are used for three primary purposes:

- Reducing downstream flood potential,
- Providing water quality treatment, and
- Minimising, to the extent possible, downstream channel erosion.

It may not be necessary in every situation to address all three purposes, but there will be sites, as discussed later in this Section, where all three functions will be included in the design.

7.4.6.1 Water quantity/quality performance

Ponds detain runoff, typically from a design storm, and then discharge it, usually at the pre-development peak discharge rate.

Traditionally ponds, especially dry ones, have been used primarily for flood protection. They normally detain runoff and then discharge it at a specified rate, reducing the potential for downstream flooding by delaying the arrival of runoff from upper parts of a catchment. More recently, wet and dry pond designs have been modified to extend the detention time of runoff thereby increasing particulate contaminant settling and minimising downstream channel erosion. Wet ponds are normally designed to have a permanent pool for storage of a specified water quality volume, in the Hawke's Bay Region; this is 90% frequency storm. Wet ponds also have an outlet design that increases residence time and flow path.

<u>Contaminant removal mechanism</u>

The primary contaminant removal mechanism of all pond systems is settling or sedimentation. However, the effectiveness may vary to some degree depending on the type of detention system (dry or wet).

Flood detention ponds have limited effectiveness at providing sedimentation, as detention times may be several hours only, so only the coarser particles can be removed from the water column.

Extended detention ponds that are normally dry also rely on sedimentation during shore periods of live storage only although they typically hold flows for longer than flood detention ponds.

The best approach for particulate removal is the combination of extended detention in conjunction with a normal wet pool. The pool allows for displacement of water previously stored and the extended detention allows for better sedimentation of excess storm flows.

<u>Expected performance</u>

Ponds can be effective at reducing peak discharge rates. Depending on their design and their location within a catchment, they may also be effective in reducing downstream channel erosion, downstream flood levels and flooding.

Effectiveness at contaminant removal depends on the type of pond system. In general, they can be ranked, from least to most effective, in their ability to remove stormwater contaminants: dry detention, extended dry detention, and then wet detention.

Unlike dry detention ponds, wet ponds provide mechanisms that promote the removal of dissolved stormwater contaminants, and not just particulates. Table 7-10 illustrates expected contaminant reduction.

Table 7-10 Expected Contaminant Reduction Range of Ponds (%)			
Contaminant	Dry (flood)	Dry (ext. detention)	Wet
Total suspended solids	20 - 60	30 - 80	50 - 90
Total Phosphorus	10 - 30	15 - 40	30 - 80
Total Nitrogen	10 - 20	10 - 40	30 - 60
COD	20 - 40	20 - 50	30 - 70
Total lead	20 - 60	20 - 70	30 - 90
Total zinc	10 - 50	10 - 60	30 - 90
Total copper	10 - 40	10 - 50	20 - 80
Bacteria	20 - 40	20 - 60	20 - 80

<u>Constraints on the use of ponds</u>

- Dry ponds
- Need fairly porous soils or subsurface drainage to ensure that the bottom stays dry between storms
- 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 as detention ponds do not detain water long enough to reduce temperatures from impervious surfaces.

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 dug into the groundwater
- Minimum contributing drainage area of 2 - 3 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 need to be addressed, depending on normal pool depth

Dry flood detention ponds are not normally recommended for stormwater management systems. They have ongoing maintenance needs because standing water in areas where positive drainage is impeded may cause mosquito problems, and their overall performance for water quality treatment is less than provided by wet ponds. A study in the U.S. (DNR. 1986) indicated that over 70% of the dry ponds in a given jurisdiction were not functioning as designed. In addition, dry ponds tend to have less aesthetic appeal than wet ponds.

7.4.6.2 Pond component disclaimer

As part of the Guidelines for Waterways, the Hawke’s Bay Regional Council has Small Dam Design guidelines, and that will have a general discussion of dam components. The technical safety criteria for pond design and construction that are beyond the scope of this document include:

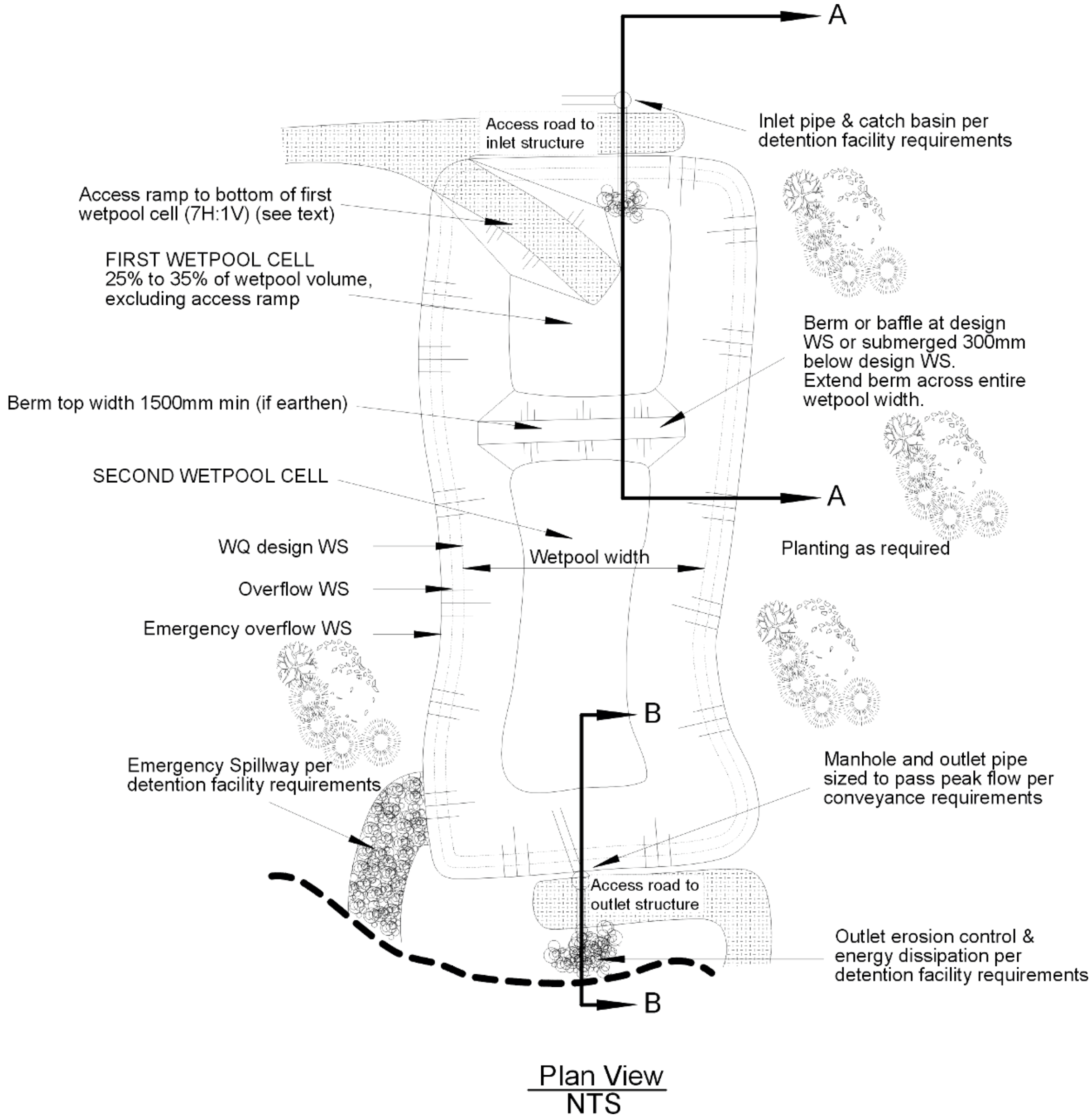
- Minimum dam top width
- Embankment side slopes

- Seepage control
- Foundation standards
- Foundation cut-off
- Outlet protection
- Access and set aside area for sediment drying

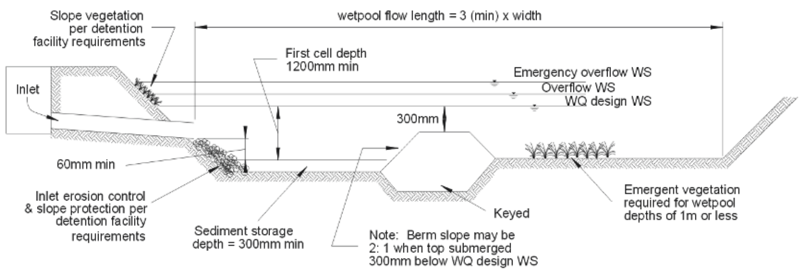
Two issues that will be discussed in this Chapter are minimum spillway capacity, as spillway design will affect the duration of detention and therefore stormwater quantity and quality control, and pond forebay areas and capacity. These will be discussed in the Design Procedure section.

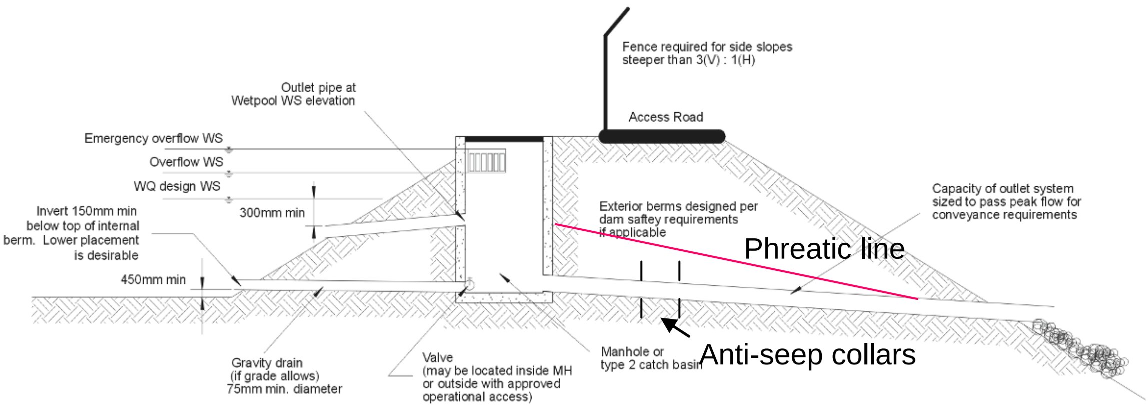
A typical wet pond is shown in Figure 7-16.

Figure 7-16
Schematic of a Stormwater Management Pond



Continuation of Figure 7-16 showing Cross-Sections





7.4.6.3 Design approach

<u>Objectives</u>

Water quantity objectives

Urbanisation has dramatic impacts on the amount of stormwater runoff that is generated from a catchment. Ponds, when properly sized, can be a primary quantity control practice.

Hawke's Bay Regional Council criteria for water quantity control depend on the receiving environment. If the receiving environment is a piped stormwater reticulation system with adequate capacity for the increased runoff or tidal (either estuarine or marine), then water quantity control is not an issue and a number of practices can be used to achieve water quality goals. If the receiving environment is a stream, then control of peak rates of runoff may be a requirement, and ponds become a primary option for controlling discharge rates.

Hawke's Bay Regional Council may require on a case-by-case basis that both the 2 and 10-year storms remain at their pre-development peak rates for those storms. The intent of peak discharge control of storms of two different frequencies is to achieve benefits for a range of discharges. Controlling the peak rates for the 2 and 10-year storms provides control of storms between those intervals and also will provide management for a percentage of peak flows from storms of greater magnitude (Maryland, 1982).

Where there are downstream flooding issues, peak discharges for the post development 100 year 1% AEP storm event may need to be managed to ensure that downstream flood levels are not increased. Depending on the catchment, the number of tributaries and the location of the project in a catchment, timing of flow discharges may be an issue. If so, a catchment wide study may be necessary to ensure that downstream flood risks are not increased. If there is no catchment-wide study, work done by Manukau City Council and overseas has indicated that limiting the peak discharge of the 100-year storm to not exceed 80% of the pre-development 100-year storm will reduce downstream flood increase concerns. The 80% peak discharge rate reduces potential for coincidence of elevated flow downstream by extended release of the flows. The Hawke's Bay Regional Council may accept this approach as an alternative to a catchment wide study.

Water quality objectives

Water quality objectives aim for 75% removal of TSS. Ponds are not as appropriate for dissolved contaminants. They are more appropriate where sedimentation can achieve stated goals.

Where possible, water quality ponds need a bypass for larger flows. Because all flows travel through the pond, water quality performance during larger events will be reduced as first flush contaminants are carried through it. Ideally, larger flows should bypass the pond in order to avoid a drop in water quality performance, albeit at the expense of its ability to provide peak flow reduction for larger storms.

In those situations, it may be best to use a treatment train approach to stormwater where other practices provide primary water quality treatment while the pond is primarily used for water quantity control. Although desirable, this approach may not always be possible due to site constraints.

There is a direct linkage between water quality treatment and flow control. If catchment considerations necessitate peak controls, it is recommended that 50% of the calculated water quality volume be placed as dead storage while 50% of the water quality volume can be live storage and released as part of the 1.2 times the water quality rainfall capture and release requirement (as discussed in the next section). This water quality credit can only be provided when storage and release of the runoff from 1.2 times the water quality rainfall is required. The permanent storage will reduce flow velocities entering the pond, while the extended detention will facilitate (in addition to the wet pool) settlement of particulates. If there is no requirement for either extended detention or peak control, the entire water quality volume can be stored within the permanent pool level.

Channel protection objectives

Urban development has the effect of increasing the frequency and magnitude of floods, particularly during frequent small storm events. As a consequence streams can suffer an increase in erosion, as channels enlarge to cope with the increased storm response. The objective of criteria related to channel protection is to maintain or improve the in-stream channel stability to protect ecological values of the stream and reduce sedimentation downstream.

A study (BECA, 2001) recommends that the pond outlet should be designed to convey the volume generated by the first 30 mm of runoff over the total catchment area and release that volume over a 24 hour period from a 2 year frequency storm event. However, because more extensive impervious surfaces upstream require more storage to achieve the discharge target, the Hawke's Bay Regional Council may require on a case-by-case basis that the runoff from 1.2 times the water quality volume to be stored and released over a 24 hour period to minimise potential for stream channel erosion.

This provision is in addition to normal stormwater quality and flow attenuation requirements. However, by using extended detention for some of the stormwater quality treatment rather than a full wet pond, the treatment and erosion attenuation volumes may be partially combined, reducing total pond volume.

Ponds in series

The Hawke's Bay Regional Council does not generally recommend the use of ponds in series instead of a single pond with an equivalent surface area. If the single pond were divided into two ponds in series then each of the two ponds would have approximately 1/2 of the surface area of the single one. Each pond then has half the detention time, so the first pond takes out the coarser sediment. The flow is then remixed in the channel between ponds, and the second pond is too small to take out the finer fractions. Therefore ponds in series may be less efficient than single large ponds of equivalent volume.

However, sometimes site constraints make it necessary to use two or more treatment ponds in series rather than one larger single pond. To offset the reduction in sediment removal, where two or more ponds in series are necessary they should be sized at 1.2 times the volume specified in this document for a single pond. Where there are no specific site constraints, a single pond is preferred.

<u>Preferences</u>

Preferences for wetlands versus ponds

While this Guideline is a 'toolbox' of available stormwater management practices, constructed wetlands are preferred to open water ponds because they provide better filtration of contaminants, including dissolved ones due to densities of wetland plants, incorporation of contaminants in soils, adsorption, plant uptake, and biological microbial decomposition (more in depth discussion in Section 7.4.7). In addition, wetlands, being shallow water bodies do not have the safety issues associated with deeper water ponds. For these reasons, the Hawke's Bay Regional Council has a preference for shallow wetland ponds where ponds are used.

On-line versus off-line

The Hawke's Bay Regional Council has preference for 'off-line' placement of ponds rather than 'on-line'. Off-line ponds are considered to be those ponds not physically located in perennial watercourses. They can be in gullies or upland areas. On-line ponds are located on streams having perennial flows and their impact to the stream itself can be significant. On-line ponds alter geomorphic and biological character of streams and these alterations may adversely impact on the streams natural character and function.

However, while off-line ponds are a preference, it is not a hard and fast rule. On-line ponds will be considered on a case-by-case basis to determine suitability.

There may be mitigation requirements placed on on-line ponds to compensate for the loss of stream habitat when an on-line pond is accepted for a specific location.

Dry ponds versus wet ponds

Dry ponds are not normally recommended. They need more maintenance and have a lower water quality performance than wet ponds. In terms of preference when ponds are the selected options, constructed wetlands are a first choice, followed by wet ponds, and finally dry ponds.

Maintenance responsibility

The issue of ensuring an entity is responsible for maintenance must be considered as an issue to determine whether ponds are applicable in a given situation. Ponds are expensive and require routine and non-routine maintenance to ensure proper long-term performance or failure of the pond system can occur. While a swale can fill in or a sand filter clog, pond failure can have significant effects, such as property damage and potential loss of life. Ponds must therefore be regarded as small dams, and evaluated in the context of best practice for dam operation. If maintenance responsibility cannot be defined during the design phase, ponds should not be selected for a given site.

<u>Safety features</u>

Depth

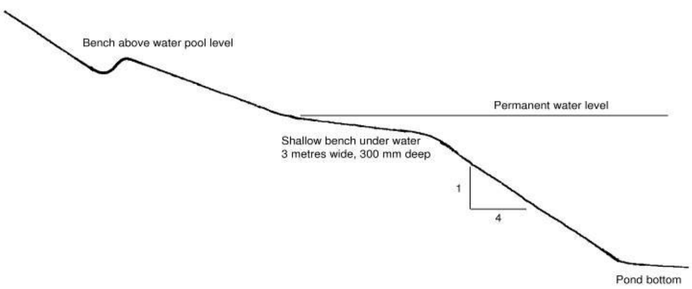
Deeper ponds can be attractive to children who like open water. Historically, ponds have been 1 - 3 metres deep, sometimes over anyone's head. Stormwater ponds should not be deeper than 2 metres, if at all possible. If water quality volume requirements and site limitations limit pond area, then use a wetland and extended detention live storage to achieve the water quality volume.

Benches

A reverse slope bench or slope break should be provided 300 mm <u>above</u> the normal standing water pool (where there is a normal pool) for safety purposes. All ponds should also have a shallow bench 300 mm deep that extends at least three metres from the shoreline, before sloping down to the pond floor. This shallow bench will facilitate the growth of emergent wetland plants and also act as a safety feature.

In addition to the benches, the steepness of the pond slope down to the invert of the pond should not exceed 4 horizontal to 1 vertical. Steeper slopes will make it very difficult for someone who is in the pond to get out of it. A schematic of pond safety

Figure 7-17
Schematic of Safety Benches and Slopes



features is shown in Figure 7-17.

The reverse slope above the waterline has at least three functions. It:

1. Reduces erosion by rilling that normally would be expected on longer slopes.

2. Intercept particulates travelling down the slope and conveys them to the pond inflow.

3. Provides an additional safety feature to reduce the potential for children running or riding uncontrolled down the slope and falling into the pond.

Fences

The Hawke's Bay Regional Council does not require fencing of ponds, because it is considered that use of natural features such as reverse benching, dense bank planting, and wetlands buffers (which consists of a dense stand of vegetation) will provide a similar level of protection. Territorial authorities retain their own discretion about fencing.

Aesthetics

Aesthetics must be considered as an essential pond design component. Ponds can be a site amenity if properly designed and landscaped or can be a scar on the landscape. The developer and designer should consider the pond as if they themselves were to be living in the development. Small items can have a big influence on the liveability of a given area to residents and the best time to consider the issue is during the design phase. There is a greater discussion of landscaping in Chapter 8.

7.4.6.4 Design procedure

Approach

Pond sizes are determined to remove 75% of the incoming sediment load on a long-term basis.

Pond design tasks, in order, include the following:

1. Determine the need for water quantity control. In normal situations if it is required, that requirement will be to limit post-development peak discharges for the 2 and 10-year frequency storms to their pre-development peak discharge release rates.

   If downstream flooding is documented, the post-development 100-year storm peak discharge rate may also need to be limited. In this case, a catchment analysis may be necessary or, as an option to the catchment analysis, limiting the 100-year peak discharge to 80% of the pre-development release rate.

2. Protect channel form in receiving environment. If the discharge enters a perennial natural stream channel, its channel will need to be protected from erosion. In such cases the runoff from 1.2 times the water quality rainfall shall be stored and released over a 24-hour period.

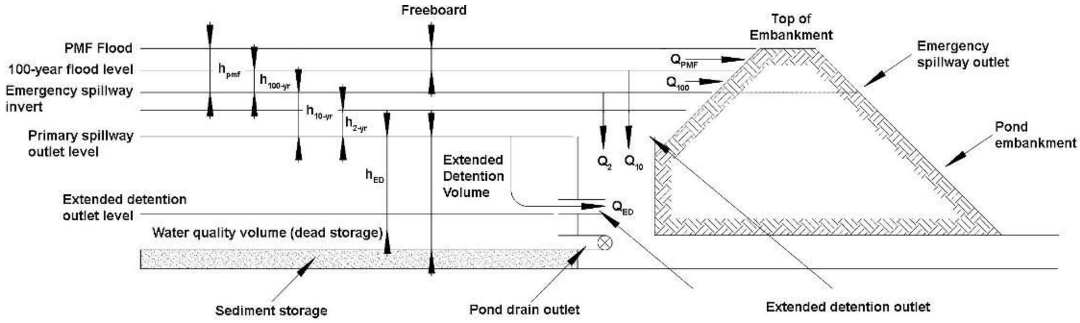
3. Determine the need for water quality control. Calculate the water quality volume (based on 90% rainfall) that needs to be treated when detention is required, and provide at least 50% of that volume as permanent pond storage. The other 50% stores and releases runoff from 1.2 times the water quality rainfall over a 24-hour period.

A hydrological analysis is needed for up to five rainfall events including the 2-year, 10-year, possibly 100-year, 1.2 times the water quality rainfall, and the water quality rainfall. The 2, 10, and 100-year events must be done for both pre- and post-development while 1.2 times the water quality rainfall (erosion protection) and 90% rainfall (water quality treatment) events are based on the post-development condition.

<u>Spillways and outlet capacity</u>

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 7-18 illustrates the various outlet elements and components. The terms detailed in the figure are those used in the Hydraulic Flow discussion of this chapter.

Figure 7-18
Schematic of Pond Outlet Components



Service outlet

The service outlet should be designed to at least accommodate the flows from the primary drainage system entering the pond. The service outlet will normally convey the flow from the extended detention orifice, the 2-year storm and the 10-year storm. In addition, the service outlet should also have a gate valve at the invert of the normal pool to allow for drainage of the pond during maintenance.

When an extended detention orifice is required, that orifice shall not be less than 50 mm in diameter (or 50 mm wide if a slot) unless a cover plate or screen device is used to prevent clogging of the orifice. If calculations indicate an orifice (or slot) of smaller size, attention must be given to implementation of protective measures such as cover plate or other means, to prevent blockage of the orifice. It is important to consider blockage on all outlet devices but the extended detention outlet will be susceptible to blockage unless specifically designed for.

Emergency spillway

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

The emergency spillway should be located 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 in natural ground in order to determine the need for additional armouring. If the emergency spillway is placed on fill, the embankment should be constructed higher than the final design to allow for settlement.

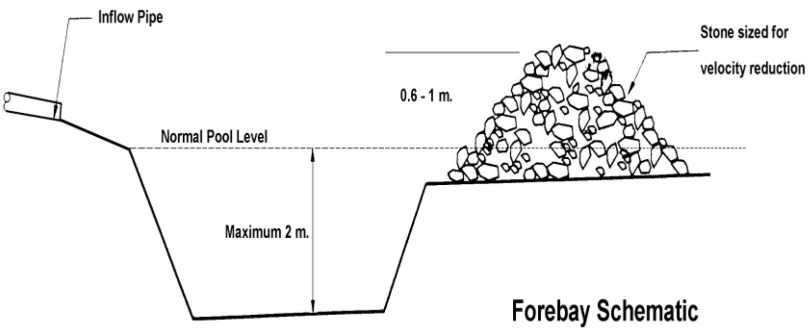
In situations where embankment failure may lead to loss of life or extreme property damage, the emergency spillway must be able to:

- Pass an extreme flood, which may be the Probable Maximum Flood (PMF), with no freeboard (after post-construction settlement) and with the service outlet blocked. The PMF is defined as the largest probable flood event that could occur at the site, or the theoretical upper limit to flood magnitude. The extreme flood (Q_V) is defined as detailed in NIWA Science and Technology Series No. 19, "A Guide to Probable Maximum Precipitation in New Zealand", June 1995. For high-risk dams discussion with the Hawke's Bay Regional Council is essential to determine the needed factor of safety.
- Pass the full Q_{IV} (the 1% AEP event flow) assuming the service spillway is blocked with at least 0.5 metres of freeboard (after construction settlement).

<u>Forebay</u>

A forebay must be provided for all wet ponds. The sediment forebay is intended to capture only coarse sediments and is the location where most frequent sediment clean will be needed because coarser particles comprise the highest proportion of incoming sediments in terms of total volume.

Figure 7-19
Forebay Schematic



Thus the more frequent cleanout of the forebay area. Figure 7-19 provides a schematic for a typical forebay.

The forebay should meet the following criteria:

1. The volume of the forebay should be at least 15 % of the water quality volume (or 30% of the adjusted volume when extended detention is required). It should be cleaned out when filled in to about 50% of its design volume.

2. Flow velocities from the forebay during the 1 in 10 year storm must be less than 0.25 m/s, in order to avoid resuspension of sediment. In some cases this may necessitate more than the minimum forebay volume. The recommended depth of the forebay is 1 metre or more, to reduce velocities.

Hydraulic flow characteristics

1. Calculate the water quality volume to be treated using the 90% rainfall event.

2. Take a minimum of 50% of that volume for normal pool (dead) storage (when detention is required).

3. Use 1.2 times the water quality rainfall to determine the depth of runoff that is to be stored and released over a 24-hour period.

4. Conservatively assume that the entire extended detention volume is in the pond at one time even though this will not actually be the case since the outlet orifice will be sized to release this volume over a 24-hour duration.

    - Use an elevation - storage table to estimate the elevation required to store the full extended detention volume

    - Calculate the average release rate (equal to the volume/duration) = Q_{avg}

    - At the full extended detention design elevation, the maximum release rate is assumed to be $Q_{max} = 2(Q_{avg})$

    - Calculate the required low flow orifice size: $Q_i = 0.62A(2gh_i)^{0.5}$ by trialling various orifice sizes.

    - h_i = elevation difference = 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)
  $Q_i = n\ 0.62A(2gh_i)^{0.5}$

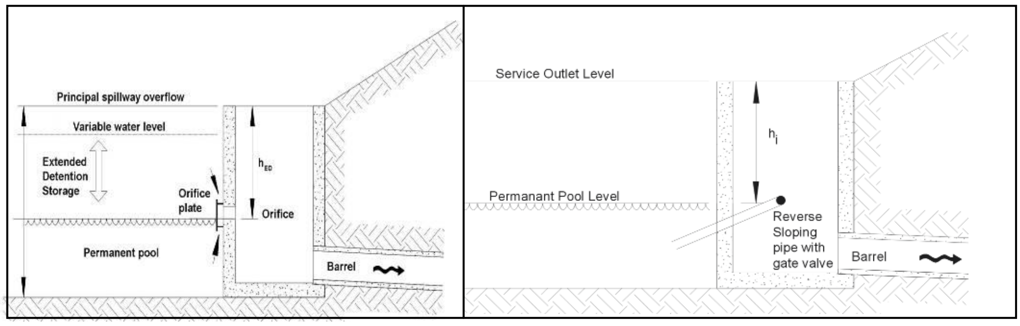
- Vertical slot extending to water surface (width w)
  $Q_i = 1.8\ w\ h_i^{3/2}$

- Vertically spaced orifices (situated h_1, h_a, h_b from surface of pond filled to the WQ volume. Each orifice area A)
  $Q = 0.62A(2gh_1)^{0.5} + 0.62A(2gh_a)^{0.5} + 0.62A(2gh_b)^{0.5}$

- Pipe (area A) $h = (1.5Q_i^2/2gA^2) + h_f$
  where h_f is pipe friction loss

A number of different outlet designs for extended detention are detailed in Figure 7-20.

Figure 7-20
Schematic of Several Extended Detention Outlet Structures



5. 2 and 10 year stormwater management

Set the invert elevation of the 2 year release point at the extended detention water surface elevation (based on the elevation - storage table mentioned in step 4)

The service outlet may consist of 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 frequency storms to be controlled, the discharge is clearly defined in terms of the following equations.

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, (hii) can be used to calculate the flow according to:

$$Q_{ii} = 3.6pR\, h_{ii}^{3/2} \qquad \text{(SI units)}$$

Where R is the radius of the inlet.

For a box weir:

$$Q_{ii} = 7.0wh_{ii}^{3/2}$$

where w is the length of the side 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:

$$h_{ii} = k(v^2/2g)$$

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:

$$v = Q_{ii}/pR^2$$

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 (Q_V), the outlet discharges at 75% full. Standard pipe friction and pipe outlet loss calculations can be performed to determine the required outlet size (USBR, 1977).

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 (see Figure 5-4):

$$Q_{ii} = 0.57(2g)^{1/2}(2/3Lh^{3/2} + 8/15zh^{5/2})$$

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

$$Q_{ii} = 1.7\ L\ h_{ii}^{3/2}$$

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. Appropriate texts may be consulted for refined weir calculations, but 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). The channel may require covering for safety reasons.

6. 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:

$$Q = 0.57(2g)^{1/2}(2/3Lh^{3/2} + 8/15Zh^{5/2})$$

   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)

Designs to avoid short-circuiting

Dead zones and short-circuiting are undesirable because they reduce effective pond detention times. The flow path length must be at least twice the pond width, and preferably three times the width (but not much greater). The narrower the flow path, the greater the velocity and the less settling will occur. The designer should minimise dead zones and short-circuiting to improve the treatment performance of the pond.

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 7-17 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 at the inlet to the pond or at the outlet from the pond.

Ease of maintenance

Ease of maintenance must be considered as a site design component. Access to the stormwater management pond or wetland must be provided for in the design, and land area adjacent to the pond must be set aside for drying out of sediments removed from the pond when maintenance is performed. The land set aside for pond maintenance should be sized as follows:

1. The set aside area shall accommodate at least 10 percent of the stormwater management pond volume at a maximum depth of one metre, and
2. The slope of the set aside area shall not exceed 5 percent, and
3. The area and slope set aside may be modified if an alternative area or method of disposal is approved on a case-by-case basis.

7.4.6.5 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, water flow velocity and detention period. In addition, it is recommended that the length to width ratio be 3 horizontal to 1 vertical or greater to facilitate sedimentation. These parameters should be considered in light of the existing topography. Generally, a pond will look more natural and aesthetically pleasing if it is fitted into existing contours.

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. Generally, sloping margins require a more sophisticated management approach to ensure growth of plants. Areas of gradually varied wetness should be identified and specific planting strategies should be developed for these areas. 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.

Islands

Islands, properly located, can be used to manipulate flow characteristics, to increase the distance that water travels and to help segregate first flush inflow from later flows within a storm event. They also increase the extent of planted margin and can provide a wildlife habitat that offers some protection from domestic animals or people, as well as offering additional aesthetic appeal.

7.4.6.6 Landscaping

Design of a stormwater pond system should ensure that the pond fits in with the surrounding landscape. General landscape design principles will apply. The area should develop a strong and definite theme or character. This might be generated from particular trees, or views from the site, topographical features, or the cultural character of the surrounding neighbourhood. The landscape design for the area will provide a setting for the pond so that the pond will appear a natural component of the overall setting.

7.4.6.7 Case Study

Project Description

A 100 lot residential subdivision is being constructed in Taradale. It is 7.5 hectares in size with no off-site drainage passing through it. It has gentle slopes and pre-development land use is pasture (C = 0.3). Post development is expected to change 50% of the area to imperviousness (roofs, roads etc, with C = 0.9). The site drains into a stream channel so extended detention is a design component.

Hydrology

Water quality storm is 20 mm of rainfall
2-year 1-hour rainfall is 17.7 mm
10-year 1-hour rainfall is 25.3 mm.
Soil condition - Class C

Pre-development peak discharges and volumes are the following::

Q_{wq} = CIA/360

Q = peak discharge
C = Runoff coefficient
I = Rainfall intensity (mm/hr.)
A = catchment area in hectares

Pre-development peak discharge:

$Q_2 = (0.3)(17.7)(7.5)/360 = 0.11 m^3/s$

$Q_{10} = (0.3)(25.3)(7.5)/360 = 0.16\ m^3/s$

Post-development peak discharges and volumes are the following:

Water quality volume
A_{eff} = impervious%/100 x total Area (ha) = 0.5(7.5) = 3.75

The first flush volume $V_{wq} = 10 \times A_{eff} \times d_{ff}$ (m^3) = 10(3.75)(20) = 750 m^3

Post development runoff coefficient = (0.3)(3.75/7.5) + (0.9)(3.75/7.5) = 0.6

$Q_2 = (0.6)(17.7)(7.5)/360 = 0.22\ m^3/s$

$V_{estimated} = 1.5(Q_{post})D = 1.5(.221)(3600) = 1{,}195\ m^3$

$Q_{10} = (0.6)(25.3)(7.5)/360 = 0.32\ m^3/s$

$V_{estimated} = 1.5(Q_{post})D = 1.5(.32)(3600) = 1{,}708\ m^3$

Table 7-11 Summary Table		
Parameter	Pre-development	Post-development
Q_2	$0.11 m^3/s$	$0.221 m^3/s$
V_2		$1,195 m^3$
Q_{10}	$0.16 m^3/s$	$0.32 m^3/s$
V_{10}		$1,708 m^3$
Water quality volume		$750 m^3$
Extended detention (ED) volume (1.2 x WQV)		$900 m^3$

The key elements of the table are the pre-development peak discharges and post-development volumes. The peak discharges cannot exceed the pre-development peak discharges but the volumes to be stored are the post-development ones.

Pond Design

An essential component of pond design is knowing what the available storage is at the pond location. As such, it is important to develop a stage-storage relationship table to calculate the volumes versus depths for storage and discharge purposes.

For this site the following table reflects available site storage.

Table 7-12 Stage-Storage Relationships	
Elevation	Available volume
51.5	0
52.0	735
52.5	1745
53.0	3073
53.5	4763
54.0	6858

As the pond will discharge 1.2 times the water quality volume over a 24-hour period, the permanent water quality volume can be reduced by 50%.

The adjusted water quality volume is 375 m^3 and rises to elevation 51.78.

The sediment forebay must contain a volume of at least 30% of the adjusted water quality volume, so **the sediment forebay must contain 113 m^3**.

The lowest outlet is the extended detention outlet, whose invert is set at a level that impounds the required permanent water quality storage (375 m^3) and the live storage for extended detention (900 m^3). In this case **the elevation of the extended detention volume and water quality volume (1275 m3) is at elevation 52.29**.

The extended detention (ED) outlet is sized to release the extended detention volume (EDV) over a 24-hour period. To do this, the outlet is sized so that when the pond is holding the full EDV the release rate is the following:

$Q_{ED} = 900 m^3/24 \text{ hours} = 0.01 m^3/s$

At the full EDV elevation, the maximum release rate is assumed to be $Q_{max} = 2Q_{ED}$
$Q_{max} = 0.02\ m^3/s$. The discharge through the ED outlet cannot exceed this.

Calculate the low flow orifice by assuming an orifice size and ensuring that the outlet discharge does not exceed Q_{max}.

$Q = 0.62A(2gh)^{0.5}$ where A = area of ED orifice

Try an orifice size of 120 mm diameter

Where h = 52.29 - (51.78 + D/2) where D is the ED outlet diameter = 0.45 m

$Q = 0.62(0.0113)((2)(9.8)(0.45))^{0.5} = 0.02\ m^3/s$ which meets the design criteria. Note that trial and error may be required to arrive at a suitable diameter so as not to exceed the combined ED and 2 (or 10) year discharges. This will be evident when the 2 year and 10 year discharge calculations are carried out.

If the orifice size is less than 50 mm, a cover plate or screen is required to prevent clogging of the orifice.

Extended Detention orifice is 120 mm.

Consideration of 2- and 10-year storm control will consist of consideration of a rectangular weir to provide for the appropriate outflow rates. Peak outflows should not exceed the pre-development peak discharges which are 0.11 m^3/s and 0.16 m^3/s.

To size the weir we can ignore the outflow that occurs during the rainfall and size the weir so the entire runoff volume can be held with the outflow rate not exceeding the pre-development peak flows. Routing of flows through the pond is also acceptable for this calculation but not for determining the ED volume sizing.

2-year event

Pond volume required for the post-development case = 375 (WQ vol.) + 1195 (2-year post-development volume) = 1570 m^3

Ponded water level is at 52.42 m.

Outflow must be determined using the ED orifice and an outlet structure (rectangular weir). (Note: calculations below were performed using a spreadsheet and rounding differences may result in small differences in the answers).

Weir invert level is at elevation 52.29 m.

Outflow from ED orifice = $Q = 0.62A(2gh)^{0.5}$

h = 52.42 -(51.78 + 0.120/2) = 0.59 m

$Q_{ED} = 0.62(0.0113)((2*9.81*0.59))^{0.5} = 0.024\ m^3/s$ from ED orifice.

Outflow over weir = $Q = 1.7\ Lh^{1.5}$ where L = weir width
Try L = 380 mm
$Q = 1.7(0.38)\ (52.42-52.29)^{1.5} = 0.033\ m^3/s$

Total outflow = ED + 2-year discharges = 0.024 + 0.033 = 0.06 which meets the 2-year peak control requirement. Note, weir width could be increased, for the 2 year control, but the 10 year control needs to be considered.

10-year event

Pond volume required for the post-development case = 375 (WQ vol.) + 1708 (10-year post-development volume) = 2083 m^3

Ponded water level is at elevation 52.64 m.

Outflow must be determined using the ED orifice and the 2-year weir control.

Outflow from ED orifice = $Q = 0.62A(2gh)^{0.5}$

h = 52.64 -(51.78 + 0.120/2) = 0.80 m

$Q_{ED} = 0.62(0.0113)((2*9.81*0.80))^{0.5} = 0.03$ m^3/s from ED orifice.

$Q_{ED} = 0.03$ m^3/s

10-year weir flow = $1.7Lh^{1.5} = 1.7(0.38)(52.64\text{-}52.29)^{1.5} = 0.13$ m^3/s.

Total peak discharge using 2-year weir and ED orifice = 0.03 + 0.13 = 0.16 m^3/s which means that **the 10-year weir width of 380 mm in conjunction with the ED orifice will control both the 2- and 10-year storms.**

7.4.7 Wetlands

Description: Wetlands are designed and constructed to capture and treat stormwater runoff through:

- Sedimentation
- Filtration
- Adsorption, and
- Biological uptake

Stormwater Management Function

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

Wetlands are complex natural water environments that are dominated by hydrophytic (water loving) vegetation. They differ from stormwater wet ponds that are dominated by large areas of open water.

Until recently, the filling and draining of wetlands was accepted practice to "improve" land. We now know that wetlands provide many important benefits including the attenuation of flood flows, maintenance of water quality and support aquatic life and wildlife.

Constructed wetlands have become increasingly popular in recent years for improvement of water quality. Wetlands can be designed to accomplish a number of purposes and Wong et al (1998) provides the following list of purposes and benefits that are commonly combined:

- Flood protection,
- Flow attenuation,
- Water quality improvement,
- Landscape,
- Recreational amenity, and
- Provision of wildlife habitat

From a contaminant removal perspective, wetlands provide a number of different removal processes that are not available in deeper wet ponds. Those removal processes are listed in Table 7-12.

Table 7-12
Overview of Stormwater Contaminant Removal Mechanisms of Constructed Wetlands (adapted from Mitchell, 1996)

Contaminant	Removal Processes
Organic matter	Biological degradation, sedimentation, microbial uptake
Organic contaminants	Adsorption, volatilisation, photosynthesis and biotic/abiotic (pesticides) degradation
Suspended solids	Sedimentation, filtration
Nitrogen	Sedimentation, nitrification/denitrification, microbial uptake, plant uptake, volatilisation
Phosphorus	Sedimentation, filtration, adsorption, plant and microbial uptake
Pathogens	Natural die-off, sedimentation, filtration, predation, UV degradation, adsorption
Heavy metals	Sedimentation, adsorption, plant uptake

A key benefit of a stormwater wetland is its shallow nature. The shallow nature promotes dense vegetation growth that acts as a natural barrier to small children or the general public. Being shallow water systems, they do not have the safety concerns that deeper ponds have. Fewer safety concerns is an important consideration in selecting wetlands for water quality treatment.

7.4.7.1 Basic design parameters

It is important to specify the contaminants that a stormwater treatment wetland is designed to treat, as effective treatment of different contaminants can require markedly different detention times within the wetland.

Suspended solids are at one end of the treatability spectrum and require a relatively short detention time to achieve a high degree of removal. At the other end of the treatability spectrum are nitrogen and phosphorus. Given sufficient area and time, wetlands are capable of removing nutrients to very low levels but their efficiency depends on their design.

The most common design priority for vegetated wetlands and highways will be for the removal of:

- Sediments,
- Hydrocarbons
- Dissolved metals

Wetlands are most appropriate on sites that meet or exceed the following criteria:

- Catchment areas at least 4 hectares in size (Table 5-4)
- Soils that are silty through clay
- No steep slopes or slope stability issues
- No significant space limitations

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

Design parameters for wetlands are the same as the parameters for wet ponds in the context of storm peak control and stream channel erosion control. So the same design procedures need to be gone through. There is some difference in water quality sizing related to the following:

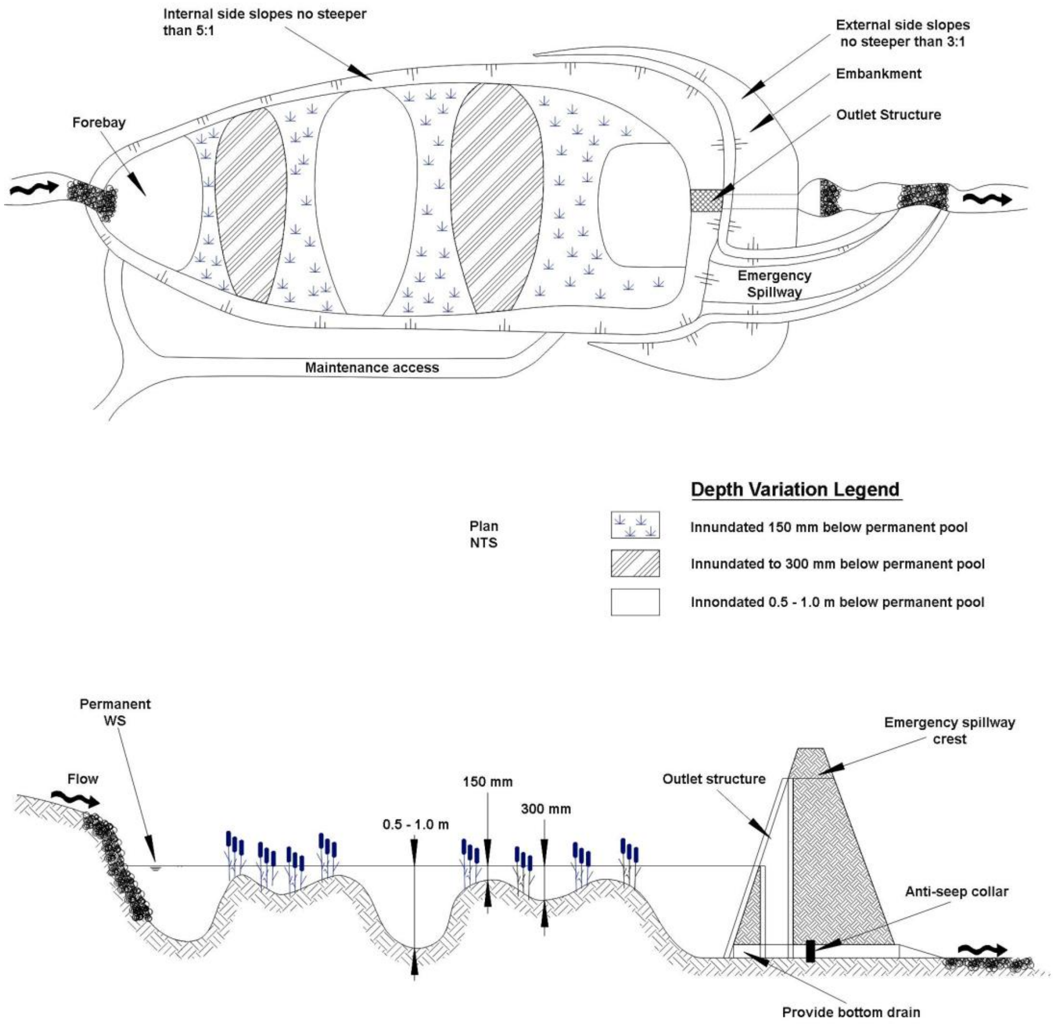
- Depth of standing water and
- Water quality volumes

Depth of standing water

Wetlands are shallow water systems and do not contain large volumes of water per surface area as do wet ponds.

The designed approach for a constructed wetland is the banded bathymetric design as detailed in Figure 7-21. A banded bathymetric design is preferred for having variable depth that allows for dispersed flow of stormwater through vegetation and has deeper areas for fish, which will assist in preventing mosquito problems from resulting.

Figure 7-21
Banded Bathymetric Wetland Schematic



Profile
NTS

The proposed depth ranges and areas for a vegetated wetland having a banded bathymetric design are the following:

Banded bathymetric design	% total wetland pool area
Dead storage 0.5 -1.0 m depth	40
Dead storage at 0 – 0.5 m depth	60

In the event that a banded bathymetric design is not used, another approach would be to use a trapezoidal design. That design has a more uniform depth (still less than 1 m depth) and may not provide the same fisheries habitat.

Trapezoidal bathymetric design	% total wetland area
Dead storage at 1 m depth	20
Dead storage at 0 – 1 m depth	80

The banded bathymetric design is recommended over the trapezoidal bathymetric design due to its configuration providing a better expectation of uniform flow throughout the wetland. The trapezoidal design may have vegetation developing unevenly and allow for short-circuiting.

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

<u>Water quality volumes</u>

As Table 7-13 lists the variety of removal processes that wetlands use to remove contaminants, sedimentation is only one of those processes with the others relying on contact between stormwater contaminants and plants and organic matter. As can be seen from Figure 7-18, wetlands are shallow water systems and rely more on surface area than on having a specific volume of storage.

There are several approaches to considering a surface area requirement. The first approach is to use the wetland surface area as a proportion of the catchment area and a recent publication (Cappiella, Fraley-McNeal, Novotney, Schueler, 2008) recommends a ratio of wetland area to catchment area of 3%. In a similar fashion a report out of Australia (Wong, Breen, Somes, Lloyd, 1999) relates hydrologic effectiveness to wetlands having a surface area as a percentage of catchment area and indicates a desirable ratio of approximately 2% for a catchment of 30% imperviousness and a 72-hour detention time for nitrogen reduction.

Another approach is to relate hydrologic effectiveness to wetland storage as a percentage of annual runoff volume. The same report (Wong, et. Al 1999) shows a “knee” point of approximately 2% where benefits start to wane for further increases in storage. Again, this design is based on the removal of nutrients as a key objective.

The ratios in both publications relate to nutrient capture and may be considered appropriate where highway runoff is discharged to lakes but in general the contaminants of concern on highways are sediments and metals (lead, copper, zinc) and they don’t need the same detention time as nutrients for significant removal.

As a result the recommended approach for wetland design is to have the surface area of the wetland as 2% of the overall catchment area draining to the wetland.

7.4.7.2 Detailed design procedure

The design basis for a stormwater wetland is twofold:

- Water quality objectives are achieved by sizing the wetland surface area to 2% of the catchment drainage area draining to the wetland. The wetland depths are then provided through the relative depths provided in the above depth discussion.
- Intermediate storm control and extended detention objectives are met through the same calculations discussed in the wet pond Chapter.

The design steps are the following:

1. Calculate the wetland surface area as at least 2% of the contributing catchment area.
2. The shape of the wetland should generally be that its length should be at three times its width. These criteria can be relaxed if extended detention were required as flows will be significantly reduced and the length to width ratio is not as important.
3. Using the depth discussion above ensure that the percentage of wetland depths meet the above criteria with a banded bathymetric design being preferred.
4. Calculate the water quality volume that the wetland would have in an identical approach to the wet pond water quality volume. Take 15% of that volume as the necessary volume of an emergency spillway. The surface area determined from this approach can reduce the wetland surface area, as the two areas together will meet the 2% criteria.
5. Determine whether the project needs peak flow control and stream channel erosion control through extended detention.
6. Do calculations identical to the wet pond design for extended detention release sizing and outlet sizing for the 2- and 10-year storms.

Table 7-13 provides a list of plant species for general consideration. Plants for a given project should be considered for suitability in the Hawke's Bay Region.

Table 7-13 Preferred Vegetation

Following is a list of the preferred wetland vegetation and it's normal depth

Deep zone 0.6 – 1.1m	
Baumea articulata	Typha orientalis (raupo)
Eleocharis sphacelata	Myriophyllum propinqum (water milfoil)
Schoenoplectus validus	Potamogeton cheesemanii (manihi)
Shallow zone: 0.3-0.6m	
Baumea articulata	Schoenoplectus validus
Bolboschoenus fluviatilus	Typha orientalis
Eleocharis sphacelata	Isolepis prolifer
Eleocharis acuta	Juncus gregiflorus
Carex secta	
Wet margin 0-0.3m	
Baumea teretifolia	Juncus gregiflorus
Baumea rubiginosa	Carex virgata
Carex secta	Cyperus ustulatus (giant umbrella sedge)
Eleocharis acuta	Phormium tenax (flax)
Live storage zone (periodically inundated)	
Syzygium maire (swamp maire)	Dacrycarpus dacrydioides (kahikatea)
Carex virgata	Cordylina australis (cabbage tree)
Carex lessoniana (rautahi)	Baumea rubiginosa
Carex dissita (flat leaved sedge)	Phormium tenax (flax)
Cyperus ustulatus	Coprosma tenuicaulis (swamp coprosma)
Juncus articulatus	Blechnum novae-zelandiae (swamp kiokio)
Juncus pallidus	
Land edge	
Coprosma robusta (karamu)	Schefflera digitata (pate)
Phormium tenax	melicytus ramiflorus (mahoe)
Cordyline australis	Pneumatopteris pennigera (gully fern)
Carpodetus serratus (putaputa weta)	Dacrycarpus dacrydioides (kahikatea)
Laurelia novae-zelandiae (pukatea)	Cortaderia fuluida (toetoe)
Leptospermum scoparium (manuka)	

7.4.7.3 Case study

The case study is the same case study as the wet pond design but designing a wetland instead.

Project description

A 100 lot residential subdivision is being constructed in Taradale. It is 7.5 hectares in size with no off-site drainage passing through it. It has gentle slopes and pre-development land use is pasture (C = 0.3). Post development is expected to change 50% of the area to imperviousness (roofs, roads etc, with C = 0.9). The site drains into a stream channel so extended detention is a design component.

The total catchment area is 7.5 hectares and the soils are typical clay soils. Pre-development adjacent land use is pasture and the site drains to the upper part of a stream.

Components to examine are:

- Peak flow control of the 2- and 10-year storms
- Extended detention of 1.2 x WQ storm
- Water quality treatment

Hydrology

Water quality storm is 20 mm of rainfall
2-year 1-hour rainfall is 17.7 mm
10-year 1-hour rainfall is 25.3 mm.
Soil condition - Class C

Table 7-14 (from earlier table) Summary Table		
Parameter	Pre-development	Post-development
Q_2	0.11 m^3/s	0.221 m^3/s
V_2		1,193 m^3
Q_{10}	0.16 m^3/s	0.32 m^3/s
V_{10}		1,728 m^3
Water quality volume		750 m^3
ED volume (1.2 x WQ *V)		900 m^3

Wetland design

1. Water quality Volume = 750 m^3 and the wetland forebay must be 15% of the water quality volume.

   Sediment forebay size is 113 m^3

   The surface area of the wetland will be 2% of the contributing catchment area, which is 7.5 hectares.

   Wetland surface area is 1500 m^2

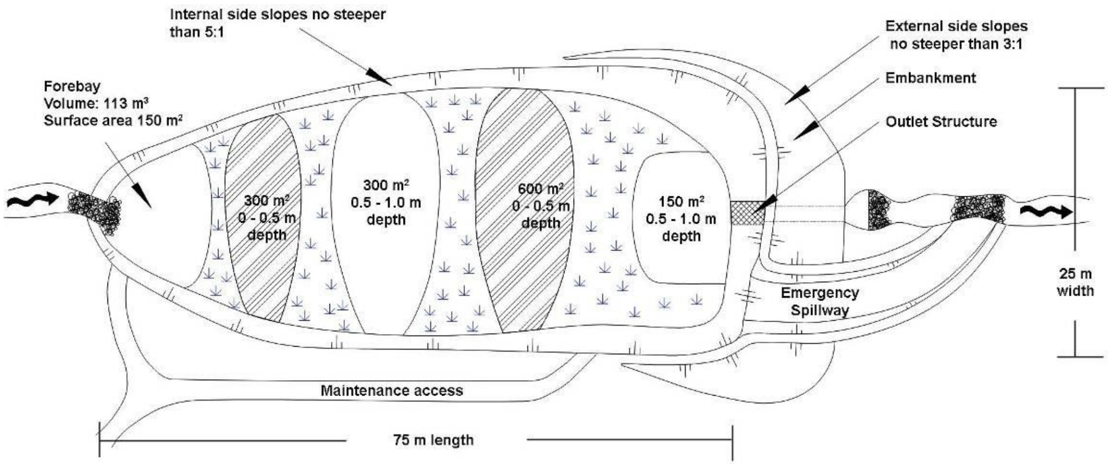
   Since extended detention is a design requirement, the length to width ratio is not as important but for this case study a length to width ratio would provide a general shape of approximately 25 metres wide by 75 metres long.

   To have the depths defined we use the relationships provided above.

Banded bathymetric design	% total wetland pool area	Area Extent (m^2)
Dead storage 0.5 -1.0 m depth	40	600
Dead storage at 0 – 0.5 m depth	60	900

   Figure 7-19 shows this visually.

Figure 7-22
Case Study Percentage Areas



As the forebay elevation is considered part of the wetland surface area, the areas detailed in the banded bathymetric design have been reduced proportionally to account for the forebay area. If the individual areas are added together the total recommended levels are achieved.

2. Extended detention design and peak storm control are done identically as the wet pond design detailed design procedure. They are not replicated here but are detailed in the Wet Pond case study section under the Extended Detention and 2- and 10-year sections of the case study.

7.4.8 Green roofs

7.4.8.1 Description of practice

Description: Green roofs are roofs with a growing media that reduces stormwater runoff through evaporation and evapotranspiration. Their primary benefit from a stormwater management perspective is to reduce the total volume of stormwater runoff.

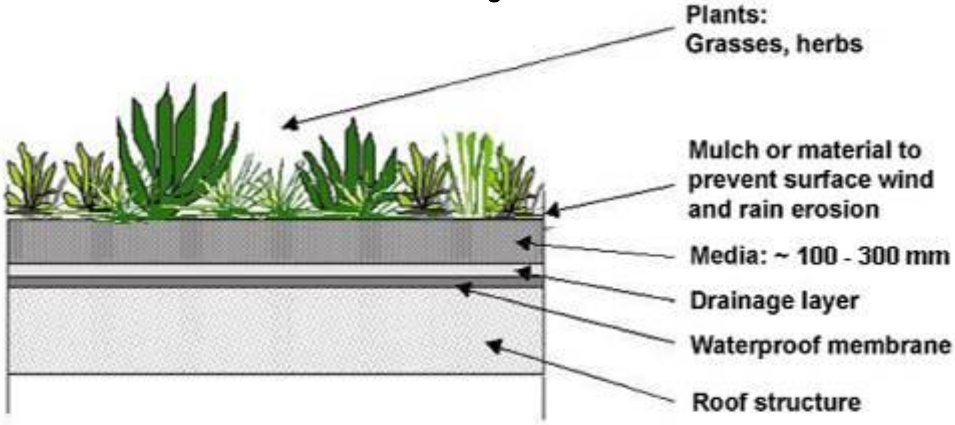
Stormwater Management Function	
✓	Water quality ✓ Metals ✓ Sediment
X	Flood protection
✓	Stream channel erosion protection

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

7.4.8.2 Design considerations

Typically, as shown in Figure 7-23, a green roof consists of the following:

Figure 7-23
Green Roof Cross-section Showing Elements



- A waterproof membrane to prevent water from leaking into the structure,
- A drainage layer to allow lateral movement of water to the down spout,
- Filter media for passage of stormwater and a growth media for plants,

- Mulch or other material to prevent surface wind and rain erosion, and
- Plants.

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

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

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

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

7.4.8.3 Targeted contaminants

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

7.4.8.4 Advantages

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

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

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

7.4.8.5 Limitations

There are several issues that may be considered as limitations.

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

7.4.8.6 Design sizing

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

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

7.4.8.7 Depth of media

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

While these are both fairly new installations, some guidance can be given on plant propagation that relates to the depth of media. The University of Auckland site has media between 50 mm - 70 mm in depth. Over the 2007-2008 summer, plants were severely stressed due to the lack of moisture in the shallow subgrade. The Waitakere City green roof fared much better due to its depth being 70 - 150 mm.

Deeper media depths are better than shallower ones.

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

7.4.8.8 Composition of media

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

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

7.4.8.9 Plant selection

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

Recommended plants include the following:

- Disphymae australe (NZ ice plant)

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

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

7.4.8.10 Additional support consideration

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

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

7.4.8.11 Roof slope

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

7.4.8.12 Drainage layer and impermeable liner

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

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

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

7.4.8.13 Stormwater management benefits

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

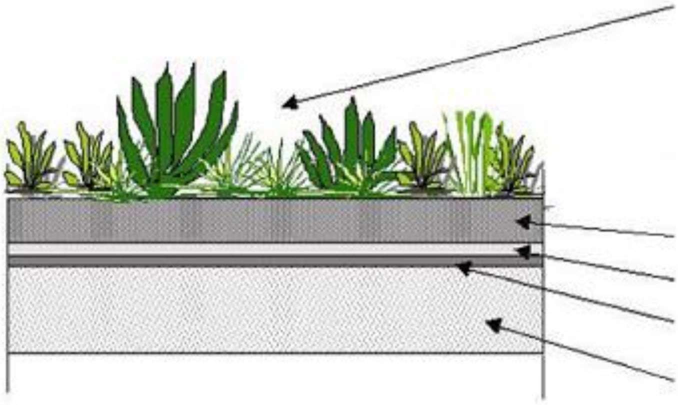
The media recommended includes zeolite, which is a hydrated aluminosilicate mineral having a micro-porous structure. Pumice also has a very high porosity and being highly porous is very lightweight. Design can assume a 50% void ratio for the compost bark, zeolite and pumice.

Stormwater quantity control is not required for green roofs.

7.4.8.14 Case study

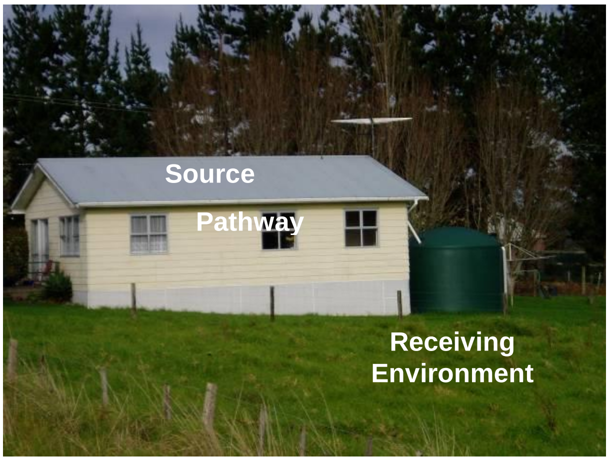
This is a typical green roof design is shown in Figure 7-24.

Figure 7-24
Case Study Parameters for a Green Roof



7.4.9 Water tanks

7.4.9.1 Description of practice



Description: Water tanks provide detention storage for stormwater runoff and water supply for domestic use. They reduce stormwater runoff through domestic use and thus reduce the total volume of stormwater being discharged during a storm event.

Stormwater Management Function	
✓	Water quality ✓ Metals ✓ Sediment
X	Flood protection
✓	Stream channel erosion protection

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

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

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

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

7.4.9.2 Design considerations

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

- The annual average rainfall amount and inter-event dry periods,
- The roof area,
- The anticipated water use,
- The percent of water from the roof that can be used,

- Peak flow considerations, and
- Sizing outlets

It is assumed that water tanks, in the context of this Guideline, will be both full service tanks and limited to non-potable uses.

It is not intended in this Guideline that roof areas compensate for impervious surfaces beyond the roof area itself.

7.4.9.3 Targeted contaminants

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

7.4.9.4 Advantages

Water tanks have several advantages.

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

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

7.4.9.5 Limitations

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

Where water tanks are the only means of providing domestic water (residential use), the minimum tank size shall be 25,000 litres.

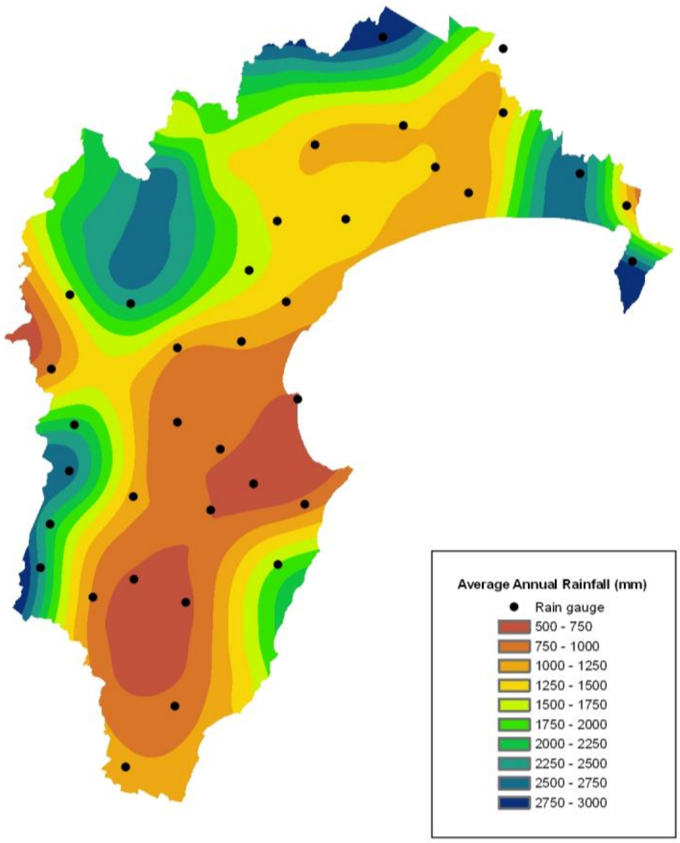
7.4.9.6 Design sizing

As mentioned in Section 7.4.9.2, there is a logical progression of analysis that needs to be done for water tank sizing.

The average annual rainfall amount and the inter-event dry periods

Much of lowland Hawke's Bay Region receives approximately 800 mm of rainfall per annum but this level is expected to decrease due to global warming. The overall Hawke's Bay Region has the most variation in annual rainfall in New Zealand as shown in Figure 7-25. The Hawke's Bay Region also experiences annual water

Figure 7-25
Average Annual Rainfall for the Hawke's Bay Region



deficits in the 300-500mm range. Most of the rainfall is usually distributed from April (or mid to late autumn) to the beginning of January with the summer months being a period of rainfall deficit. In other words, it is very difficult to design a water tank system that can meet needs due to a very variable nature of rainfall.

Inter-event dry periods are important to consider if the water is being used for domestic or industrial use. During periods of dry weather additional water is not available so storage must be provided for those expected dry periods. For the Hawke's Bay Region the following information is provided:

- Through 136 years of record the average dry period between storms that are greater than 5 mm during the summer is 12.9 days.
- The mean annual number of dry days in the summer is 43 days and the 5-year average is 52 dry days.
- During winter the average dry period is 11.4 days with the annual number of dry days being 36 days and the 5-year average number of dry days is 44 days.

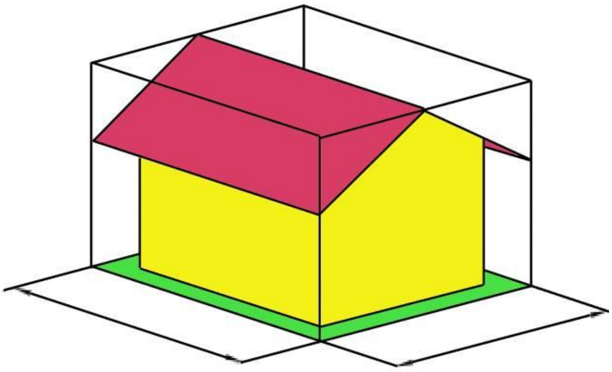
Although there is little difference in the inter-event dry period between winter and summer there is some slight difference with the average dry period in summer being approximately 13 days. That means that the volume of storage needs to be provided for the daily-anticipated water usage multiplied by at least 13 to provide for needs during the dry periods. It must be recognised that the 13 is an average value and additional storage would provide longer-term protection.

The first bullet mentions rainfall greater than 5 mm as the basis for determination of dry periods. Given that a roof is impervious some additional runoff will be provided by these small events but the relative contribution will be minor so larger volumes of storage would be beneficial.

How much water can be captured from the roof

The first aspect of design is to calculate the roof area that will be drained to a water tank. Figure 7-26 details how that is done. The area that is green and covers the whole plane of the green area is the roof area that is then used in calculations.

Figure 7-26
Calculation of Rood Surface Area



When calculating catchment area, measure at ground level below edges of the roof, including eaves

Another component of roof runoff capture is what percentage of stormwater that runs off the roof can be used depending on roof area, tank size, daily usage and whether there are detention requirements associated with roof imperviousness.

The anticipated water use

Table 7-15 provides information on anticipated water use for residential properties. The values can be extrapolated for more or less members but an average assumption of three members is reasonable given the potential of people relocating. It is recommended that water use for a residence be 500 litres/day unless there is evidence that the actual number will be more or less and that number is expected to remain standard for at least 10 years. Otherwise use 500 litres/day as the average water use.

Table 7-15
Estimated Residential Demand Based on 500 l/d for a 3 member household

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

The same assumption cannot be made for rural commercial or industrial land use. In this situation, assumptions need to be made regarding the number of people that will occupy the workplace. Table 7-16 provides information on occupancy ratios.

Table 7-16
Building Occupancy Ratios for Different Activities (NSCC, 2008)

Activity	Floor to Person Ratio
Office	25 m^2
Showroom	35 m^2
Warehouse	50 m^2
Shops, retail	35 m^2
Restaurant/dining areas	15 m^2
Local shopping centres	35 m^2
Manufacturing	25 m^2

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

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

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

<u>The percent of water from the roof that can be used</u>

There will be periods of time when the water tank is full due to longer periods of wet weather. The concern related to this situation is when detention storage is required for peak discharge control. It is not an issue for domestic or business use as more water does not present a problem related to consumption.

As a guide to collection capacity, consider that each **1mm of rain = 1 Litre (L) of water per square metre (m^2)** of roof area, then allow a 15% wastage factor. This will allow for a good understanding of whether the roof can provide the needed amount of water that is needed.

As an example, 800 mm of rainfall on a 200 m^2 roof would result in 160,000 litres/year - 24,000 litres = 136,000 available litres for site use. If partial site usage was 325 litres/day then having an adequately sized water tank could provide for 100% of site usage while reducing stormwater runoff.

The 15% wastage factor accounts for the time of year that the tank overflows due to rainfall exceeding tank storage.

<u>Peak flow consideration</u>

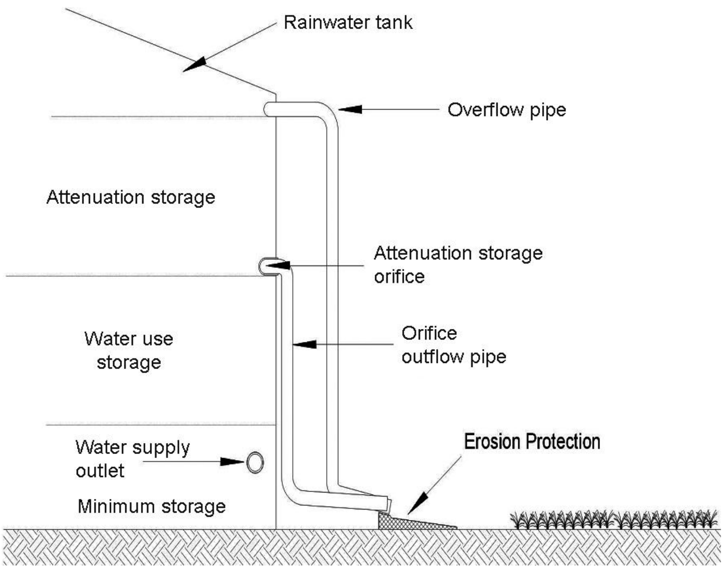
When sizing a water tank, there are two possible storage components.

- The water needs component, and
- An attenuation volume that reduces peak rate of discharge.

The attenuation volume occupies the upper storage area of the tank with its outlet orifice placed immediately above the water needs volume as shown in Figure 7-27.

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

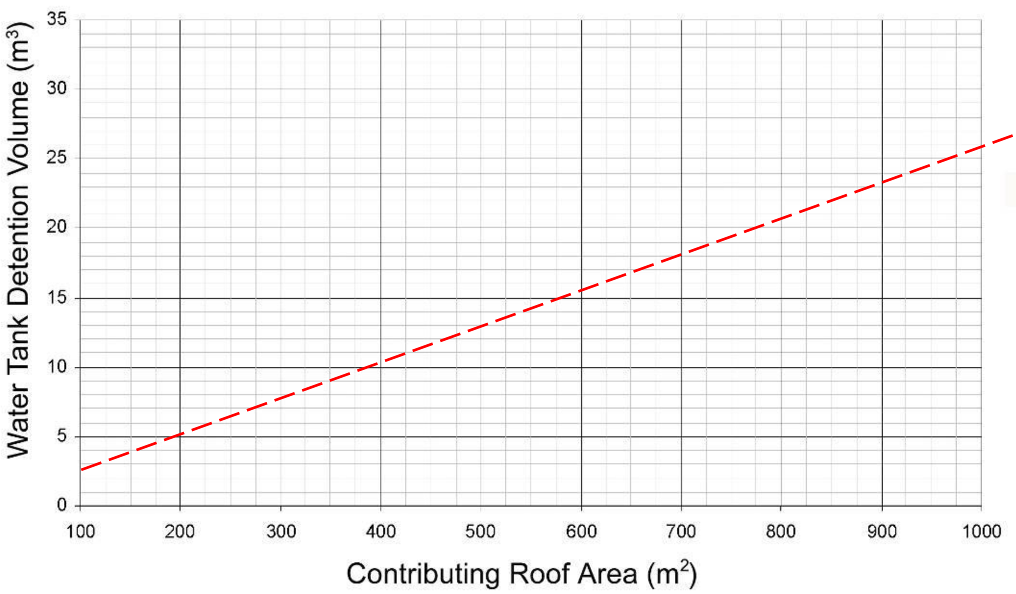
Figure 7-27
Combination Attenuation and Water Use Tank



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

Normally, detention volumes would be determined by the difference in volumes of the pre-and post-development 2- or 10-year storms. Using the 10-year storm as the worst-case scenario, the 10-year 1-hour rainfall should be used for storage purposes. The amount of detention storage required depends on the daily water use by the commercial or industrial operation. Storage volumes related to roof area are shown in Figure 7-28.

Figure 7-28
Water Tank Detention Figure



The required orifice size is a function of the storage volume and the depth of water above the orifice. This depends on the tank size selected to accommodate the water use and attenuation volume. The tank diameter, in conjunction with the attenuation volume to be stored will provide the depth of water number. This number will then be used with an orifice equation to determine the discharge from the tank and that discharge should not exceed the 10-year peak discharge.

The design approach is to determine the Q_{10} for the predevelopment condition and design the orifice size based on the depth of attenuation storage in the tank and the limitation on peak discharge.

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

In those situations, attenuation of runoff may be required due to a possible larger expanse of roof area in conjunction with smaller water use.

As detailed in Figure 7-27, the water tank has three outlets:

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

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

Determining detention volumes and sizing outlets

The volume of storage needed for detention purposes can be addressed with one storage volume, as the volume needed for the 2- and 10-year storms is very similar. The only difference would relate to the size of the outlet orifice. For the purposes of The tank elevations can be calculated once the attenuation storage and orifice size have been determined per the following.

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

7.4.9.7 Case studies

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

7.4.9.8 Case Study 1

A water tank is proposed for a Rissington location has to be sized for a home. The architects design plans show that the home has a roof area of 250 m^2 and it is being designed for a daily water use of 500 litres/day as the tank is the sole supply of domestic water.

Design steps

1. With the roof area being 250 m^2 and a water use of 500 l/d, calculate the calculate the amount of water that can be used at Rissington where the annual rainfall is 850 mm. With one m^2 of roof area providing one litre of water, the total amount of water available is 212,500 litres minus a 15% wastage factor. So annual amount of rainfall that can be used for water supply is 180,625 litres.
2. Daily water consumption is 500 litres/day or 182,500 litres. This indicates that regardless of how large the water tank is, there will be a periodic need to have water supplied to top the system up unless the roof area is increased.
3. At a usage rate of 500 litres/day, the minimum tank size has to accommodate 6500 litres for dry periods. As the water tank is the sole source of water for the residence it is recommended that the water tank be sized to hold 25,000 litres to account for times where large storm capture can augment supply.
4. Except for large storm events, where the ground would be saturated anyway, the residence roof area becomes non-contributing of stormwater runoff so no further consideration of roof runoff management is required.

7.4.9.9 Case study 2

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

Design steps

1. Based on the gross floor area calculate the number of individuals who will be working in the dairy. Use a local shopping centre figure of 35 m^2/person to calculate the number of individuals. The result is 4.7 individuals so say 5 people working at any one time.
2. Calculate the water used by 5 individuals at 25 l/individual or 125 l/d of water being used.
3. Select a storage amount that will supply needs. At 125 l/d 3000 litres would last 24 days if there is no additional rain during that period. The selection of volume should be based on a reasonable assumption of storage needs during the summer months when several weeks can go by without rainfall.
4. Based on Figure 7-28, the amount of detention storage needed is 5 m^3 or 5,000 litres.
5. Q_{10} = 0.001 m^3/s pre-development.
6. Calculate the minimum storage volume needed. If the tank is a 10,000 litre tank its dimensions will be 2.2 m diameter and 3.2 metres tall. To check the

minimum storage volume amounts, divide the volume of the tank by its height and multiply by 200 mm. 10,000/3200 x 200 = 625 litres.

7. The tank volume needs to be 3,000 litres + 5,000 litres + 625 litres = 8,625 litres so the 10,000 litre tank has more than enough capacity.
8. Determine the elevations of the various storages. Minimum storage level = 200 mm. Site water use = 3200/10,000 = 0.32 mm/l x 3000 l = height of water use elevation or 960 mm. This must be added to 200 mm to get elevation in tank of attenuation orifice invert = 1,160 mm from the bottom of the tank. The detention storage of 5,000 litres has an elevation of 5,000x.32 = 1,600 mm.
9. Detention orifice size is determined by trial and error using the following orifice equation.

   $Q = 0.62A(2gh)^{0.5}$ where A = area of detention orifice

   Try a 20 mm orifice
   h = 1.59 for orifice flow

   $Q_{10} = 0.001\ m^3/s = 0.001$ so a 20 mm orifice provides detention storage so runoff does not exceed the pre-development peak discharge rate.
10. Calculate invert height of overflow pipe. Overflow invert height = height of tank/volume of tank (0.32) x attenuation storage volume (5,000) = height of overflow pipe invert elevation or 1600 mm. This must be added to the site water use orifice invert elevation to get the correct overflow elevation = 2,750 mm from the tank invert as a minimum elevation. As the tank is 3.2 metres tall, additional storage can be provided to the site water use to increase total storage for additional safety of supply.

Figure 7-28 shows a tank cross-section with elevations provided. Figure 7-29 shows a detail of the attenuation orifice and the exterior overflow pipe.

Figure 7-28
Water Tank Schematic Showing Case Study Elevations and Volumes

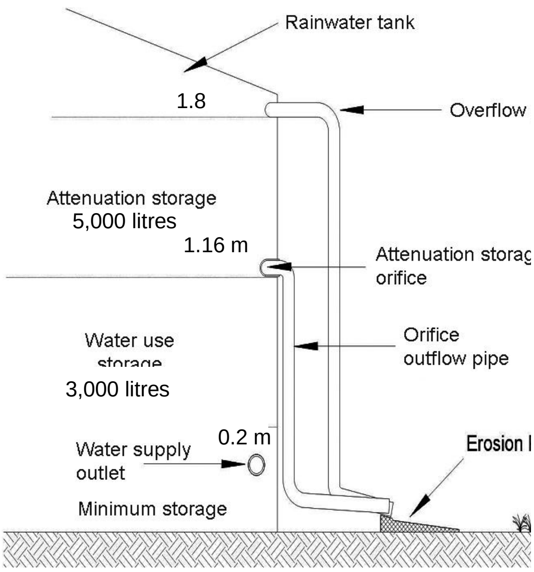
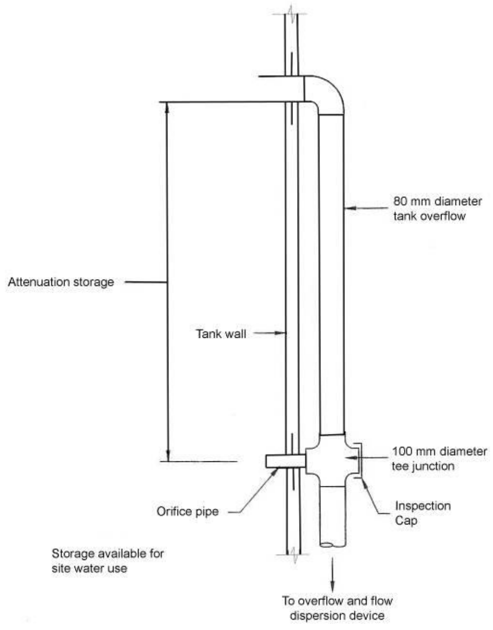


Figure 7-29
Orifice and Exterior Pipe Details



Notes

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

7.4.10 Oil/water separators

Description: Oil and Water Separators are designed and constructed to capture and treat stormwater runoff through:

- Specific gravity separation
- Surface area increases
- Sedimentation (limited)

Stormwater Management Function

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

Oil water separator devices are applicable for treating stormwater runoff from areas where hydrocarbon products are handled or where hydrocarbon loads can be very high. They should be located as close to the source of the hydrocarbons as possible to retain the oil in a floatable, non-emulsified form.

Oil/water separators are not usually applicable for general urban stormwater runoff treatment as the oil is often emulsified or has coated sediments and is too difficult to separate. For stormwater runoff, oil/water separators would primarily be applicable in areas where there is a very high hydrocarbon load and the oil/water separator would be used in conjunction with another practice to function as part of a treatment train.

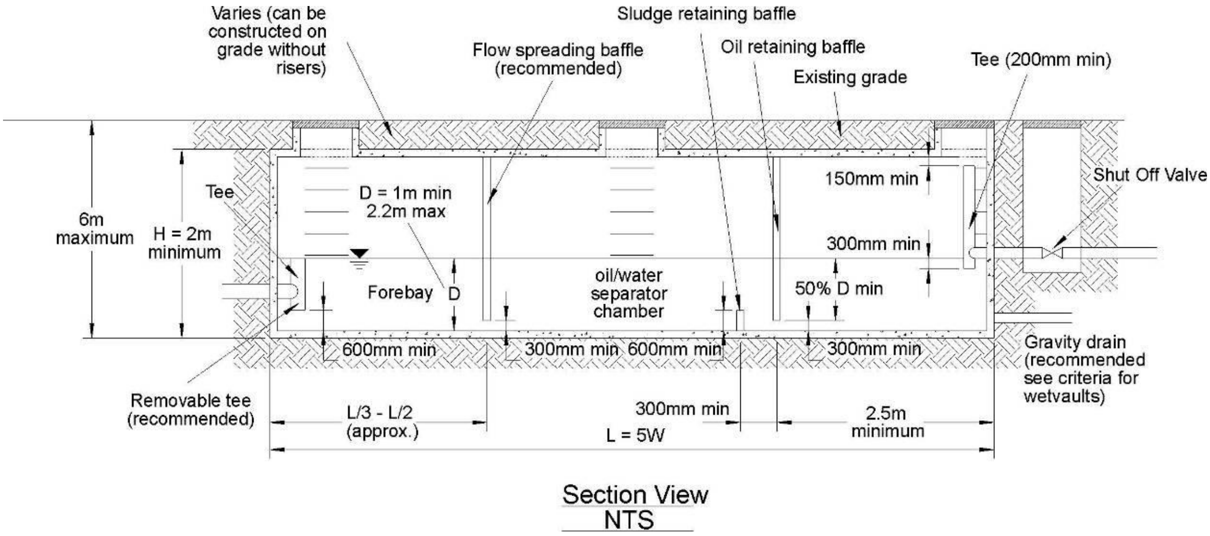
Emulsification occurs when two liquids that normally do not mix do so either through a turbulent environment or through the use of an emulsifying agent. In the case of oil/water separators, the turbulence of stormwater flows can cause the mixing of oil and water. It is important that catchment areas draining to oil/water separators be as small as possible to reduce the potential for emulsification to occur. If that happens the effectiveness of oil/water separators will reduce significantly.

In areas where there is significant potential for accidental spills, oil/water separators may be applicable if the material having spill potential has a specific gravity less than water. From a sedimentation standpoint, oil/water separators will capture sand or grit particles but smaller sediments will either pass directly through the system or may be resuspended in subsequent storms.

There are a number of different products that are available for use as oil/water separators. This discusses the one most commonly used: the API (American Petroleum Institute) separators. The other available products should be designed according to their manufacturers' recommendations.

API separators use baffles to ensure that oil droplets rise above the outlet openings so the oil is trapped in several different compartments. A cross-section of an API separator is shown in Figure 7-30.

Figure 7-30 - API Separator Schematic



The API is discussed in good detail in the MfE Guidelines (1998).

7.4.10.1 Basic Design Parameters

Oil and water separators can be designed to remove oil and TPH down to 15 mg/l. Their performance depends on a systematic, regular maintenance programme. Without that programme, oil and water separators may not achieve oil and TPH removal to the required level.

In light of overseas experience that oil and water separators used for stormwater treatment have not performed to expectations, proper application, design, proper construction and operation and maintenance are essential. Other treatment systems, such as sand filters or other emerging technologies should be considered for removal of insoluble oil and TPH.

The following general design criteria should be followed:

- If practicable, determine oil/grease and TSS concentrations, lowest temperature, pH, and empirical oil rise rates in the runoff, and the viscosity and specific gravity of the oil. Also determine whether the oil is emulsified or dissolved. Do not use oil and water separators for the removal of dissolved or emulsified oils such as coolants, soluble lubricants, glycols and alcohols.
- Locate the separator off-line and bypass flows in excess of the water quality storm flow rate.
- Use only impervious conveyances for oil contaminated stormwater.
- Oil and water separators are not accepted for general stormwater treatment of TSS.

7.4.10.2 Design Procedure

Grease and oil, which is not emulsified, dissolved or attached to sediment, will be present as oil droplets of different sizes or as a surface slick.

No data are available on the size distribution of oil droplets in stormwater from commercial or industrial areas, but some data are available for petroleum products storage terminals. These data indicate that about 80% of droplets (by volume) are greater than 90 *u*m and 30% are greater than 150 *u*m in diameter.

Traditionally, 150 *u*m separation has been used, which typically results in an effluent oil and grease concentration of 50 - 60 mg/l. Typically, standards for industrial discharges in Australia are 10 - 20 mg/l, which generally corresponds to the removal of droplets larger than 60 *u*m.

Separation of the 60 *u*m droplet will be adopted as the basis for design for devices in this Standard, which corresponds to the lower tail of the droplet size distribution and should result in an effluent quality of 10 - 20 mg/l at the design flow.

The rise velocity for a 60 *u*m droplet can be calculated, given the water temperature (which affects the viscosity of the water) and the density of the oil. This rise velocity is then used in the sizing calculations for the device. At 15^{o}C and for an oil specific gravity of 0.9, the rise velocity of a 60 *u*m droplet is 0.62 m/hr and this is the recommended value.

The use of oil specific gravity of 0.9 is considered appropriate for general use as diesel has a specific gravity of 0.85, kerosene of 0.79, and gasoline has a specific gravity of 0.75.

For other conditions, the rise velocity may be calculated according to:

$$v_r = (gD^2\ (1\text{-}s)) / 18\upsilon$$

where:

s	= specific gravity of the oil
D	= droplet diameter
υ	= kinematic viscosity of the water
g	= gravitational acceleration

A key element in oil/water separator design is the design flow rate. That can be determined from the Rational Formula and using the 90% storm.

7.4.10.3 Detailed design procedure

The API area (A_d) is based on the rise velocity (V_r) and design flow rate (Q_d), according to the formula

$$A_d = (FQ_d)/V_r$$

Where:

A_d	= API area (m^2)
F	= dimensionless
Q_d	= design flow rate (m^3/hr)
V_r	= rise velocity

Table 7-17 provides values of F related to horizontal and rise velocities:

Table 7-17 F Factor for API separators	
U/V_r	F Factor
15	1.64
10	1.52
6	1.37
3	1.28

Based on plug flow, the above relationship ensures that a droplet with rise velocity V_r will rise to the surface during its passage through the tank. The required rise velocity is 0.62m/hr. as discussed earlier. The factor F (dimensionless) accounts for short-circuiting and turbulence effects which degrade the performance of the tank. The factor depends on the ratio of horizontal velocity (U) to rise velocity (V_r) as shown in Table 7-17.

The volume and area determined from this tank sizing refer to the dimensions of the main compartment of the tank. Additional volume should be allowed for inlet and outlet sections in the tank.

Other sizing details:

- $U \leq 15\ V_r$
- $0.3\ W \leq d \leq 0.5\ W$ (typically d = 0.5W)
- 1.5 m < W < 5 m
- 0.75 m < d < 2.5 m

Where:
d is the depth (m), and
W is the width of the tank (m).

Some of these dimensions will not be appropriate for smaller catchments, and may be relaxed. It is necessary, however, to keep the length at least twice the width, the depth at least 0.75 m and $U < 15V_r$ at the design flow.

To avoid re-entrainment of oil and degradation of performance, it is required that the maximum horizontal flow velocity in the main part of the tank be less than 25 m/hr.

7.4.10.4 Case study

A section of highway in Napier that has a history of high oil and grease discharge is being retrofitted to reduce downstream discharge of them. The area to be retrofitted has a catchment area of 300 m^2 draining to the device.

1. The separator design flow is the flow from 21 mm/hr of rain, which, from the equation provided earlier, is.

$$\begin{aligned} Q_d &= CiA \\ &= (0.9)(300)(0.021) \\ &= 5.67\ m^3/hr \end{aligned}$$

2. The separator will be sized for a rise velocity of 0.62 m/hr. First an API will be

considered. The maximum design flow velocity (U) for separation at the separator design flow is 15 V_r = 15 (0.62 m/hr) = 9.3 m/hr. Therefore the flow cross section (depth times the width) is $Q_d/U = 0.61\ m^2$. The depth is chosen to be half the width, which gives a depth of 0.55 m and a width of 1.1 m.

This depth is smaller than recommended, so a depth of 0.75 m (the minimum recommended depth) and width of 1.5 m (twice the depth) is chosen, giving $U = Q_d/A = 5\ m/hr = 5V_r$ at the design flow. An F of 1.33 (from Table 10-3) is then used to calculate A_d, giving:

$$A_d = (FQ_d)/V_r = (1.33)(5.67)/0.62$$
$$= 12.2\ m^2$$

With this plan area and the width of 1.5 m, the length is 8.1 m. The volume of the main chamber of the tank will be 9.15 m^3 (excluding inlets and outlets). The tank will actually be longer to allow for an inlet chamber and an outlet section, which, as an approximate guide, could add an additional 20% to the total tank volume.

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8 Landscaping

8.1 Introduction

Landscaping is critical to improving both the function and appearance of stormwater management practices. It has aesthetic, ecological and economic value that is often not recognised during site design and construction. In almost all cases, compliance with regulatory requirements is the key driver and the issue of how a stormwater practice fits into the local landscape can be overlooked.

Example of a Very Unattractive Stormwater Management Pond. This would Deflate adjacent Property Values

Moreover, where the initial developer is not the eventual property owner, there may not be a long-term interest in landscaping.

Where the local territorial authority assumes the maintenance responsibility for the practice and/or becomes the owner of the practice, landscaping issues must become a standard asset management cost in the Council's financial plans.

If the practice is considered an eyesore, property values will go down and the general public response to stormwater management will be negative. The stormwater practice must be an integral part of the development and given the same landscape attention as other parts of the site.

8.2 Objective

The objectives of landscaping stormwater management practices are to:

- Improve their aesthetics,
- Improve their water quality and ecological function, and
- Increase the economic value of the site.

A good landscape plan will consider all three objectives. This means involving a professional landscape architect with experience in natural system design.

Considerations include:

- Site soils,
- Slopes,
- Hydrologic conditions, and
- Water quality/ecological benefits.

The following discussion expands on the three objectives.

8.2.1 Improve the aesthetic appeal of stormwater practices

Aesthetics is a subjective yet very important aspect of everyday life. It is a concept that is difficult to define quantitatively. Something that is good aesthetically tends to be considered tasteful, pleasing, appropriate and fitting for its location. Tastes differ, and disagreement about what is aesthetic is common. The goal of this section is to ensure that stormwater practices are designed as an asset to the property owner and to the overall community.

8.2.2 Improve the water quality and ecological function of the practices

Attention to landscaping as a component of a stormwater management practice can have a significant positive effect on water quality and ecological function. Shading of practices can reduce thermal impacts on receiving systems. Vegetated buffer zones (woody or grassed) can reduce sediment entry, and natural vegetation promotes local ecological diversity.

Landscaping plans should consider:

- Chemical use reduction
- Contaminant source reduction
- Impervious surface mitigation.

Projects should be designed to minimise the need for toxic or potentially contaminating materials such as herbicides, pesticides, and fertilisers within the stormwater management practice area.

Materials that could leach contaminants or pose a hazard to people or wildlife should not be used as components of a stormwater practice (examples can include chemically treated wood or galvanised metals).

Good landscaping can also reduce impacts of impervious surfaces by incorporating swales by paths and access ways.

8.2.3 Increase the economic value of the site

A number of studies demonstrate the economic benefits of properly landscaped stormwater systems:

- Study in Maryland in the U.S. found that properly designed stormwater management ponds increased adjacent property values by 10 - 15 %,
- The U.S. EPA's literature review of the impacts of urban runoff ponds on property values is available on EPA's website at www.epa.gov/OWOW/NPS/runoff.html, and
- City of Christchurch has been engaged in natural stream restoration and has identified significant monetary benefit to property values for properties abutting the restored stream channels.

8.3 Use of native species

This stormwater management guideline encourages the use of native plants in stormwater management practices, where they are appropriate. Native plants are defined as those species found in the Hawke's Bay Region before European migration.

Native species have distinct genetic advantages over non-native species for planting. As they have evolved here naturally, indigenous plants are best suited for our local climate. This translates into greater survivorship when planted and less replacement and maintenance during the life of a stormwater management practice. Both of these attributes provide cost savings for the practice owner.

People often plant exotic species for their ornamental value. While it is important to have aesthetic stormwater management practices for public acceptance and the maintenance of property value, it is not necessary to introduce foreign species for this purpose. There are a number of native species that are aesthetically pleasing and can be used as ornaments.

8.4 General landscape guidance for all stormwater practices

There are several components of a landscape plan. They should be considered individually and together to ensure implementation of a successful landscape plan. The components include the following:

- Stormwater practice area,
- Landscape screening,
- Soils,
- Site preparation,
- Planting, and
- General guidance.

An Attractive Practice can be an Amenity to an Adjacent Community

8.4.1 Stormwater practice area

The practice area includes the stormwater management practice itself, maintenance access ways, fencing and a minimum buffer around these elements. The buffer ensures that adequate space is available for landscaping. Other site elements can be located within the buffer if the need arises. The landscape plan should designate the practice and buffer area.

8.4.2 Landscape screening

Practice elements such as chain link fences, concrete headwalls, outfall pipes, riprap, gabions, steel grates, steep side slopes, manhole covers, and so on. These elements can be screened from general public view with plant materials. Landscape screens of shrubs and trees could have a significant beneficial effect on public perception if used effectively.

8.4.3 Soils

It is necessary to test the soil in which you are about to plant in order to determine the following:

- pH,
- Major soil nutrients,
- Minerals, and
- Seasonal wetness and water-retention capacity

The soil samples should be analysed by a qualified professional who will explain the results and their implications for plant selection.

8.4.4 Site preparation

Construction areas are often compacted, so that seeds wash off the soil and roots cannot penetrate it. No material storage or heavy equipment should be allowed in the stormwater practice or buffer area after site clearing has been completed, except to excavate and grade the stormwater management area. All construction and other debris must be removed before topsoil is placed.

For planting success, soils should be loosened to a depth of approximately 150mm. Hard clay soils will require disking to a deeper depth. The soil should be loosened regardless of the ground cover. This will improve seed contact with the soil, increase germination rates and allow the roots to penetrate the soil.

Providing good growing conditions can prevent poor vegetative cover. This saves money, as vegetation will not need to be replanted.

8.4.5 Planting

In selecting plants, consider their desired function in the landscape. Is the plant needed as ground cover, soil stabiliser or a source of shade? Will the plant be placed to frame a view, create a focus or provide an accent? Does the adjacent use provide conflicts or potential problems and require a barrier, screen, or buffer? Nearly every plant and plant location should be provided to serve some function in addition to any aesthetic appeal.

A Well Landscaped Rain Garden

Certain plant characteristics are obvious but may be overlooked in the plant selection, especially:

- Size, and
- Shape.

Tree limbs, after several years, can affect power lines. A wide growing shrub may block an important line of sight to oncoming vehicular traffic. A small tree, when full grown, could block views. Consider how these characteristics can work today and in the future.

It is critical that selected plant materials are appropriate for soil, hydrological conditions and other practice and site conditions. More information on adequacy of specific plant species is provided in the individual practice chapters.

8.4.6 General guidance

- Trees, shrubs, and any type of woody vegetation are not allowed on a dam embankment.
- Check water tolerances of existing plant materials prior to inundation of area.
- Stabilise aquatic and safety benches with emergent wetland plants and wet seed mixes.
- Do not block maintenance access to structures with trees or shrubs
- To reduce thermal warming, shade inflow and outflow channels as well as northern exposures of ponds.
- Shading of standing water reduces undesirable algae blooms
- Avoid plantings that will require routine or intensive chemical applications.
- Test the soil to determine if there is a need for amendments
- Use low maintenance ground cover to absorb stormwater runoff
- Plant stream and water buffers with trees and shrubs where possible to stabilise banks and provide shade
- Maintain and frame desirable views. Take care not to block views at road intersections or property entrances. Screen unattractive views into the site.
- Use plants to prohibit pedestrian access to ponds or steeper slopes.
- Consider the long-term vegetation management strategy of the stormwater practice, keeping in mind the maintenance obligations of the eventual owners.
- Preserve existing bush areas to the extent possible.

8.5 Specific landscape provisions for individual stormwater management practices

In addition to the general guidance presented above, more specific guidance is given below for individual stormwater practices (this guidance is subject to variation from site to site).

8.5.1 Ponds and wetlands

Chapter 7 provides design guidance for ponds and wetlands. Ponds and wetlands have several defined elements that affect landscaping, including:

- Pond shape,
- Pond topography, and

- Zones of water inundation and periodic saturation.

8.5.1.1 Pond shape

A Well Landscaped Pond

Pond or wetland shape strongly influences public reaction. A rectangular pond is not seen as a 'natural' site feature and offers little in terms of amenity value. A pond with an irregular shoreline or one that apparently fits in with natural contours is more attractive. In addition, an irregular shape has a longer edge than a rectangular pond and allows for more planting, both above and below the water line. The Hawke's Bay Regional Council strongly recommends an irregular shoreline or one that follows existing contours. A minimum recommended buffer area around the pond is five metres above the shoreline where a reverse safety bench, as detailed in Chapter 7, and plantings can be established.

8.5.1.2 Pond topography

Topography has a major effect on the range of plants that can be grown, the movement of water through the pond or wetland and public safety. Steep side slopes can be dangerous for people slipping into a pond and will affect the types of plants that can be used.

The Hawke's Bay Regional Council recommends a 300 mm deep three metre wide level bench below the normal pool level. This is recommended for safety reasons and for growth of emergent wetland plants. The plants will act to restrict public access to deeper water.

Islands, effectively placed, can also be used for multiple benefits. They can increase stormwater flow paths, provide additional landscaped areas and provide wildlife habitat. Islands also increase edge lengths and vegetated areas.

8.5.1.3 Zones of water inundation and periodic saturation

Normal pond and wetland function will result in a number of zones becoming established, each providing different landscaping opportunities.

Zone 1 Periodic flooding zone

Sometimes flooded, but usually above the normal water level
This zone is inundated by floodwaters that quickly recede in a day or less. Key landscaping objectives may be to stabilise steep slopes and establish low maintenance natural vegetation.

Zone 2 Bog zone
Apart from periods in the summer, the soil is saturated

This encompasses the pond or wetland shoreline. The zone includes the safety bench and may also be periodically inundated if storm events are subject to extended detention. Plants may be difficult to establish in this zone, as they must be able to withstand inundation of water during storms or occasional drought during the summer. These plants assist in shoreline stabilisation and shading the shoreline, contaminant uptake and limiting human access. They also have low maintenance requirements.

Zone 3 0 - 150 mm deep of normal pool depth

This is a transition zone between the bog zone and the 150 - 500 mm ponded depth in which the water level sometimes drops and the area becomes a bog. Plants in this area must be able to tolerate periodic (but not permanent) saturated soil conditions.

A Well Vegetated Wetland Pond

Zone 4 150 - 500 mm deep

This is the main zone where wetland plants will grow in stormwater ponds and wetlands. Plants must be able to withstand constant inundation of water and enhance contaminant uptake.

Plants will stabilise the bottom and edge of the pond, absorbing wave impacts and reducing erosion. They will slow water velocities and increase sediment deposition rates along with reducing re-suspension of sediments.

Zone 5 500 - more than 1000 mm deep

This zone is not generally used for planting because there are not many plants that can survive and grow in this zone.

8.5.1.4 Infiltration and filter practices

Infiltration and filter practices either take advantage of existing permeable soils or create a permeable medium such as sand. When properly planted, vegetation will thrive and enhance the functioning of the practices. For example, pre-treatment buffers will trap sediments. Successful plantings provide aesthetic value and wildlife habitat, making the facilities more acceptable to the general public.

Planting around infiltration or rain garden practices for a 5 - 10 metre distance will cause sediments to settle out before entering the practice, thus reducing the frequency of maintenance clean out. As a planting consideration, areas where soil saturation may occur should be determined so that appropriate plants may be selected. Shrubs or trees must not be planted in areas where maintenance access is needed.

8.5.1.5 Swales and filter strips

Key considerations include:

- Soil characteristics,
- Plant interaction,
- Effects on stormwater treatment, and
- Riparian buffers

An Attractive Swale in a Residential Neighbourhood

The characteristics of the soil are perhaps as important as practice location, size, and treatment volume. The soil must be able to promote and sustain a robust vegetative cover.

Plant interaction is also important. Planting woody vegetation next to a swale or filter strip may shade the swale intolerant grass species in it.

The landscape plan will have to consider the effects that overall landscaping will have on stormwater treatment.

Riparian Buffer as an Attractive Amenity to the Stream

Riparian buffers are an excellent example of filter strips with high ecological, water quality and aesthetic value. When appropriately designed, they can treat dispersed runoff from adjacent land. The buffer, as shown in the adjacent picture, can be an amenity to the community and increase economic value of adjacent lands.

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9 Outlet Design

9.1 Introduction

Erosion at pipe or channel outlets is common. Determination of the flow condition, scour potential and channel erodibility should be a standard component of stormwater management design. The only safe procedure is to design the outfall on the basis that erosion at the outlet and downstream channel is to be expected. A reasonable procedure is to provide at least minimum protection, and then inspect the outlet channel after major storms to determine if the protection must be increased or extended. Under this approach, the initial protection against channel erosion should be sufficient to provide some assurance that extensive damage would not result from one runoff event.

Two types of erosion result from stormwater discharges:

- Local scour in the vicinity of pipe or channel outfall
- General channel degradation further downstream

Local scour is the result of high velocity flow at the pipe outlet. It tends to have an effect for a limited distance downstream. Natural channel velocities are almost universally less than pipe outlet velocities, because the channel cross section, including the floodplain, is generally larger than the pipe flow area while the frictional resistance of a natural channel is less than the frictional resistance of a concrete pipe. Thus, flow eventually adjusts to a pattern controlled by the channel characteristics.

Channel degradation represents a long term lowering of the stream channel, which may proceed in a fairly uniform manner over a long length or may be evident in one or more abrupt drops. A number of stream channels in the Region are degrading as a result of increased stormwater runoff volumes from changed land use, initially from forest to rural use and further from rural to urban use. Waterway instability issues, which stormwater systems discharge in to, is an essential part of overall stormwater management design.

Outlet protection for culverts, stormwater outfalls or ditches is essential to prevent erosion from damaging downstream channels and receiving environments. Outlet protection can be a channel lining, structure or flow barrier designed to lower excessive flow velocities from pipes and culverts, prevent scour, and dissipate energy. Good outlet protection will significantly reduce erosion and sedimentation by reducing flow velocities.

9.2 Objective

Outlet protection aims to protect outfall areas from local scour. It is necessary whenever discharge velocities and energies at the outlets of pipes or ditches are sufficient to erode the downstream reach.

When an outfall is sited in a coastal environment, it is essential to also consider wave energy in determining appropriate rock sizing.

9.3 Design approach

Key design elements include:

- Pipe grade
- Outlet velocity
- Riprap aprons
- Engineered energy dissipaters
- Flow alignment and outfall setback in freshwater receiving environments
- Erosion control in coastal environments

These are summarised below.

9.3.1 Pipe grade

To minimise the complexity of analysis and design of outlet protection structures, the first step to look for was to reduce the need for outlet protection by laying the pipe at as low a grade as possible, for example by using a drop structure in the pipe a short distance above the outfall.

9.3.2 Outlet velocity

In order to identify the need for further outlet protection, it is useful to compare outfall velocities with the velocities that natural channels can tolerate without accelerated erosion, as shown in Table 9-1.

The design and analysis of riprap protection, stilling basins, and other types of outlet structures can be a complex task to accomplish. The first step is to look for ways to reduce the need for outlet protection by laying the pipe at a grade no steeper than possible (possibly using a drop structure in pipe). When considering outfall velocities, there is value in considering what velocities that natural channels can tolerate prior to eroding. Table 9-1 (Fortier and Scobey,1926) provides those values.

The primary consideration in selecting the type of outlet protection is the outlet velocity for pipes or channels, which is dependent on the flow profile associated with the design storm.

Pipe flow may be controlled by:

- The type of inlet
- The throat section
- The pipe capacity or
- The type of outlet.

The type of control may change from outlet control to inlet control depending on the flow value.

For inlet control, the outlet velocity is assumed to be normal depth as calculated by Manning's equation.

For outlet control, the outlet velocity is found by calculating the channel flow from Manning's equation with the calculated tailwater depth or the critical flow depth of pipe, whichever is greater.

Table 9-1
Maximum permissible velocities for unlined channels

Material	Mean Velocity (m/sec)
Fine Sand, colloidal	0.4
Sandy loam, noncolloidal	0.5
Silt loam, noncolloidal	0.6
Alluvial silts, noncolloidal	0.6
Ordinary firm loam	0.8
Volcanic ash	0.8
Stiff clay, very colloidal	1.1
Alluvial silts, colloidal	1.1
Shales and hardpans	1.8
Fine gravel	0.8
Graded loam to cobbles, noncolloidal	1.1
Graded silts to cobbles, colloidal	1.2
Coarse gravel, noncolloidal	1.2
Cobbles and shingles	1.5

9.3.3 Riprap aprons

Outlet protection can take the form of riprap placement with the stone sizing being done as part of the storm drainage design, and using these guidelines. Riprap outlet protection is usually less expensive and easier to install than concrete aprons or energy dissipaters. A riprap channel lining is flexible and adjusts to settlement; it also serves to trap sediment and reduce flow velocities.

Riprap aprons should not be used to change the direction of outlet flow: an impact energy dissipaters is more appropriate for this. Riprap aprons aim to manage the transition of piped stormwater into a stream channel primarily by their higher Manning's roughness coefficient, which slows the water velocity.

Riprap aprons should be constructed, where possible, at zero percent grades for the specified length.

Grouted riprap may be subject to upheaval from periodic saturation of clay subgrades and is therefore not generally recommended for outlet velocity protection. Upheaval can crack the grout resulting in undersized riprap size for the velocities of flow. In general ungrouted, properly sized riprap provides better assurance of long-term performance.

Laying riprap directly on soils can allow the water to hit soil particles, dislodging them and causing erosion. Filter cloth laid between the soil and riprap will assist this. Filter cloth is graded on the thickness and permeability characteristics. A qualitative judgement is usually made on the appropriate grade to prevent erosion and prevent puncture by riprap.

9.3.4 Engineered energy dissipaters

There are many other types of energy dissipaters. An older document is the Culvert Manual, Volume 1 done by the Ministry of Works and Development in August, 1978. There have been many types developed over the years. Commonly used varieties include stilling basins, baffle blocks within a headwall and impact energy dissipaters.

Engineered energy dissipaters including stilling basins, drop pools, hydraulic jump basins or baffled aprons are required for outfalls with design velocities more than 6 metres per second. These should be designed using published or commonly known techniques found in such references as *Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC 14, September 1983, Metric Version*. This design approach can be downloaded from the Internet at www.fhwa.dot.gov/bridge/hydpub.htm.

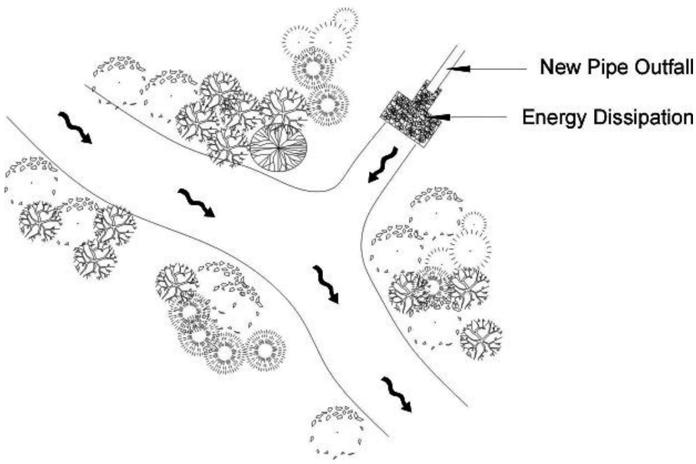
9.3.5 Flow alignment and outfall setback in freshwater receiving environments

Depending on the location and alignment of the pipe outfall and the receiving stream, outfall structures can have a significant effect on receiving channels. Alignment at a right angle to the stream will force the flow to make a 90° angle to the direction of flow. This can cause scour of the opposite stream bank in as well as causing significant turbulence at the point of entry.

The preferred approach is to align the pipe flow at no more than a 45° angle to the stream. Figure 9-1 shows an angled outfall entry to a stream.

If the pipe outfall must be directly into the stream channel, riprap must be placed on the opposite stream channel boundary to a depth of 300 mm above the elevation of the pipe crown. This is in addition to a riprap apron at the pipe outfall.

Figure 9-1
Angled Entry of Outfall into a Stream



The impact of new pipe outfalls can be significantly reduced on receiving streams by locating them further back from the stream edge and digging a channel from the outfall to the stream. This would allow for energy dissipation before flows enter the stream, as shown in Figure 9-1. At a minimum, the pipe outfall should be located far enough back from the stream edge to prevent the energy dissipater intruding on the channel.

9.3.6 Erosion control in coastal receiving environments

Discharges and outlet structures may give rise to a number of adverse effects on the coastal environment if they are constructed of inappropriate materials and/or are poorly sited. For example, a discharge may cause or exacerbate erosion of a beach or an outlet may detract from the natural character or amenity value of the coastal environment or impede public access to, from and along the coast.

Before locating a discharge in the coastal marine area particular consideration should be given to the following matters to avoid/minimise any adverse effect on the natural character, amenity or public access values of the coastal environment:

1. Discharging in such a location that will not unnecessarily cause or exacerbate erosion, particularly of beach materials. For a discharge to a beach, this may involve locating the point of discharge away from the active beach system, e.g. at or near an adjacent headland.
2. Where there are more than one points of discharge to a beach system, consideration should be given to combining discharges to a common point of discharge, including via a common structure.
3. Ensuring the visual form and appearance of the outlet does not detract from its immediate surrounds and the natural character of the coastal environment, e.g. ensuring the structure is assumed into its locality rather than contrasts with that environment. The use of locally sourced rock and/or coloured and sculpted concrete forms may be appropriate.
4. Keeping the “footprint” of the structure to a minimum.
5. Incorporating the discharge pipe into another structure, e.g. a boat ramp, to minimise the number of structures in the coastal environment.
6. Locating the outlet and discharge in such a position as to not create an obstacle to public access to, from or along the coastal marine area.

9.4 Detailed design

The design of outlet protection can be done in two ways. The most accurate approach is that in *Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC 14, July 2006, Metric Version*. This is widely used by design professionals and is recommended by the Hawke’s Bay Regional Council.

The second approach is a simplified approach, which is conservative in order to ensure that adequate channel protection is provided. The approach still requires that velocities for the design discharge to be calculated and inputted into the equations. The design approach based on Figure 9-2 is:

1. Determine the discharge velocity for the design storm. For stormwater management structures the design storm is the maximum flow that can be carried by the pipe. This will normally be the 10-year design flow.

2. Enter that value into the following equation to determine the equivalent diameter of the stone.

   $d_s = 0.25 \times D_o \times F_o$

   where

   d_s = riprap diameter (m)

D_o = pipe diameter (m)
F_o = Froude number = $V/(g \times d_p)^{0.5}$
d_p = depth of flow in pipe (m)
V = velocity of flow in pipe (m/s)

3. The thickness of the stone layer is 2 times the stone dimension. $D_A = 2d_s$
4. The width of the area protected is 3 times the diameter of the pipe. $W_A = 3D_o$
5. The height of the stone is the crown of the pipe + 300 mm.
6. The length of the outfall protection is determined by the following formula.

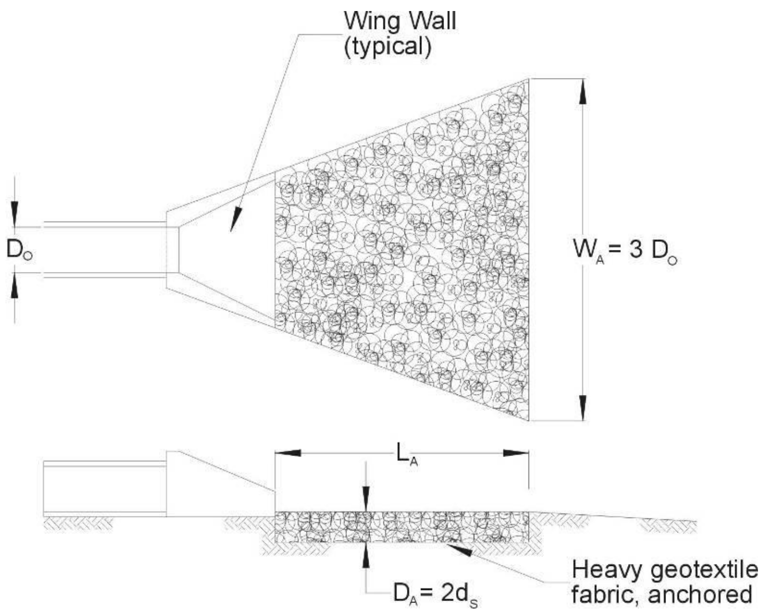
$$L_a = D_o(8 + 17 \times \text{Log } F_o)$$

Where

L_a = Apron length (m)
g = 9.8 m/sec^2

As can be seen from the equations, any reduction in the discharge velocity will reduce the stone size and apron length. Mechanisms to reduce velocity prior to discharge from the outfall are encouraged, such as drop manholes, rapid expansion into pipes of much larger size, or well up discharge designs.

Figure 9-2
Schematic of Riprap Outfall Protection



9.5 Construction

Construction of the outfall protection must be done at the same time as construction of the pipe outfall itself. In terms of environmental protection and timing of construction, it is best to construct the outfall unit from the bottom up, to prevent concentrated flows from being discharged into an unstabilised location. If construction of the outfall system is done from the top end first, the entrance to the system should be blocked off to prevent flow from travelling through the pipe until the outfall protection is completed.

Outfall structures associated with stormwater management ponds shall be done in a similar fashion. Once the embankment has been completed and the pipe outfall structure installed, the outfall erosion protection must be constructed.

It is important that a sequence of construction be established and followed, such as, for example:

1. Clear the foundation area of trees, stumps, roots, grass, loose rock, or other unsuitable material.

2. Excavate the cross-section to the lines and grades as shown on the design plans. Backfill over-excavated areas with moist soil compacted to the density of the surrounding material.
3. Ensure there are no abrupt deviations from the design grade or horizontal alignment.
4. Place filter cloth and riprap to line and grade and in the manner specified. Sections of fabric should overlap at least 300 mm and extend 300 mm beyond the rock. Secure the filter cloth at the edges via secure pins or a key trench.
5. Ensure the construction operations are done so as to minimise erosion or water contamination, with all disturbed areas vegetated or otherwise protected against soil erosion.
6. For coastal sites, undertake construction at periods of low tide.

9.6 Operation and maintenance

Key tasks are:

- Inspect outlet protection on a regular basis for erosion, sedimentation, scour or undercutting
- Repair or replace riprap, geotextile or concrete structures as necessary to handle design flows
- Remove trash, debris, grass, or sediment

Maintenance may be more extensive as smaller riprap sizes are used, as children may be tempted to throw or otherwise displace stones or rocks.

9.7 Bibliography

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U.S. Army Corps of Engineers, Hydraulic Design of Energy Dissipators for Culverts and Channels, HEC 14, September, 1983, *Metric Version*.

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10 Innovative Practices

10.1 Introduction

As the stormwater programme continues to mature, alternative technologies will be proposed to meet water quality design goals. These innovative practices may be developed where site or catchment development intensity make it difficult to achieve desired water quality treatment levels with conventional systems, or provide a level of treatment that is not possible with conventional approaches.

The Hawke's Bay Regional Council through the consent process encourages the development of innovative, cost-effective stormwater management technologies, subject to approval. Approval will depend on submission of objective, verifiable data that supports the claimed efficiency, although a single pilot site may be approved for purposes of data collection to document performance.

Innovative practices tend to be new technologies that have not been evaluated using approved protocols, but for which preliminary data indicate that they may provide a desirable level of stormwater contaminant control. Some innovative practices have already been installed or are proposed in the Region as parts of treatment trains or as a stand-alone practice for a specific project. In some cases, innovative practices may be necessary to remove metals or hydrocarbons. Innovative practices can also be used for retrofits and where land availability does not permit larger conventional practices.

10.2 Objective

This chapter outlines the information that should be submitted to evaluate the performance of alternative technologies whose operating parameters have not yet been verified to the satisfaction of the Hawke's Bay Regional Council.

This chapter deals with stand-alone and pre-treatment / retrofit practices.

10.2.1 Stand alone practice

An innovative practice should not be used for new development sites unless there are data indicating that its performance is expected to be reasonably equivalent to that provided by conventional practices, or as part of a treatment train. In retrofit situations, the use of any practices that make substantial progress toward the specified environmental objectives is encouraged.

Any alternative stand-alone practice must generally comply with the Hawke's Bay Regional Council water quality recommendations.

Specific contaminant issues may warrant use of an alternative system that may be less effective at TSS reduction while providing enhanced reduction in other contaminants such as hydrocarbons. Performance at specific contaminant reduction will be monitored appropriately.

Water quantity issues may also affect practice acceptance, depending on location in a catchment.

10.2.2 Pre-treatment or retrofit

Individual practices that are not capable of providing desired water quality treatment may nevertheless play a useful pre-treatment supplementary role together with other approved stand-alone practices.

A practice proposed for pre-treatment of flows into another practice may, for example:

- Remove coarse sediments, in order to reduce the frequency of maintenance of the primary stormwater treatment practice,
- Provide water quantity control, and
- Reduce stream erosion.

Retrofit of a site or catchment for water quality treatment depends on land availability, specific contaminants of concern and cost. Water quality goals must be tempered by what can realistically be accomplished in a catchment. It is in these situations where innovative practices have a potentially significant role to play.

10.3 General information required from an applicant for approval of innovative systems

Innovative systems are being introduced on a routine basis. Current ones include:

- Storm drain inserts,
- Underground vaults,
- Hydrodynamic structures, and
- On-line storage in the storm drain network.

This subsection summarisers the basic information that should be submitted with any request for approval in a specific application in order to promote consistency in the submission of information for approval of an innovative practice. Consistency provides surety for a product manufacturer, a consent applicant and the general public that implementation of an innovative practice is based on the best information available. The ultimate goal is clean water and implementation should be based on an estimation of the best practice being used in a given situation.

It is important to be cautious with using innovative technologies for new development and retrofits. Before selecting an innovative practice for a limited application, available information should be evaluated using an acceptable protocol.

For these reasons, submission of an innovative practice in a given situation or for general compliance should include a description of the innovative technology or product including:

- Whether the operating parameters of the system have been verified.
- Existing or proposed monitoring data (detailed in Section 10.4),
- Documentation of processes by which TSS and other contaminants will be reduced (physical, chemical, biological),
- Documentation and/or discussion of potential causes of poor performance or failure of the practice,
- Key design specifications or considerations,
- Specific installation requirements,

- Specific maintenance requirements,
- Data to support the claimed TSS removal efficiency. If the technology is new or the existing data is not considered reliable, a detailed monitoring programme to assess the TSS removal may be required, and
- Ownership issues that could influence use of innovative practices on individual sites. Examples of this issue could be refusal of a TA to accept responsibility for operation and maintenance.

10.4 Information needed to judge adequacy of existing or proposed monitoring data

The following summaries the detailed information that is needed to properly judge the adequacy of existing or proposed monitoring data to evaluate performance compliance of an innovative practice, from catchment related information, practice related information and water quality information.

10.4.1 **Catchment parameters**

The context in which the practice helps define situations where an innovative practice is (or is not) appropriate by assessing collection sites for known or new data. This in turn helps to determine the data's applicability to other locations.

It is also important that monitoring be done in the field, as opposed to the laboratory, as field monitoring better reflects actual practice performance.

Key catchment parameters include:

- Catchment area served,
- % impervious area,
- Total impervious area,
- Hydraulic connectivity,
- Baseflow or storm generated runoff only, and
- Catchment land use and expected contaminants

10.4.2 **Practice design parameters (where applicable)**

Detailing specific elements of the innovative practice provides a clear understanding of the water quality treatment processes that occur in the various components of the practice. If the practice has a standard design based on catchment size or maximum flow rate, that information should be clearly stated in the discussion of practice parameters as detailed in the general discussion.

Key practice parameters include:

- Basic shape (length/width, volume, importance of local topography),
- Any permanent pool elevation and levels of service,
- Surcharge elevation,
- Forebay characteristics,
- Inlet/outlet locations and relative elevations,
- Water level control options,
- 'On-line' or 'off-line',

- Age of practice where monitoring has been or will be done, and
- Specifications for practice components (filter media, sieve sizes, geotextile specifications, etc.).

10.4.3 Water quality analysis

Analyses detailed here are primarily for those done in New Zealand. Recognising that many innovative practices are being developed overseas, all information may not be available. In those situations a degree of judgement is involved regarding the relative importance of specified criteria. The Hawke's Bay Regional Council will consider the submission of overseas data as full or partial fulfilment of the water quality analyses, depending on the applicability of the collected data to the Hawke's Bay situation. Compliance assurance may necessitate water quality analysis on a more limited basis only for those parameters where gaps exist.

The following analyses are to be done for practice performance documentation:

- Flow weighted composite samples used to determine the TSS concentrations in the influent and effluent of the device,
- General water quality constituents for monitoring include TSS, pH, conductivity, DO, enterococci and total hydrocarbons,
- Total zinc should also be monitored as a 'keystone' contaminant for trace metals,
- The performance of the practice or system should be based on the sampling results from at least 10 storms representative of those normally occurring in the Region. Depending on the relative variation in results, additional monitoring may be necessary to better understand expected performance,
- At least one storm event must be greater than 20 mm of rainfall,
- There must be at least three days of dry weather between storms sampled
- The samples must be collected and handled according to established procedures that are included in the monitoring plan,
- The laboratory selected for analysis of the samples is recognised as technically proficient,
- The efficiency of the device is calculated for individual events and is also based on the total TSS load removed for all monitored events,
- The monitoring must be conducted in the field as opposed to laboratory testing, and
- Depending on the processes involved in treatment, the practice or system may need to be in the ground for at least six months at the time of monitoring.

15.5 Discussion

While the level of information requested may seem onerous to someone developing or wanting to use an innovative practice, it is essential that programme implementation and overall success be underpinned by good technology. With millions of dollars being spent on design, implementation and operation, it is important that environmental objectives are met, especially when considering the costs associated with management.

Ultimate programme success rests on stormwater strategies, approaches and practices achieving a certain level of performance. We must have confidence that a practice will achieve stated goals and a good understanding of practice strength, limitations, and performance if we are to meet our obligations under the RMA and public expectations.

Appendix

Hawke's Bay Regional Council Stormwater Rules

6.6.5 Stormwater - Discharges to Land/Water

Rule	Activity	Classification	Conditions/Standards/Terms	Matters for Control/Discretion	Non-notification
42 **Diversion and discharge of stormwater** *Refer POL 15, 16, 17, 47, 49, 71, 75*	The diversion and discharge of stormwater from any constructed open drainage system or piped stormwater drainage system that: • does not drain any industrial or trade premises, or • Drains any industrial or trade premises covering an area of less than 2 ha, excluding premises used for the storage of any hazardous substance.	**Permitted**	a. The activity shall not cause any permanent: i. reduction of the ability of the receiving channel to convey flood flows. ii. bed scouring or bank erosion of the receiving channel. b. The discharge shall not cause the production of conspicuous oil or grease films, scums or foams, or floatable or suspended materials in any receiving water body after reasonable mixing.		
42a ***Diversion and discharge of non-point source stormwater***	*The discharge of stormwater from any road, roof, car park, yard or grassed area or any other structure (not including stormwater discharged into or from a reticulated stormwater system)*	***Permitted***	*a. The discharge shall not contain stormwater that originates from any industrial or trade premises where hazardous substances are used or stored, unless the hazardous substances cannot enter the stormwater, nor shall it come from any contaminated site.* *b. The discharge shall not contain sewage, foul water, agricultural effluent, animal waste or by-products.* *c. The discharge shall not result in any of the following in any receiving water after reasonable mixing:* *i. The production of any conspicuous oil or grease film, scum or foams, or floatable or suspended materials.* *ii. Any conspicuous change of colour or visual clarity.*		

Rule	Activity	Classification	Conditions/Standards/Terms	Matters for Control/Discretion	Non-notification
			iii. Any emissions of objectionable odour. iv. Any significant adverse effects on aquatic life. v. The rendering of freshwater unsuitable for consumption by farm animals. d. The discharge does not cause flooding or any other person's property, erosion or land instability.		
42b **Diversion and discharge of reticulated stormwater**	*The diversion and discharge of stormwater from any reticulated stormwater system into a surface water body, artificial watercourse or land that does no enter a Sensitive Receiving Environment as identified in Schedule XII.*	***Permitted***	a. The discharge shall not contain stormwater that originates from any industrial or trade premises where hazardous substances are used or stored, unless the hazardous substances cannot enter the stormwater, nor shall it come from any contaminated site. b. The discharge shall not contain sewage, foul water, agricultural effluent, animal waste or by-products. c. The discharge shall not result in any of the following in any receiving water after reasonable mixing: i. The production of any conspicuous oil or grease film, scum or foams, or floatable or suspended materials. ii. Any conspicuous change of colour or visual clarity. iii. Any emissions of objectionable odour. iv. Any significant adverse effects on aquatic life. v. The rendering of freshwater unsuitable for consumption by farm animals. d. The discharge does not cause flooding or any other person's property, erosion or land instability.		

Rule	Activity	Classification	Conditions/Standards/Terms	Matters for Control/Discretion	Non-notification
43 **Diversion and discharge of stormwater**	*The* ~~d~~*D*iversion and discharge of stormwater*, that is not a permitted activity and does not enter a Sensitive Receiving Environment, as identified in Schedule XII.*~~except as provided by Rule 42.~~	**Controlled**	a. All reasonable measures shall be taken to ensure that the discharge is unlikely to give rise to ~~all or~~any of the following, ~~effects~~in any receiving water, after reasonable mixing: i. The production of any conspicuous oil or grease films, scum's or foams, or floatable or suspended materials. ii. Any conspicuous change in the colour or visual clarity. iii. Any emission of objectionable odour. iv. The rendering of fresh water unsuitable for consumption by farm animals. v. Any significant adverse effects on aquatic life.	a. Location of the point of diversion and discharge including ~~it's~~*the* catchment ~~area~~. b. Volume, rate, timing and duration of the discharge~~, in relation to a specified design rainfall event~~. c. Effects of the activity on ~~downstream~~ flooding*, erosion and land instability*. d. Contingency measures in the event of pipe capacity exceedence. e. Actual or ~~likely~~ potential adverse effects on ~~fisheries, wildlife, habitat or~~ ecological amenity, recreational or cultural values~~ of any surface water body~~. f. Proximity to and effects on any Sensitive Receiving Environment.~~Actual or likely adverse effects on the potability of any ground water.~~ g. *An appropriate mixing zone.*~~Duration of the consent.~~ h. *Actual or potential adverse effects on the potability of ground water.*~~A compliance monitoring programme.~~	Applications may be considered without notification and without the need to obtain the written approval of affected persons in accordance with section 94 (1) (b) of the RMA. Applications may however be notified if special circumstances exist in terms of section 94 (5) of the RMA. In considering whether or not special circumstances exist, the Council will include consideration of: 1. The record of compliance with any previous regulations relating to the activity for which a ~~discharge permit~~ *consent* is sought. 2. ~~The d~~*D*ownstream users of the receiving water body. 3. ~~The e~~*E*xtent of public and tangata whenua interest in the activity and/or its effects.

[1] [1] **Non-compliance with rules –** If any of the rules in this section cannot be complied with, then the activity is a discretionary activity un Rule 52

Rule	Activity	Classification	Conditions/Standards/Terms	Matters for Control/Discretion	Non-notification
				i. *The development of a site management plan or catchment management plan for managing stormwater discharges.*~~A bond~~. ~~j.~~ *Duration of the consent.*~~Administrative charges.~~ k. *A compliance monitoring programme.* l. *A bond.* m. *Administrative charges.*[1]	4. *Scale of the activity.*