Hydrological and Hydraulic Guidelines

Prepared by Environmental Hazards Group

Bay of Plenty Regional Council
Guideline 2012/02

5 Quay Street
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Whakatane
NEW ZEALAND

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Bay of Plenty
REGIONAL COUNCIL

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Contents

Part 1: Introduction

1.1 Objective of guidelines

This document is a guideline in so much as it provides general procedures and principles to be applied with reference to existing textbooks, manuals and design guides provided for detailed information.

The objective of this document is to provide design guidelines for activities that require hydrological, hydraulic and/or general civil engineering calculations or assessments.

Such activities occur in surface waters and generally result in a modified waterway and typically include constructions such as:

- culverts
- bridges
- services crossing a watercourse
- impermeable surfaces (roads, car parks)
- stream realignment and channelling
- small embankments
- flood detention or soil conservation dams
- infilling of land acting as flood plains
- stormwater systems
- erosion controls (lined channels, drop structures)

The document is for use by anyone carrying out any of the above constructions.

These guidelines only cover design matters and are not intended to cover the broader issues relating to land development such as sustainability, matters of national importance, preservation of natural character, assessment of alternatives and consultation. These other matters are dealt with through regional planning and consent processes. It is important to also carry out an ecological, cultural and perhaps historic evaluation of the site where necessary to determine whether any proposed works are acceptable and whether there are any alternatives. During the design stage of any modification, comments should be sought from Regional Council staff as to whether a resource consent is necessary.

The intent of the guidelines is to cover the majority of situations. However, it is recognised that there will be exceptions requiring specialist analysis. Similarly, in some situations other solutions and procedures to those presented may be more appropriate. The guideline does **not** cover the specialist areas of:

- Coastal protection works design. Council has published a separate guideline for this purpose which identifies criteria and standards for the design of coastal erosion protection works in the Tauranga Harbour (2002).

- Stormwater quality and stormwater quantity design. Council has published a separate guideline for permanent stormwater management for land development areas (2012). Also Tauranga City Council has published a separate guideline covering aspects of urban stormwater design applicable to Tauranga City (2012).
- Erosion and sediment control for land disturbing activities. Council has published a separate guideline for erosion and sediment control related to earthworks operations (including quarries and in and around watercourses) (2010).
- Erosion and sediment control for forestry operations. Council has published a separate guideline for erosion and sediment control related to planning and implementation of forestry operations (2000).

In terms of financial and environmental costs, there is a difference between spending more money on an activity up front (capital) as against more money over time ('whole of life' costs that include design, construction and operational costs). Bay of Plenty Regional Council is aiming at minimising the 'whole of life' costs in order to avoid continually disturbing the environment. However, each situation can be evaluated on a case-by-case basis.

1.2 Issues

The guidelines have been developed to highlight some of the adverse environmental effects that can arise as a result of the above activities and how these might be avoided, remedied or mitigated.

There are five main adverse effects considered in this document:

(a) The discharge of sediment – causing decreasing water quality in streams, lakes, estuaries and harbours; smothering fish habitats and flora and fauna; damaging water pumps; raising stream bed levels and consequently increasing flood risks; causing unsightly debris and sediment deposits.

(b) Change to hydraulic grade - raising of water levels upstream of and adjacent to the modified waterway causing flooding of land, buildings and equipment, danger to human life and creating alternative flow paths.

(c) Erosion – destroying streambed and streambank habitats, flora and fauna; creating unsightly and dangerous holes and cliffs; damaging any modifications to the waterway; creating hazards to recreation.

(d) Barriers to fish passage – especially to upstream migrating species.

(e) Increased flooding and runoff.

All of these effects can be caused in a variety of ways by the incorrect design and construction within the waterway as follows:

(a) Modifications to a waterway can decrease the cross-sectional area of the waterway ("waterway area"). This causes water levels to rise upstream, leading to flooding and alternative flow paths, and velocities to increase, leading to increased amounts of scour and erosion and sediment deposition downstream.

(b) Some modifications (culverts, bridges) create barriers to the natural flow of debris (logs, trees, bushes) in the waterway during floods. This can block the waterway causing upstream flooding or force the water through an unexpected flow path.

(c) The embankments associated with bridge approaches or culvert fills can act as small dams in high flows, retaining considerable volumes or levels of water. Without appropriate design or construction these embankments can collapse leading to further erosion, sediment deposition, habitat destruction, short term flooding and loss of use of the structure.

(d) Some modifications (culverts, weirs, drop structures) create barriers to migrating fish by creating steps, projections, ramps etc. which cannot be scaled or that increase the velocity to impassable levels.

(e) Inappropriate design of impermeable surfaces (carparks, roads, compacted soils) will cause increased runoff leading to flow concentrations.

(f) Infilling of existing flood storage areas will cause flooding in adjacent areas.

1.3 About this document

Part 2 describes the principles of good waterway design.

Part 3 gives an outline of a typical investigation and design process for a waterway modification.

Part 4 gives advice on the standards of design to be applied to various modifications from those associated with state highways and lightly trafficked forestry roads to erosion control structures and small dams including Building Act requirements.

Part 5 describes how the design flow should be calculated using standard hydrological techniques. Guidance regarding stormwater mitigation and climate change is also provided.

Part 6 highlights the importance of knowing what the existing water levels and velocities are and what they will be in the design event after the waterway has been modified. Different modifications are discussed.

Part 7 describes the design details that need to be considered and applied including advice on channel erosion protection.

Part 8 points out that flows in excess of the design flow must be expected and gives advice on how these can be handled cost effectively to reduce the potential for adverse effects.

Part 9 is a list of references referred to in the text.

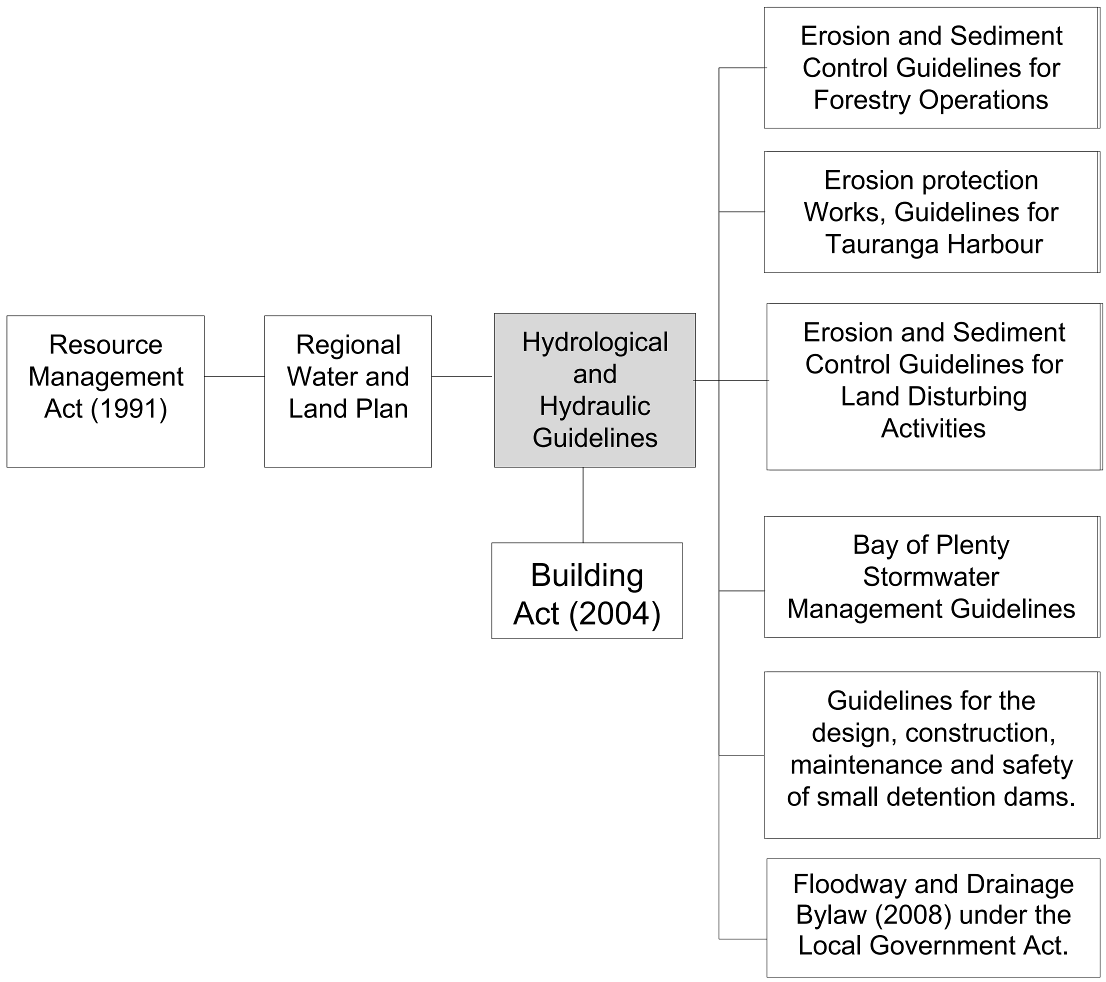
1.4 Role of Bay of Plenty Regional Council

The purpose of the Resource Management Act 1991 (RMA) is to promote sustainable management of natural and physical resources. Under the Act regional councils have the functions of managing the use of land, air, water and coastal resources to give effect to the purpose of the Act within their regions. Bay of Plenty Regional Council does provide a limited advisory service to applicants undertaking activities, whether these require consents or not. However for detailed calculations applicants should employ their own consultant. Over the years, the Bay of Plenty Regional Council has gained knowledge of its region’s hydrology and land characteristics. This knowledge is reflected in these guidelines and is made available for the use of applicants to ensure a consistent approach for activities.

1.5 Relationship with other council plans and guidelines

This Guideline is subordinate to the overarching RMA and Council's Regional Water and Land Plan (RWLP). The RWLP recommends use of these guidelines as it relates to activities undertaken in beds of rivers, streams, and lakes (refer s9.10).

These guidelines are subject to requirements of the Building Act, RMA and other Council guidelines as indicated in the flow chart below.



Part 2: Principles of water design

2.1 Introduction

Waterways in the natural environment or those modified at some time in the past have generally reached a “stable” state of equilibrium, albeit fluctuating from time to time through greater or lesser degrees of stability. This state of equilibrium is very fragile and easily disturbed by both natural events and man- made actions. Any proposal to alter a waterway must recognise this fragility and apply principles of good waterway design.

2.2 Principles of waterway design

Good waterway design should adopt the following principles:

(a) Avoid significant changes being made to waterway area. In particular, reductions to waterway area can lead to increased velocity and corresponding erosion. Velocity reduction (by increasing water way area) will ensure lower energy losses, which directly reduces the extent of any rise in upstream water level caused by changes to the waterway.

(b) Minimise sudden changes in the waterway geometry (shape, slope, direction). Sudden changes induce local turbulence increasing the potential for erosion and scour. Smooth changes in bed slope or channel sides over long distances reduce this effect. Maintaining the existing channel slope is important to maintain the stability of the channel. Maintaining existing channel roughness will also assist in energy dissipation as opposed to a smooth homogenous channel lining which will increase velocities and risk of erosion.

(c) Minimise embankment heights – higher embankments can increase the hazard posed by a failure.

(d) Protect against erosion – modifications to the existing waterway can reduce or remove any natural armouring or vegetation that has established. Where any velocity increase is likely to occur this needs to be accompanied by provision of appropriate erosion control devices e.g. rip-rap, suitable vegetation, energy dissipaters etc.

(e) Reflect the waterway’s natural environment – where a waterway is to be modified. It is desirable to use appropriate materials and designs that are compatible with the surrounding area and reflect its ecological and other values.

(f) Recognise private property rights. It is not appropriate to assume that it is acceptable to cause flooding on another person’s property.

(g) Allow for the needs of the fauna. It is easier to allow for fish pass needs etc. in the original design instead of trying to retrofit a solution.

Part 3: Design process

Once it has been determined that an activity requires modification of a waterway, it is usual for concept design to be advanced. This results in a design and accompanying assessment of effects that is able to be used for consenting purposes, but not able to be used for construction. Following consent approval detailed design usually follows, which involves completion of the design to a level consistent with construction requirements. A typical sequence may be as follows:

Step 1 Decide on the design standard to be applied (refer to Part 4).

Step 2 Site selection. Where choice exists or is necessary, choose a reach of the waterway that is of straight alignment, has a flat slope along the waterway and has firm foundations. Sites that are close to bends, on steep sections or have weak soils will have the potential to erode quickly once modified.

Step 3 Obtain the catchment data (refer to Part 5 – Bay of Plenty Regional Council holds various data). This will include:

- rainfall, flow and water level data.
- catchment size, shape, topography and vegetation.
- ecological values for protection.
- historic water level and anecdotal information.

Step 4 Estimate the design rainfall for the catchment above the site and, where appropriate, factor these values to account for expected climate change effects (refer to Part 5).

Step 5 Estimate the design flow at the site (refer to Part 5).

Step 6 Obtain the site data and estimate or calculate the existing or natural flow conditions at the site:

- site long-sections, plans and cross-sections.
- waterway characteristics (foundation materials, vegetation).

Step 7 Select and/or conceptualise the proposed waterway modification and determine the changed flow conditions (refer to Part 6).

Step 8 Design the modification in detail.

Step 9 The structure should be checked for its performance in flows in excess of the design flow (refer to Part 8).

At all steps of the design process adequate information in accordance with these guidelines is to be collected and documented demonstrating an appropriate understanding of the physical processes involved in the waterway modification.

Generalised designs process flow chart

1 Determine design standard

2 Select site

3 Collect catchment and site data

4 Determine design rainfall

5 Determine design flow

6 Determine the present flow conditions

7 Conceptualise the modification & determine the modified flow conditions

8 Design the details of the modification

9 Check performance with flows exceeding design flow

Part 4: Design standards and practices

4.1 Introduction

The design standard or "level of service" refers to the frequency at which the design may be exceeded. The design standard may also include requirements such as debris clearances above water levels for bridges, maximum heading up for culverts or prevention of flooding upstream. These effects are to be designed for by applying the design standards listed in Section 4.

The waterway modifications to which this chapter generally applies are bridges and culverts under roads. These are discussed in detail. Other modifications such as small embankments, stormwater systems and erosion controls are also considered.

In the following sections, clearance ("freeboard") is a standard engineering provision for estimate imprecision/uncertainty (even the most sophisticated design techniques are unlikely to exactly predict complex hydraulic scenarios) plus phenomenon not explicitly included in the hydraulic calculations e.g. waves, aggradation, bend effects and debris blockage and passage.

The standards set out below can be regarded as minimum acceptable standards without further analysis or justification. They are standards to help avoid, remedy or mitigate the adverse environmental effects described in Section 1 and other regulatory authorities (e.g. Department of Conservation (DOC), NZTA, district councils etc.) may have other higher standards for fish passage, the protection of public safety or for economic reasons. In any case the higher standard should be applied.

4.2 Exceptions

The design standards set out in this chapter are applicable to the whole of the Bay of Plenty Region <u>except</u> in the following circumstances:

(a) Where there is a higher risk of adverse environmental effects in sensitive areas such as the Erosion Hazard Zone (Appendix 1).

(b) Where substantial waterway modification already exists such as in land drainage schemes or flood control schemes.

In these circumstances Bay of Plenty Regional Council should be contacted to determine appropriate design standards.

4.3 Definitions

For waterway modifications associated with stream crossings for roads, the definitions in Table 4.1 are used:

Table 4.1 Definitions of road types

Road type	Definition
Major Road	Either: (a) a state highway, or (b) within 1 km of any urban* area or settlement, or (c) carrying more than 750 vehicles per day.
Rural Road	Any other road except as described below.
Remote Road	Public or private roads accessing property that does not have dwellings and which cross a waterway with a contributing catchment of less than 50 km^2.
Access Tracks	Remote roads as defined above, but with a contributing catchment of less than 100 ha.
**The Council's Regional Water and Land Plan defines Urban area or settlement – an area which contains an aggregation of more than 50 lots or sites of an average size of no more than 1000 m^2 (2008).*	

4.4 Bridges and culverts

The following Table 4.2 shows the recommended design standards for bridges and culverts. Overflow points for storms in excess of the design standard should be defined for each category. Overflow velocities should then be calculated to ensure erosion of the downstream slope does not occur. In all cases it is recommended that overflow should be considered for events of Average Recurrence Interval that exceed the design level of service.

Table 4.2 Design standards for bridges and culverts

Road type	Bridge standard	Culvert standard
Major road	Passage of the 100-year return period flood* with minimum clearance of 0.6 m normally but with up to 1.2 m where large trees can be transported in the river.	• Passage of the 100-year return period flood by heading up to a maximum 0.5m below road and adjacent house floor levels, and • Passage of the 10-year flood without heading up.
Rural road	Passage of the 50-year return period flood with a minimum clearance of 0.6 m.	• Passage of the 50-year return period flood by overtopping the embankment to a maximum depth of 0.2 m, and • Passage of the two year return period flood with no heading up.
Remote road	Passage of the 20-year return period flood with a minimum clearance of 0.3 m.	• Passage of the 20-year return flood with 0.3 m freeboard, and • Passage of the two year return period flood with no heading up.
Access tracks	Passage of the 10-year return period flood with a minimum clearance of 0.3 m.	• Passage of the 10-year return period flood by heading up to a maximum 0.3 m below road level.
**Note design standards for major roads and culverts are based generally on NZ Transport Authority's (NZTA) Bridge Manual Guidelines (2005).*		

The 100-year return period flood is the standard flood event nominated by NZTA for the design of waterways passing under its major road (or highway) bridges or through road culverts. NZTA also specify the same freeboard i.e. 0.6-1.2 m for bridges and 0.5 m for culverts (Section 2, NZTA Bridge Manual, July 2005).

If the heading up condition is considered the design shall ensure embankment stability under flood conditions, and adequate protection to safeguard against piping, and against scour due to increased water velocity (Section 6.3, Integrated Stormwater Management Guidelines for NZ Roading Network, Transfund NZ, 2004).

When designing hydraulic structures such as bridges or roads, designers should take account of climate change effects and the expected life of the structure. For example if a road culvert being designed is expected to have a life span of say 30 years then design flows should be adjusted for climate change effects expected in 30 years' time. Refer also to Section 5.8 on climate change and its impact on hydraulic design.

4.5 Erosion protection and stream control structures

These include culvert inlet and outlet structures, flumes, rock-lined channels, drop structures etc, that are either integral with bridges and culverts or as stand-alone structures. Their design should allow the passage of the 20-year flow without damage, irrespective of the design standards in Chapter 4.4 above. This means a culvert, for instance, designed for the 100-year return period flood need only have its erosion protection designed for the 20-year event. However bridges and culverts designed to the 10-year event should have any integral erosion control structures designed to the 20-year event.

4.6 Sea levels

For all waterway modifications where the site water level is influenced by sea level, the sea levels for design purposes are listed in Table 4.3. All levels in Table 4.3 are static levels at high tide. This level is Council's best estimate of sea level rise to 2100, appropriate datum and makes allowance for storm surge (namely high tide, inverse barometric pressure, wind set-up), wave set up and estuary effects where appropriate. The levels do not include freeboard and wave run-up. Before the release of the next version of the Hydrological and Hydraulic Guidelines further investigations will be conducted into estuary effects, storm surge, and wave run-up. It is recommended that the figures presented in Table 4.3 be used for the present.

The figures presented in Table 4.3 include a predicted sea level rise of 0.49 m (to 2090). For the design of structures with a long life, or high risk or potential retrofit difficulties (or other similar type considerations) then a greater allowance for sea level rise should be considered. Refer to Section 5.8 for further details.

For stormwater pipelines the estimated design life is 70 years. The MfE guidelines (2008) for predicted sea level rise to 2090 (approximately 100-years in future) is up to 0.8 m. Therefore the 70 year component of 0.8m sea level rise is approximately 0.55 m which is 0.25 m less than the 100-year amount. Refer to Note 1 on Table 4.3.

A description of components that make up levels produced in Table 4.3 is provided in Appendix 2.

For modelling purposes the tidal cycle can be separated out as a separate component however timing of the peak tide must coincide with arrival of the peak flood.

Table 4.3 Sea levels at various locations for design purposes (See References for Sources of Data).

Location	**Design Sea Level (L) for Stated Return Period (Moturiki Datum)*[1]**			
	L_2	L_{20}	L_{50}	L_{100}
Tauranga Harbour	1.9	2.2	2.4	2.5
Ōhiwa Harbour	2.0	2.5	3.0	3.2
Other estuaries	1.9	2.3	2.7	2.9
Open Coast – west of Matata	1.7	2.1	2.3	2.5
Open Coast – east of Matata (Incl.)	1.8	2.3	2.6	2.8
	Note[1]: For stormwater pipelines these levels may be further reduced by 0.25 m, reflecting a shorter design life.			

It is common for the same meteorological storm event to cause both storm surge and flooding.

These sea levels should be combined with the floods as shown in Table 4.4 below. Both cases should be considered and the critical case selected. In circumstances where the consequence of exceeding design levels is serious (e.g. synergistic effects where flooding of an effluent treatment plant may result) then consideration should be given to applying more severe combinations. Based on Council experience the same storm event that causes a flood from an onshore system such as a cyclone will likely also cause a surge albeit to different quantum's i.e. catchment storm and storm surges are interdependent events.

Designers of hydraulic structures should confirm whether the structures are influenced by tidal effects and thus subject to the provisions made in Tables 4.3 and 4.4. During catchment storms much focus has been on the potential for peak discharges from the upstream catchment coinciding with high tides. Stormwater systems must be capable of operating under high tides scenarios irrespective of the probability combinations stated in Table 4.4. If a culvert cannot operate due to an outlet water level limitation then provision must be made to convey or accommodate the design inflow until such time that the culvert is again operational. (e.g. provide storage area to accommodate volume while the tide is high).

Table 4.4 Design standard combinations for floods and sea level.

Design Return Period	Case 1	Case 2
100-year	$Q_{100} : L_{20}$	$Q_{20} : L_{100}$
50	$Q_{50} : L_{20}$	$Q_{20} : L_{50}$
20	$Q_{20} : L_2$	$Q_2 : L_{20}$
10	$Q_{10} : L_2$	$Q_2 : L_{10}$

For example, a bridge on a remote road (requiring a 20-year design) must be checked for both the 20-year flow with a two year sea level and the two year flow with the 20-year sea level.

4.7 Combined events

For structures near the confluence of two waterways the flood levels may be influenced by the flow in both waterways. However the return period of the flood events in both waterways may be different. The relative return periods will depend on a number of factors including the relative catchment sizes, locations and hydrological characteristics. Furthermore they will be affected by the storm areal size and location. Therefore combinations of flows in the two waterways should be analysed to determine the critical case. The combinations shown in Table 4.5 below should be investigated. The two values presented are the respective flood magnitudes in the two waterways.

Table 4.5 Design standard combinations for two waterways

Design Return Period	Case 1	Case 2
100-year	$Q_{100} : Q_{20}$	$Q_{20} : Q_{100}$
50	$Q_{50} : Q_{20}$	$Q_{20} : Q_{50}$
20	$Q_{20} : Q_{2}$	$Q_{2} : Q_{20}$
10	$Q_{10} : Q_{2}$	$Q_{2} : Q_{10}$

The above table is a guideline of events that may occur simultaneously. In some cases the combination of events may be different. A full probabilistic analysis could be carried out for very large projects. For smaller projects the following modifications may be appropriate:

(a) Where catchments are small and of roughly equal size, then there is a good case to consider a more severe combination of events. For example, the design 100-year return period storm could well be more accurately assessed with the Q_{100} in one catchment combined with Q_{100} in the other;

(b) Where catchments are different in size by more than one order of magnitude (10 times) then the combinations may be relaxed. For example the design 100-Year return period storm could be more accurately assessed with the Q_{100} in one catchment and the Q_5 or Q_{10} in the other (depending on the relative sizes, land use and times of concentration etc).

Before applying the above modifications advice should be sought from Bay of Plenty Regional Council engineering staff.

4.8 Upstream water levels

Where there is potential for increases in upstream water levels over and above existing, the flooding impacts on land and on existing and proposed buildings should be checked. Property owners should be consulted on the effect of these water levels on their land.

4.8.1 Floor and subdivision platform levels

According to section E1.3.2 of the New Zealand Building Regulations 1992;

"surface water, resulting from an event having a 2% probability of occurring annually (i.e. a 50-year flood event), shall not enter buildings"

The regulation states that this standard only applies to housing, communal residential and communal non-residential buildings.

Similarly for all new subdivisions the platform levels are to be set above the secondary stormwater system flood level. Secondary systems comprise ponding areas and overland flows paths that are used when the capacity of the primary system (generally piped reticulation) is exceeded. In accordance with Section 4.3.5.1 of NZS4404:2010: Land Development and Subdivision Infrastructure Standard the recommended secondary stormwater system flood level shall be based on the climate changed adjusted 100-year return period storm.

Freeboard should be added to the computed top water flood level (same as design flood level). According to NZS4404: 2010 freeboard is defined as:

"a provision for flood level design estimate imprecision, construction tolerances, and natural phenomena (such as waves, debris, aggradations, channel transition, and bend effects) not explicitly included in the calculations"

Section 4.3.5.2 of the same standard states:

"the minimum freeboard height additional to the computed top water flood level of the 1%AEP design storm should be as follows or as specified in the district or regional plan:

Freeboard	*Minimum height*
Habitable dwellings (including attached garages)	*0.5 m*
Commercial and residential buildings	*0.3 m*
Non-habitable residential buildings and detached garages	*0.2 m*

The minimum freeboard shall be measured from the top water level to the building platform level or the underside of the floor joists or underside of the floor slab, whichever is applicable".

Adequate access and egress should also be provided to new buildings and subdivisions. In this regard Section 4.3.4.2 of NZS4404:2010 states that the standard recommended for ponding or secondary flow on local roads shall be limited to 100 mm maximum at the centreline and velocity such that the carriageway is passable in a 20-year return period flood events.

4.8.2 Determining minimum floor and subdivision levels

Historically floor levels have been determined by a number of sources including local and regional authorities as well as developers and private landowners.

Building Consent Authorities (or BCA's), who are typically local authorities are required by statute (Building Act 2004) to refuse a building consent if the land is subject to inundation or erosion or if the building work will worsen the inundation or erosion of any other property. This requirement is one of the main reasons why local authorities indicate floodable areas in their district plans. If data is available then the local authority may show minimum floor levels on their district plan maps. Hydraulic modelling is used to help identify the floodable areas which may be:

- commissioned by the local authority who are responsible for managing suburban and rural stormwater reticulation systems.
- sourced from regional authorities who are responsible for managing floodplains.
- obtained from private landowners and developers.

A simple check list is provided below which outlines steps used to determine minimum floor levels. The same principals apply for determining minimum subdivision platform levels.

1 Has survey of subject site been completed? If no, undertake local survey to confirm actual ground levels.

2 Determine flood level to set minimum building floor (or subdivision platform) levels by checking existing flood maps and reports produced by hydraulic modelling.

3 Add freeboard to the determined flood level.

4 Compare flood level obtained Step 2 with actual recorded flood levels if available experienced in recent storm events e.g. July 1998, July 2004 and May 2005. Records might consist of photographs, personal accounts etc. Are any of the actual recorded flood levels flood level higher than the floor (or subdivision platform) level obtained from Step 2.

   - If yes, set minimum building floor (or subdivision platform) level to the higher actual flood level.
   - If no, set minimum building floor (or subdivision platform) level to the flood level obtained from Step 2.

5 Is there any other nearby natural or manmade hydraulic features that could flood the subject site in scenarios other than those identified in Step 2 e.g. breach of a drainage canal stopbank or a lahar from steep catchments vulnerable to high intensity rainfalls? If yes, consult the Bay of Plenty Regional Council for advice.

6 Will the proposed development on subject site increase or worsen flooding of adjacent properties? If yes, then recommend measures to reduce, mitigate or avoid flood risk on adjacent properties.

4.9 Stormwater mitigation

Stormwater drainage systems are generally designed for a moderate sized storm event such as the 10-year return period flood (used for say pipe sizing). Allowance is also made for more severe and less frequent storm events up to the 50 and 100-year return period floods (used for sizing overland flow paths). These two systems are referred to as the primary and secondary systems. Further discussion on storm event sizes is provided in Section 2.3 of Council's Stormwater Guidelines (2012).

The two and 10-year return period daily storm events are used to confirm the ability of a stormwater device (e.g. ponds, swales) to convey peak flows under moderately severe storm conditions. Refer Section 7 of the Council's Stormwater Guidelines (2012).

The issue of which storms to control has been considered and this is discussed in s7.1.2 of the Council's Stormwater Guidelines (2012). Studies comparing pre- and post-development peak flows show that by providing multiple storm control the post-development flow frequency curve comes closest to the predevelopment flow frequency curve. Specifically matching the two and 10-year return period post-development storms to their pre-development levels is proven to be common way of minimising downstream intermediate storm peak discharge increases.

Where there are downstream flooding problems, peak discharges and total runoff volumes for the post-development 100-year return period flood event must 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 site in a catchment, timing of stormwater discharges may also need to be taken into account (refer Section 7 in the Council's Stormwater Guidelines (2012)).

Any development which increases the runoff from sites (both rate and volume) shall be responsible, up to the design standard, for mitigation of these effects. The impact of increasing hardstand areas is normal in say subdivision or roading development so resulting flood effects (both peak flow and volume) determined by a catchment wide analysis must be mitigated to that of predevelopment levels. Mitigation should be applied to post development effects which might otherwise increase the flood risk to neighbouring properties.

The infilling of floodplains is not recommended. However, where necessary, the impact of floodplain infilling on post development flood levels shall be mitigated to that of the predevelopment levels. Pre and post flood levels must be determined based on a catchment wide analysis.

In the absence of catchment wide analysis being carried out to determine post development effects, it is important to err on the side of caution, 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 return period storm for a new highway be limited to 80% of the pre-development peak discharge. The indicative target of 80% will help avoid any cumulative hydrological effects that could increase peak flow downstream. Refer Section 7.1.1, Council's Stormwater Guidelines (2012).

Mitigation options would be reviewed on a case-by-case basis however they might include provision of culverts and/or bridges to not block or restrict flow along overland flow paths, retention ponds, compensatory lowering of land and pumping equivalent volume of additional run off (although pumping is not usually recommended for as a stormwater mitigation option due to on-going operational cost requirements) and lost storage.

With all mitigation measures to be applied, it is strongly recommended that the receiving environment sensitivity to changes be investigated and understood prior to selection of design solutions. Mitigation measures need to be applied that deal with the relevant sensitivity.

The maximum allowable difference between pre- and post-development flood levels is 15 mm. The 15 mm range does not include any allowance for survey errors since inaccuracies in survey and hydraulic modelling precision are common to both pre- and post-development scenarios and can potentially impact both levels.

4.10 Dams

This section includes water supply dams, flood detention dams, soil conservation dams or any other embankment that has the potential to impound water e.g. a culvert embankment.

A small dam is one less than four metres in height (from the base of its foundation with the natural ground to its crest). A small dam is also defined as one that retains not more than three metres of water depth and not more than 20,000 cubic metres of water (NZ Society of Large Dams, NZSOLD, 2000). The water retention height is defined as the vertical height relative to the natural bed as measured from the centre line of the dam structure as shown in Figure 4.1.

Note the water retention height is measured from the lowest downstream outside limit or elevation (toe) of the dam.

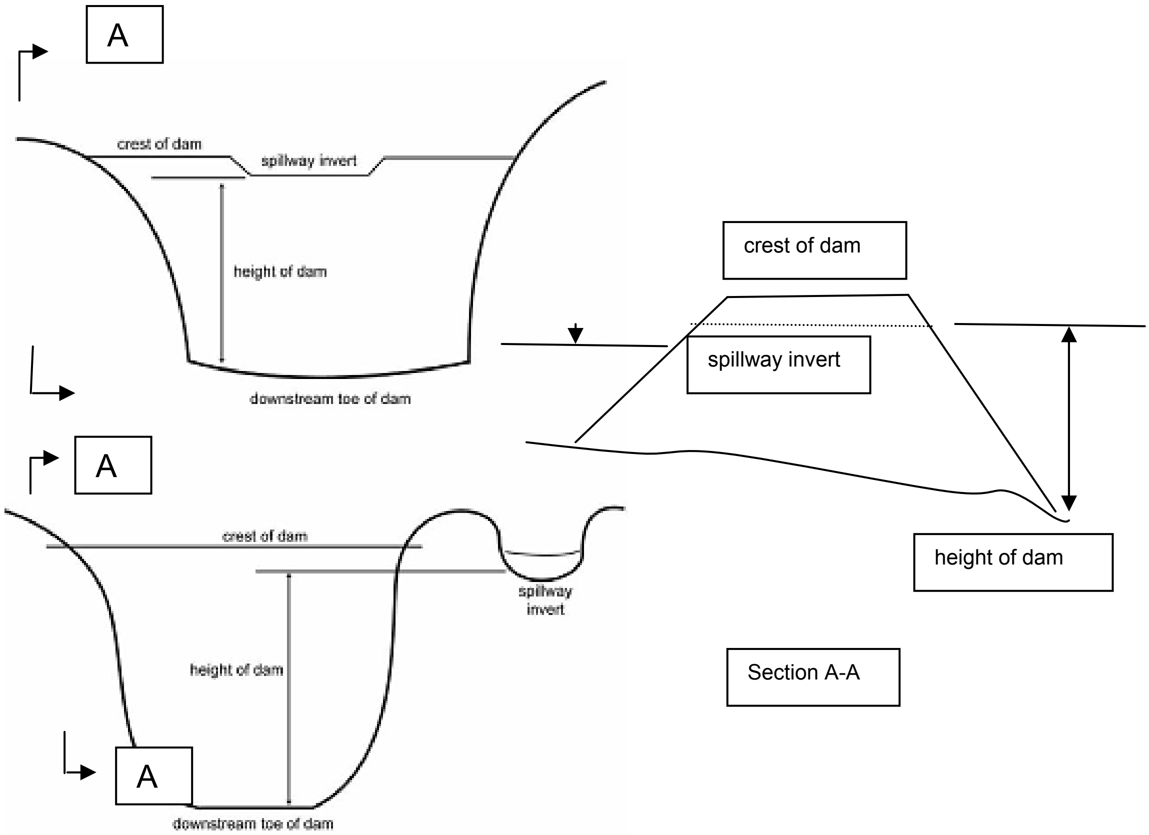


Figure 4.1 Cross-section of a typical small dam defining measurement of the dam water retention height (Source: Regional Water and Land Plan, Bay of Plenty Regional Council, 2008).

A small dam should be designed in accordance with Section 7.4 of these guidelines. Dam owners should also check and satisfy requirements of the:

- RMA
- Building Act
- Other non-statutory guidelines

4.10.1 RMA requirements

Any of the structures detailed in these guidelines may require consent pursuant to the RMA. The requirement may be outlined in a regional plan such as the Land Plan. The activity may also be permitted if it complies with certain rules in a plan.

4.10.2 Building Act requirements

In addition to meeting requirements of the RMA and Regional Water and Land Plan, new and existing dams will need to satisfy requirements of the Building Act namely:

- Obtaining building consents to construct a new dam.
- Adhering to the Dam Safety Scheme for existing and new dams.

Building consents can be obtained by applying to the Building Consent Authorities. Environment Waikato carries out this BCA function on behalf of the Bay of Plenty Regional Council.

The Building (Dam Safety) Regulations 2008 are a section of the reforms of the Building Act 2004. The Dam Safety Scheme aims to ensure that the safety of existing and new dams is checked regularly and that new dams are built and maintained to stay safe. Any dam capable of holding 20,000 m^3 or more of water and is at least three metres in height is subject to satisfying the requirements of the Dam Safety Scheme.

If the dam is subject to the Dam Safety Scheme then owners will need to have their dam classified as either low, medium or high potential impact (in the event the dam fails resulting in an uncontrolled release of reservoir water). Further details on Building Act requirements as it pertains to dams can be found on the Department of Building and Housing website *http://www.dbh.govt.nz/.*

4.10.3 Other non-statutory requirements

The NZSOLD (NZ Society on Large Dams) publication “Dam Safety Guidelines” should be consulted when designing dams. This publication provides helpful guidance on all aspects of dams including regulations, design, construction, commissioning, operational and maintenance as well as dam safety.

Guidelines for the design of detention dams are outlined in a separate Council document titled “Guidelines for the Design, Construction Maintenance and Safety of Small Detention Dams” (BOPRC, 2006).

Stormwater Management Guidelines for Bay of Plenty Regional Council, provide further discussion on the design of detention ponds (both dry and wet ponds) with particular focus on water quality aspects (2012).

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Part 5: Calculation of design flow

5.1 Introduction

The calculation of the design flow should follow standard documented procedures. This will usually require either:

(a) The use of available flow data; or

(b) The determination of the design rainfall and conversion of this to a design flow by examination of the characteristics of the catchment above the site in question.

5.2 At-site analysis

Historical flow data is available for some catchments and Bay of Plenty Regional Council have carried out flood frequency analyses on this data. Bay of Plenty Regional Council should be consulted to see if relevant data or analyses exist for the subject catchment or a neighbouring catchment, with similar hydrological characteristics. Whilst the data will frequently not be measured near the subject site, it can usually be transposed by adjusting the respective catchment areas to the power point of 0.8 (this only applies to peak flow estimates). However, in many cases, the catchment area at the recorder will be an order of magnitude or more than that at the subject site. In this case the methods converting rainfall to design flow should also be applied.

5.3 Design rainfall

The design rainfall for the catchment above the site can be determined using NIWA's (National Institute of Water and Atmosphere) HIRDS Version 3 software available as a web based programme at *http://hirds.niwa.co.nz/*. However HIRDS values should be compared to other data where available, For example:

- Many automatic rain gauges are maintained throughout the region by the various local authorities, private organisations, and by NIWA. The Regional Council publishes data summaries every five years that include statistical analyses of its rain gauges. Access to the data summaries is available off the web at http://monitoring.boprc.govt.nz/MonitoredSites/summary.pdf.
- As a general rule, the sensitivity of results given (by HIRDS at a particular location) to a small shift in location should be tested.

Note that specific rainfall analysis studies have been carried out for Tauranga City Council which gives higher values than HIRDS in some areas. The Tauranga City Council rainfall analysis figures should be utilised for design purposes in Tauranga City Council controlled areas and should be given preference before HIRDS in those areas [(TCC, 2005 & 2006) (BOPRC, 2006)].

HIRDS is a straightforward programme and requires input of the longitude and latitude of the **centre** of the catchment. The HIRDS programme allows the user to nominate their choice of map grid including the New Zealand Map Grid (NZMG) and New Zealand Transverse Mercator (NZTM).

The programme returns a table of rainfall amounts for various return periods and durations. Some judgement is required on the areal extent to which the rainfall intensity is applied. Where rainfall at the centre of the catchment is not representative of average catchment rainfall then a weighted average of rainfall at several sites should be applied and a check made against local rainfall records and anecdotal information.

Designers should note that during Regional Council testing of HIRDS Version 3, it was noted that for some gauge sites HIRDS recommended longer duration rainfall depths significantly higher than the Council's single-gauge analysis. HIRDS Version 3 was calculated using a General Extreme Value (GEV) frequency distribution. The General Extreme Value Type 2 Distribution applies to the flood frequencies of almost all rivers in the Bay of Plenty.

Using NIWA's HIRDS Version 3 website, projected temperature changes can also be inserted to predict the impact on design rainfall due to climate change (Refer Section 5.7). Other data sources should be adjusted using the methods outlined in the Ministry for Environment's Climate Change Guidelines for local government which are updated regularly and available off the Ministry's website [2008].

Generally, the return period of the rainfall chosen should match the return period determined in Section 4. For practical purposes this assumption is the most reasonable to make.

The duration of the rainfall event to choose should match the time of concentration for the catchment. This is the time for water to travel from the farthest point in the catchment to the site of the modification. Its determination requires knowledge of the catchment characteristics (channel slope, flow length, catchment area, elevation difference) and can be calculated by a number of methods (Ramser-Kirpich, Bransby-Williams, US Soil Conservation Service, Nomographs) all of which are described in Technical Memorandum 61 (1980) and Department of Building and Housing (2011) Document E1/Verification Method 1.

Each of these methods for the time of concentration will give a different result. The final choice should not be arrived at by simply averaging the results, but rather should be the one considered most reasonable for the catchment. This will require some judgement and the final choice should be justified. Ranges of time of concentration, rainfall duration and rainfall intensity should be noted for later incorporation into a sensitivity analysis of the design flow calculation. As a guide the Ramser-Kirpich estimate, with 5 to 10 minutes added, usually gives an appropriate estimate for much of the non-urban areas in the Bay of Plenty. The 5 to 10 minutes is due to the unusually greater infiltration capacity of the volcanic soils.

5.4 Catchment characteristics

Most procedures that convert rainfall to runoff require input of the characteristics of the catchment. These include:

- Present ground cover (grass, roads, buildings, bush, trees etc).
- Future ground cover. If the structure is designed to last well into the future, the changes in ground cover must be considered.
- Waterway channel length.
- The direct length from the top of the catchment to the site.
- Area.
- Soils.

These characteristics can be obtained from aerial photos, contour maps and site inspections. Bay of Plenty Regional Council staff have a good knowledge of catchment characteristics and are happy to discuss these. Historical flow data is available for some catchments.

5.5 Flow calculation

There are a number of recognised methods in New Zealand for estimating catchment flows from design rainfalls. These are listed below with some comments on their use arising from their use in the Bay of Plenty.

For larger catchments or where significant storage elements (e.g. ponds) or backwater effects (e.g. tidal effects) are incorporated, surface water runoff shall be determined using an appropriate hydrological or hydraulic model.

5.5.1 TM61

This is an empirical method suitable for all catchment sizes. It has been in use for some time and in some parts of the country has been replaced by other methods. Bay of Plenty Regional Council continues to use it, recognising judgement is needed in deciding on the most appropriate runoff coefficient and time of concentration to use. Table 5.1 gives values of WIC. to use in TM61. Soil types in the Bay of Plenty are generally absorbent but in some cases are moderately or very absorbent.

Table 5.1 Values of W_{IC}. for use in TM61

Soils	Ground Surface - Cover	W_{IC}
Impervious soils (such as clay soils with poor structure e.g. northern yellow brown earths).	Urban catchments:	
	• High density development(West Coast high rainfall)	1.8
	• Moderate to low density development	1.5
Any soil, if saturated, is included in this group.	Mainly bare surfaces	1.2
	Average short grazed catchments	1.1
	30% of area in long grass, scrub or bush	1.0
	60% of area in long grass, scrub or bush	0.9
	100% of area in long grass, scrub or bush	0.8
Moderately absorbent soils (such as medium textured soils with good structure e.g. southern yellow brown earths).	Urban catchments:	
	• High density development	1.7
	• Moderate to low density development	1.3
	Mainly bare surfaces	1.1
	Average short-grazed catchments	1.0
	30% of area in long grass, scrub or bush	0.9
	60% of area in long grass, scrub or bush	0.8
	100% of area in long grass, scrub or bush	0.7
Absorbent soil (such as deep yellow brown sands and pumice soils).	Urban catchments:	
	• High density development	1.5
	• Moderate to low density development	1.2
	Mainly bare surfaces	1.0
	Average short-grazed catchments	0.9
	30% of area in long grass, scrub or bush	0.8
	60% of area in long grass, scrub or bush	0.7

Soils	Ground Surface - Cover	W_{IC}
	100% of area in long grass, scrub or bush	0.6
Very absorbent pumice soil	Mainly bare surfaces Average short-grazed catchments 30% of area in long grass, scrub or bush	0.5
	60% of area in long grass, scrub or bush 100% of area of long grass, scrub or bush	0.4

5.5.2 Rational method

Generally useful for catchments less than 50.0 ha in size, and is not applicable to catchments with any notable storage or backwater effects. This too is an empirical method of less complexity than TM61 but still requiring good judgement on runoff coefficients and time of concentration. In Bay of Plenty Regional Council's experience it tends to overestimate flows from the pumice catchments in the Bay of Plenty. Table 5.2 gives values for C for use in the Rational Method.

The rational method formula for calculating flow is as follows:

Qp = 1/360 CIA.

Where

Qp = design peak discharge, in m^3/s.

C = coefficient of runoff, which is dimensionless, refer to Table 5.2 and apply the slope adjustments noted beneath it.

I = average storm rainfall intensity (mm/hour) for the selected return period and a duration equal to the catchments time of concentration.

A = catchment area, in hectares (ha) (Note 100ha = 10 km^2).

Section 7 of Council's stormwater guidelines provides further discussion on use of the Rational Method particularly where modification of the C coefficient may be necessary e.g. on slopes.

The rational method is not to be used for:

- Catchment areas larger than 50.0 ha in area
- Drainage systems including significant storage areas
- Drainage systems including backwater effects
- Catchment analysis for storms greater than 100-year return period

The rational method analysis is unlikely to be accepted for assessment of dam spillway capacities (except small erosion and sediment control dams).

5.5.3 Modified rational method

Generally useful for catchments greater than 50.0 ha. The method is considered slightly more rigorous than the Rational Method as it allows consideration of more catchment characteristics such as catchment shape characteristics. Table 5.2 may also be used for the Modified Rational Method with slope adjustments noted beneath it.

The rational method and modified rational method can be used to calculate small dam spillway capacities for events up to the 100-year event however they are not to be used and are unlikely to be accepted for assessment for storms severer than this value.

Table 5.2 Run-off coefficients for use in rational and modified rational methods

Description of Surface	C
Natural surface types	
Bare impermeable clay with no interception channels or run-off control	0.70
Bare uncultivated soil of medium soakage	0.60
Heavy clay soil types:	
• pasture and grass cover	0.40
• bush and scrub cover	0.35
• cultivated	0.30
Medium soakage soil types:	
• pasture and grass cover	0.30
• bush and scrub cover	0.25
• cultivated	0.20
High soakage gravel, sandy and volcanic soil types:	
• pasture and grass cover	0.20
• bush and scrub cover	0.15
• cultivated	0.10
Parks, playgrounds and reserves:	
• mainly grassed	0.30
• predominantly bush	0.25
Gardens, lawns etc	0.25
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 town house developments	0.65
Residential areas in which the impervious area is less than 36% of gross area	0.45
Residential areas in which the impervious area is 36% to 50% of gross area	0.55
Source: Table 1 from DBH (2011) document	

The runoff coefficients are to be modified for slope as follows[1]:

- -0.05 for Slope < 5%
- No adjustment for 5%<Slope<10%
- +0.05 for 10%<Slope<20%
- +0.10 for Slope>20%"

[1] Source: Table 2 from DBH (2011) document.

5.5.4 Flood frequency in New Zealand (regional method)

Flood frequency in New Zealand is a regional method suitable for all rural catchments except those in which there is snow-melt, lake storage or ponding. This method developed by McKerchar and Pearson in 1989 is listed as a permissible alternative to the modified rational method by the Building Industry Authority in Document E1. It provides contour maps of the relationship between mean annual flood and catchment area and proposes a method of extrapolation to other return period floods. It out-performs the previous regional flood estimation method of Beable and McKerchar 1982.

It is a document intended for use by experienced professionals in water resource engineering and is prone to prediction errors in catchments less than 10 km^2 and should be checked against the rational method. In the Bay of Plenty Regional Council's experience the method tends to underestimate flows particularly in smaller catchments. This may be due to the contour maps being derived mainly from large catchment sizes – sometimes larger than the subject catchment by one to two orders of magnitude.

5.6 Other methods

Other sometimes more in-depth analysis methods may be required depending on the catchment specific circumstances, drainage system complexity or potential downstream impacts. The method adopted needs to be appropriate for the situation being assessed. Potential methodologies can be discussed with the Regional Council.

Other alternative methods include:

- Advanced hydrological and hydraulic modelling utilising commercially available software (e.g. HEC, DHI, Infoworks)
- Unit hydrograph methods (e.g. the SCS method)
- Other rainfall-runoff methods (e.g. RORB, SWMM)
- Extrapolation techniques

If no hydrological data is available at a site then an alternative gauging site on a nearby waterway with similar hydrological characteristics can be used. Data from more than one site should be used for comparison and sense checking purposes. Scaling of flood flows should be performed to adjust for differences in catchment areas. Flood flows should be scaled by the ratio of the catchment area to the power of 0.8 as discussed in Section 3 – mean annual floods outlined of flood frequency in New Zealand (McKerchar and Pearson, 1989) i.e.

$Q1/Q2 = (A1/A2)^{0.8}$

where Q is the flood discharge and A is the catchment area.

5.7 Mitigation

Should the proposed post-development peak discharge increase compared to the pre-development discharge, then mitigation works should be aimed at ensuring no change to peak discharge resulting from the development. This is typically achieved through detention, where storage is provided to buffer the post-development inflow such that the pre-development rate is not exceeded, with slow release to downstream.

Should the proposed post-development velocity regime change compared to the pre-development regime, then appropriate mitigation for this will be required. For example, a downstream waterway may be prone to erosion once a certain threshold velocity (linked to discharge) is attained. If this threshold velocity is achieved under current land use for the design event, then mitigation for peak discharge, described above, could result in increased erosion due to the velocity threshold being exceeded for a longer period of time even if the pre-development peak discharge is not exceeded. In this case a different set of mitigation works will be required to those needed for peak discharge impact sensitivity.

It is also possible that, due to hydraulic restrictions downstream, a waterway may show impact sensitivity to total runoff volume from a target sub-catchment. In this case, if the proposed development results in increased impervious area and hence increased runoff volume, mitigation by detention alone is unlikely to be adequate.

Depending on specific site characteristics, some mitigation measures may be able to be applied "on site" (i.e. within the bounds of the proposed change in land use and development) while in other cases "off-site" (in areas outside the boundary of the proposed development) measures may be the most viable option. These "off-site" measures may be located some distance from the area in which the change has been made. It is up to the designer to assess issues, impacts and determine appropriate potential solutions either within or outside the development site.

5.7.1 Peak discharge attenuation

For a receiving environment that is shown to be potentially impacted by only changes in peak discharge, mitigation for a changed hydrological response may be able to be achieved through detention on or off site. This involves temporary storage of the volume of inflow that is in excess of the pre-development peak outflow. Calculation of this volume required for mitigation requires approximation of a runoff hydrograph to be applied.

In many cases advanced hydrological and hydraulic modelling analyses can be used for approximation of this runoff hydrograph, both for the pre- and post-development cases. Critical duration analysis may also be required in some cases e.g. for complex drainage systems, systems including storage and when assessing potential downstream impacts. However, in the absence of this detail an approximation using straight-line hydrographs may be made as follows.

For both the pre- and post-development cases the hydrograph is assumed to start with zero discharge at time equals zero, and rise linearly to peak discharge (as estimated using an appropriate method as described above) at time equals time of concentration. The falling limb can be approximated using a linear decay from the peak to zero at time equals 2.67 times the time of concentration, as shown in Figure 5.1.

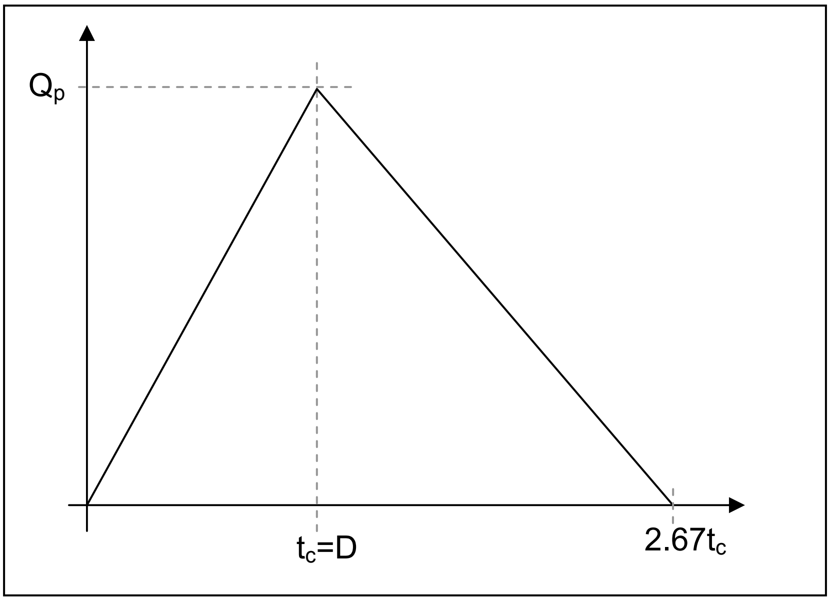


Figure 5.1 Simple hydrograph for storm flow

In Figure 5.1:

Q_p = peak discharge estimated using the Rational Method (m^3/s)

T_c = time of concentration (s)

D = rainfall duration (s)

This gives the total runoff estimation as

$V_{tot} = 4/3.\ t_c.\ Q_p$

There are a number of equations available for the calculation of t_c. (e.g. TM61 references methods by Ramser-Kirpich, Bransby-Williams and the US Soil Conservation Service). At least three different methods should be used to calculate t_c and the final value selected based on the closest two calculated values.

Hydrographs produced in this way may be integrated to yield total runoff volume for each case.

The required detention volume can be calculated by taking the post-development hydrograph as an inflow time series, with outflow limited to the pre-development peak discharge (i.e. it is not the difference between pre- and post-development total runoff volume).

An example is shown in Figure 5.2, where pre- and post-development hydrographs, of shape as given in Figure 5.1, and plotted together with the resulting (detained) hydrograph, derived by routeing the post-development hydrograph through storage. In Figure 5.3 cumulative volume time series for these three hydrographs are shown, and these can be used to estimate the required storage volume. This can be done by finding the maximum difference between the "Post" and the "Detained" cumulative volume time series (noting that the slope of the "Detained" hydrograph is equal to the maximum slope of the "Pre" hydrograph).

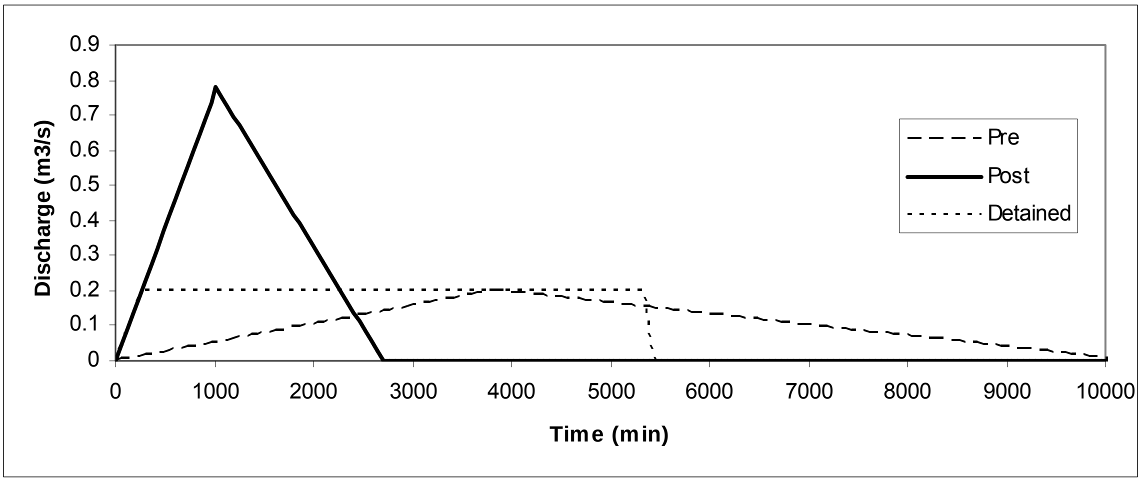


Figure 5.2 Sample Pre-, Post- and Detained hydrographs

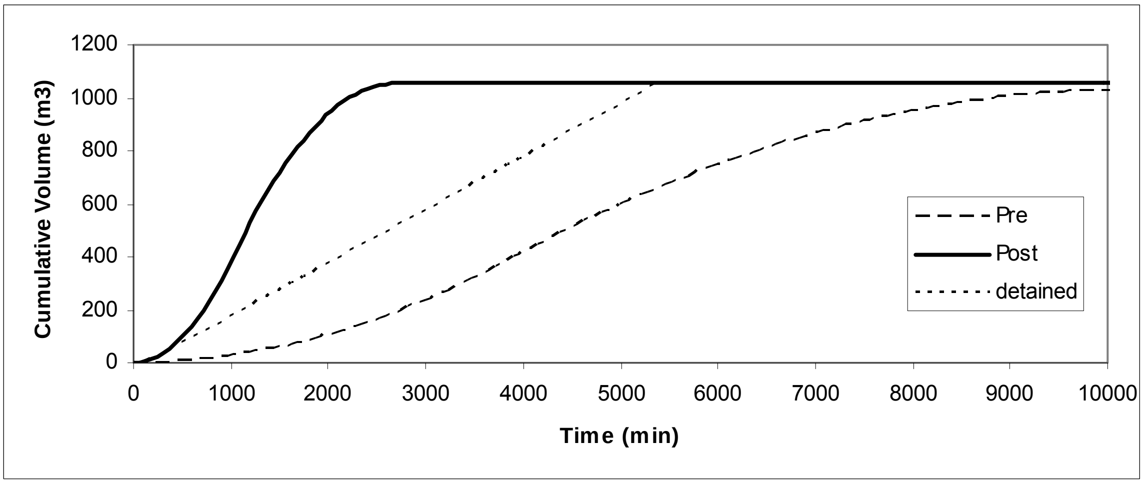


Figure 5.3 Sample Cumulative Volume Time Series

5.7.2 Velocity mitigation

Should the receiving environment be found to be potentially impacted by changes in both peak velocity and duration over which a threshold velocity is exceeded, then appropriate hydraulic analyses will be required to address the effects of a change to the hydrological response of a target catchment.

In such cases the details of the impact sensitivity to velocity requires full understanding such that case-by-case mitigation measures can be developed. In many cases, off-site mitigation may be required, and should be considered.

5.7.3 Total runoff volume mitigation

In many cases, land use change can result in increased volume of runoff in response to a given rainfall event (as compared with the existing situation). Unless the additional volume can be discharged to an alternative outfall, areas that show impact sensitivity to total runoff volume will require detailed analysis to confirm the mitigation measures proposed.

As with the mitigation for impact to velocity described above, in many cases the mitigation measures for areas sensitive to changes in total runoff volume may need to be located off-site.

5.8 Changing hydrological regime

The Bay of Plenty Regional Council has adopted the Intergovernmental Panel on Climate Change (IPCC) estimates for climate change in the region's rivers, drainage schemes and stormwater systems.

Climate change and in particular the global warming has the potential to increase the magnitude, level and frequency of flooding. Hence the capacity of existing and proposed flood protection and stormwater assets must be reviewed periodically when the new flow data becomes available.

At present the Bay of Plenty Regional Council evaluates the potential effects of global warming on a case by case basis. Assessment of effects considers:

- certainty of available information.
- cost of retrofitting new or renewed structures.
- assets design life span.

Hydrological design data may need to be adjusted for design purposes to account for future climate change that includes sea level rise and increased frequency and magnitude of floods.

The impact of climate change and guidance on how this is to be taken into account in the design of hydraulic structures is outlined in Sections 5.8.1 to 5.8.3 below. Further discussion on climate change and its effect on stormwater design is provided in s.7.1.8 of the Council's Stormwater Guidelines (2012).

5.8.1 Sea level rise

The IPCC issues projections on the impact of global warming on sea levels at five yearly intervals. In 2007 the IPCC predicted increases between 0.18 m–0.59 m around New Zealand coastlines by 2100AD. There could be an extra 0.10–0.20 m on the upper range if there is an increase in the rate of melting of the major ice sheets (MfE, 2008).

In addition to the 0.128 m rise over the last century (1900-2000) IPCC predictions are for a rise of between 0.28 m and 0.79 m over the next 100-years.

The Bay of Plenty Regional Council has adopted the IPCC estimates for the purpose of the Proposed Bay of Plenty Regional Coastal Environment Plan. By 2040 and 2090 the expected sea level rises are 0.2 m and 0.49 m respectively. Note the recent findings that sea level rise is expected to be 0.36 m by 2060 and 0.8 m by 2100 (refer section 4.6).

5.8.2 Increased frequency and magnitude of flooding

A second, but less quantified adverse effect of global warming is that the frequency and magnitude of high intensity rainfalls are expected to increase.

It will be important to take account of increased frequency and magnitude of flooding when reviewing the flood protection of assets. When key structures and those difficult to retrofit (e.g. flood walls) come up for construction or renewal then they should be designed for the likely intensification of flows during their design life.

5.8.3 Climate Change Guidelines

Advice from the MfE, Climate Change Office (2008) is that the average of the estimates for forecast increase in temperature at 2090 is 2.1°C above present temperatures. To assist designers of hydraulic structures, the Ministry of Environment (2008) has published expected temperature and sea level increases in the Bay of Plenty for the years 2040 and 2090. The MfE will next update its temperature and sea level forecasts in 2013. These climate change variables and example of applications are shown in Table 5.3.

Table 5.4 shows MfE (2008) recommended percentage adjustments **per degree** Celsius of warming to apply to extreme rainfalls; values are given for various average recurrence intervals (ARI's) and for rainfall durations between 10 minutes and 72 hours. Examples of how to apply the adjustments can be found in the Ministry of Environment's Guidelines.

Table 5.3 Design scenarios and their associated climate change factors

Design scenario	Rainfall frequency/intensity data	Sea level	Application
Existing design service levels	Uses current data sourced from say Bay of Plenty Regional Council data and HIRDS[2] etc.	Uses current sea level data based on Bay of Plenty Regional Council Guidelines and Tidal Almanac.	Current boundary conditions normally used in hydraulic models to calculate design flood levels in waterways[3].
2040 design flood levels	As above but apply MfE[4] factors to current rainfall that anticipates an average temperature increase of between 0.2ºC and 2.4ºC by the year 2040 with the mean increase of 0.9 ºC.	Use MfE estimate of sea level rise at 2040 (current expected rise ranges between is 0.20 m and 0.27 m[5]).	It is assumed that Bay of Plenty Regional Council stopbanks settle over a period of 20-years so topping up to the design level expected in 2040 could provide some future proofing in the interim.
2090 design flood levels	As above but apply MfE[6] factors to current rainfall that anticipates an average temperature increase of between 0.6ºC and 5.5ºC by the year 2090 with the mean increase of 2.1ºC.	Use MfE estimate of sea level rise at 2090 (current expected rise ranges between 0.50 m and 0.80 m[7]).	Design flood levels should be selected to reflect the lifecycle of structures such as floodgates, bridges, floodwalls and pump stations etc. Hence if concrete structures last say 70 years than it would be appropriate to use design flood levels expected in say 2090.

[2] HIRDS (High Intensity Rainfall Data System) is a rainfall intensity/frequency database produced and supported by NIWA.
[3] Some current Bay of Plenty Regional Council modelling includes a 0.5m sea level rise expected in 2080.
[4] Refer Table 2 in Ministry of Environments (MfE's) Climate Change Guidelines (2008).
[5] Refer Table 2.3 in MfE's Coastal Hazards & Climate Change Guidance Manual (2008)
[6] Refer Table 2 in MfE's Climate Change Guidelines (2008).
[7] Refer Section 2.2.4 and Table 2.3 in MfE's Coastal Hazards & Climate Change Guidance Manual (2008)

Ministry for the Environment climate change guidelines are also available on the web at http://www.mfe.govt.nz/issues/climate/resources/local-govt/index.html.

Table 5.4 % increase in rainfall intensity per degree of expected temperature increase[8]

ARI (years) Duration	2	5	10	20	30	50	100
< 10 mins	8.0	8.0	8.0	8.0	8.0	8.0	8.0
10 mins	8.0	8.0	8.0	8.0	8.0	8.0	8.0
30 mins	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	6.5	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

Designers should also be aware that due to the non-linearity of the rainfall-runoff relationship, the percentage increase in flood flows will be greater than the percentage increase in rainfall. This is because the relative proportion of rainfall losses decreases and the proportion of runoff thus increases with increasing rainfall depths. Unit hydrograph studies carried out by the regional council have shown that for a 16% increase in rainfall, the flows increased by 20% or slightly more. For example for a peak 100-year flow the expected increase in run-off in 2090 is:

Increase in temperature to 2090	2.1°C
Increase in rainfall intensity is 2.1°C x 8%	16.8%
Whereas increase in 100-year flow likely to be at least	20%

5.9 Summary

This section has focussed largely on the methods available to determine a peak design flow at a site under consideration by deriving runoff from rainfall data. By its nature it is an in-exact science and each method will produce different answers. Where at-site flood frequency data is available it should be used at the site. This will likely provide the most accurate estimate when the catchment area at the recorder is not more than one order of magnitude different from that at the subject site. Considerable judgement is required to select the most appropriate figure for design and must take account of the importance of the modification and the consequences of flows in excess of the design flow. Reasons for choosing the design flow should be clearly stated and an assessment made of the effect of flows larger than this.

[8] Refer Table 7 in Ministry of Environments (MfE's) Climate Change Guidelines (2008).

Part 6: Water level calculation

6.1 Introduction

Using the estimated design flow, the modification to the waterway can be designed. These modifications will usually decrease the waterway area and cause the effects described in Section 1. The less the waterway area is reduced the less the effects. However, any structure or improvement to a waterway will invariably increase in cost as waterway area increases. Hence there will be an optimum modification size where the total of the costs of the modification and of prevention or mitigation of adverse effects is minimised.

This section of the guidelines highlights the importance of defining the flow conditions (water levels, velocities) in the waterway prior to the proposed modification and provides guidance on how to determine the flow conditions for the modified waterway.

6.2 Site description

It is necessary to adequately describe the site. This will require at least the following:

(i) A plan showing the site location on NZMS 260 series map, the catchment boundary, soil type, vegetation.

(ii) Site plan at a scale, which adequately shows the waterway 50 m either side of the site. Recent aerial photographs at scales of 1:1000 are ideal.

(iii) A long-section along the waterway from 50 m upstream to 50 m downstream of the site. The purpose of the long-section is to show adequately the slope of the waterway and therefore its scale and detail should be carefully selected. Where other characteristics of the river more than 50 m away affect flood levels, then information on these features may be necessary and hydraulic computer modelling required.

(iv) Typical water level profile(s) plotted on the long-section.

(v) Surveyed cross-sections at appropriate intervals along the waterway from 50 m upstream to 50 m downstream of the site. The spacing of the cross-sections, their extent, scale and detail again should be carefully selected. This will require the advice of an experienced person who can assess the hydraulic effect of topographical features on the proposed waterway. All points on the cross-sections must be levelled to a datum common to the entire set of drawings.

(vi) The typical existing channel cross-sectional area shall be calculated. This will ensure that any proposed opening (e.g. bridge or culvert) will be at least larger than the existing channel area.

(vii) The above drawings also need to show, in the opinion of an experienced hydraulic engineer, any other features that will affect the hydraulics of the waterway e.g. sudden constrictions or expansions; bends; trees or bush along the waterway; downstream expanses of water (lakes, estuaries, harbours, sea); downstream river or stream confluences.

(viii) A description of the materials in the bed and banks of the waterway (rock, boulders, cobbles, gravel, sand etc.) and a sediment site grading. This is used to determine Mannings n coefficient.

(ix) A most important aspect is to obtain any existing data on historical floods recorded along the waterway. The key data will be flow rate, water levels, contributing rainfall, downstream water levels, level/design information on old bridges at the site, details of culvert washouts.

(x) Photographs of the site and upstream and downstream.

6.3 Normal water surface

With the design flow estimates from Section 5 and the site data from Section 6.2, the normal water surface (NWS) through the site should be determined. The NWS is represented by the water levels throughout the existing site that occur during the design event. The downstream length of the NWS is particularly important, as in most cases it is the water level downstream of the proposed waterway modifications that will control water levels within and upstream of the site. The exceptions to this rule are when the site is steep or where the modification creates an upstream constriction (e.g. culvert inlet control).

The determination of the NWS requires application of standard hydraulic calculations such as the Standard Step Method or those available on many computer software systems for the determination of water surface profiles (HEC-2, MIKE-11). For more detail the user is referred to Henderson (1966).

In the Bay of Plenty Regional Council's experience the commonly encountered difficulties in applying this and other methods utilising Mannings equation are as follows:

(a) Provision of adequately detailed geometric data (cross-sections, long-sections, plans) of the waterway. The more accurate and detailed this information the better the results. Good judgement by experienced people is required to ensure enough data exists. It is particularly important in situations when the flow area increases significantly at high flows such that the cross-sections are extended to include all the flood plains that become submerged.

(b) The choice of Mannings n. Although many of the references contain tables of n the choice is very much a matter of expert judgement. Hicks and Mason (1998) assign hydraulic roughness coefficients in New Zealand river reaches by a visual comparison method for rivers of flows 0.1–353 cumecs, slopes of 0.00001–0.042 m/m and bed materials of silt to bedrock. That handbook emphasises that n can vary significantly with flow and should be based on the graphs of Mannings n versus discharge not the n value for mean annual flow. It contains good photographs and data on mean annual flood, mean annual flow and bed grain size gradations. Table 6.1 lists Mannings n for various channel types and conditions.

(c) Lack of calibration and checking. If flood data and water levels have been obtained in Chapter 6.2 these can be used to check the accuracy of the NWS. For more critical structures, it is suggested gaugings be undertaken at various flows to obtain this data.

(d) Determination of flow type. In most cases the flow under consideration will be sub-critical (low velocity, mild slope, Froude number less than 1). Under these conditions it is the downstream water level that control upstream flow conditions. However in rare cases supercritical flow may exist (high velocity, steep slope, Froude number greater than one). In this case the upstream water level determines flow conditions downstream. Extreme care must be taken if the computer software predicts supercritical flow, as flood levels markedly decrease under this flow type and thus flood levels may be markedly under-estimated. In most instances the flow will not become supercritical, because of scour that will occur, thus maintaining subcritical flow. *Correct determination of flow type is vital before effects can be considered of any proposed waterway modifications.*

Table 6.1 Manning's Roughness Coefficients (from "Urban Drainage Design", Sutherland Shire Council, Sydney 1992)

I	**Closed Conduits:**	
A	Concrete pipe	0.011-0.013
B	Corrugated metal pipe or pipe arch:	
	1 68 mm by 13 mm corrugation	0.24
	2 150 mm by 50mm corrugation (field bolted)	0.030
C	Vitrified clay pipe	0.012-0.014
D	Cast iron pipe, uncoated	0.013
E	Steel pipe	0.009-0.011
F	Brick	0.014-0.017
G	Monolithic concrete	
	1 Wood forms, rough	0.015-0.017
	2 Wood forms, smooth	0.012-0.014
	3 Steel forms	0.012-0.013
II	**Open Channels, lined (straight alignment):**	
A	Concrete, with surfaces as indicated:	
	1 Formed, no finish	0.013-0.017
	2 Float finish	0.013-0.015
	3 Float finish, some gravel on bottom	0.015-0.017
	4 Gunite, good section	0.016-0.019
	5 Gunite, wavy section	0.018-0.022
B	Concrete, bottom flat finished, sides as indicated:	
	1 Random stone in mortar	0.017-0.020
	2 Dry rubble (rip rap)	0.020-0.030
C	Gravel bottom, sides as indicated:	
	1 Formed concrete	0.017-0.020
	2 Random stone in mortar	0.020-0.023
	3 Dry rubble (rip rap)	0.023-0.033
D	Brick	0.014-0.017
III	**Open channels, excavated (straight alignment, natural lining):**	
A	Earth, uniform section:	
	1 Clean, after weathering	0.018-0.020
	2 With short grass, few weeds	0.022-0.027
	3 In gravelly soil, uniform section, clean	0.022-0.025
B	Earth, fairly uniform section:	
	1 No vegetation	0.022-0.025
	2 Grass, some weeds	0.025-0.030
	3 Dense weeds or aquatic plants in deep channels	0.030.0.035
	4 Sides clean, gravel bottom	0.025-0.030
	5 Sides clean, cobble bottom	0.030-0.040
C	Dragline excavated or dredged:	
	1 No vegetation	0.028-0.033
	2 Light brush on banks	0.035-0.050
D	Rock:	
	1 Smooth and uniform	0.035-0.040
	2 Jagged and irregular	0.040-0.045
E	Channels not maintained	
	1 Dense weeds, high as flow depth	0.08-0.12
	2 Clean bottom, brush on sides	0.05-0.08
	3 Clean bottom, brush on sides, highest stage of flow	0.07-0.11
	4 Dense brush, high stage	0.10-0.14
IV	**Highway channels and swales with maintained vegetation: (values shown are for velocities of 0.6 m/s and 1.8 m/s)**	
A	Depth of flow up to 210 mm	
	1 Good stand, any grass:	
	(a) Mowed to 50mm	0.045-0.07
	(b) Length 100 mm to 150 mm	0.05-0.09
	2 Fair stand, any grass:	
	(a) Length about 300 mm	0.08-0.14
	(b) Length about 600 mm	0.12-0.25
B	Depth of flow 210 mm to 460 mm:	
	1 Good stand, any grass:	
	(a) Mowed 50 mm	0.07-0.12
	(b) Length 100 mm to 150 mm	0.10-0.20
	2 Fair stand, any grass:	
	(a) Length about 300 mm	0.06-0.10
	(b) Length about 600 mm	0.09-0.17
V	**Street and Expressway Gutters:**	
A	Concrete gutter, trowelled finish	0.012
B	Asphalt pavement	
	1 Smooth texture	0.013
	2 Rough texture	0.016
C	Concrete gutter with asphalt pavement:	
	1 Smooth	0.012
	2 Rough	0.013
D	Concrete pavement:	
	1 Float finish	0.014
	2 Broom finish	0.016
E	For gutters with small slope, where sediment May accumulate increase above values of n by	0.002
VI	**Natural stream channels**	
A	Streams	
	1 Fairly regular section	
	(a) Some grass and weeds, little or no brush	0.030-0.035
	(b) Dense growth of weeds, depth of flow greater that weed height	0.035-0.05
	(c) Some weeds, light brush on banks	0.035-0.06
	(d) Some weeds, heavy brush on banks	0.05-0.07
	(e) Some weeds, dense willows on banks	0.06-0.08
	(f) For frees within channel, with branches submerged at high stage increase all above values by	0.01-0.02
B	Flood plains, adjacent to natural streams	
	1 Pasture, no brush	
	(a) Short grass	0.030-0.035
	(b) High grass	0.035-0.05
	2 Cultivated areas:	
	(a) No crop	0.03-0.04
	(b) Mature row crops	0.035-0.045
	3 Heavy weeds, scattered brush	0.05-0.07
	4 Light brush and trees:	
	(a) Winter	0.05-0.06
	(b) Summer	0.06-0.08
	5 Medium to dense brush	
	(a) Winter	0.07-0.11
	(b) Summer	0.10-0.16
	6 Dense willows	0.15-0.20

Note: The value of n for natural channels must be increased to allow for the additional energy loss caused by bends. The increase may be in the range of perhaps 3 to 15 percent.

6.4 Modified water surface

6.4.1 Introduction

Having established the NWS through the waterway during the design event, it is necessary to determine the effect of the proposed waterway modification on water levels and velocities upstream and downstream of the modification. Each type of modification produces these effects in different ways.

6.4.2 Culverts

The design of culverts is described in the following references:

- Christchurch City Council, Waterways, Wetlands and Drainage Guide, 2011.
- Concrete Pipe Association of Australasia, Hydraulic Design Manual 1997 (also known in the past as the Humes Manual).
- Waterway Design, Section 7 of Austroads (1994).

The first two are a succinct summary of key design requirements while the third is a comprehensive but very readable design manual. The latter has been used to prepare the following notes.

As stated before, culverts invariably decrease the waterway area causing a rise in upstream water levels, leading to inundation or flooding, and an increase in velocity at the culvert entry and exit, leading to the potential for scour.

These effects are reduced by:

(a) increasing the culvert size or number of culverts,

(b) aligning the culvert vertically and horizontally to the existing waterway geometry,

(c) the use of improved inlets and outlets and;

(d) increasing allowable headwater by the construction of levees or embankments.

The most important consideration in culvert hydraulics is whether the flow is subject to inlet or outlet control. Refer to Figures 6.1 and 6.2.

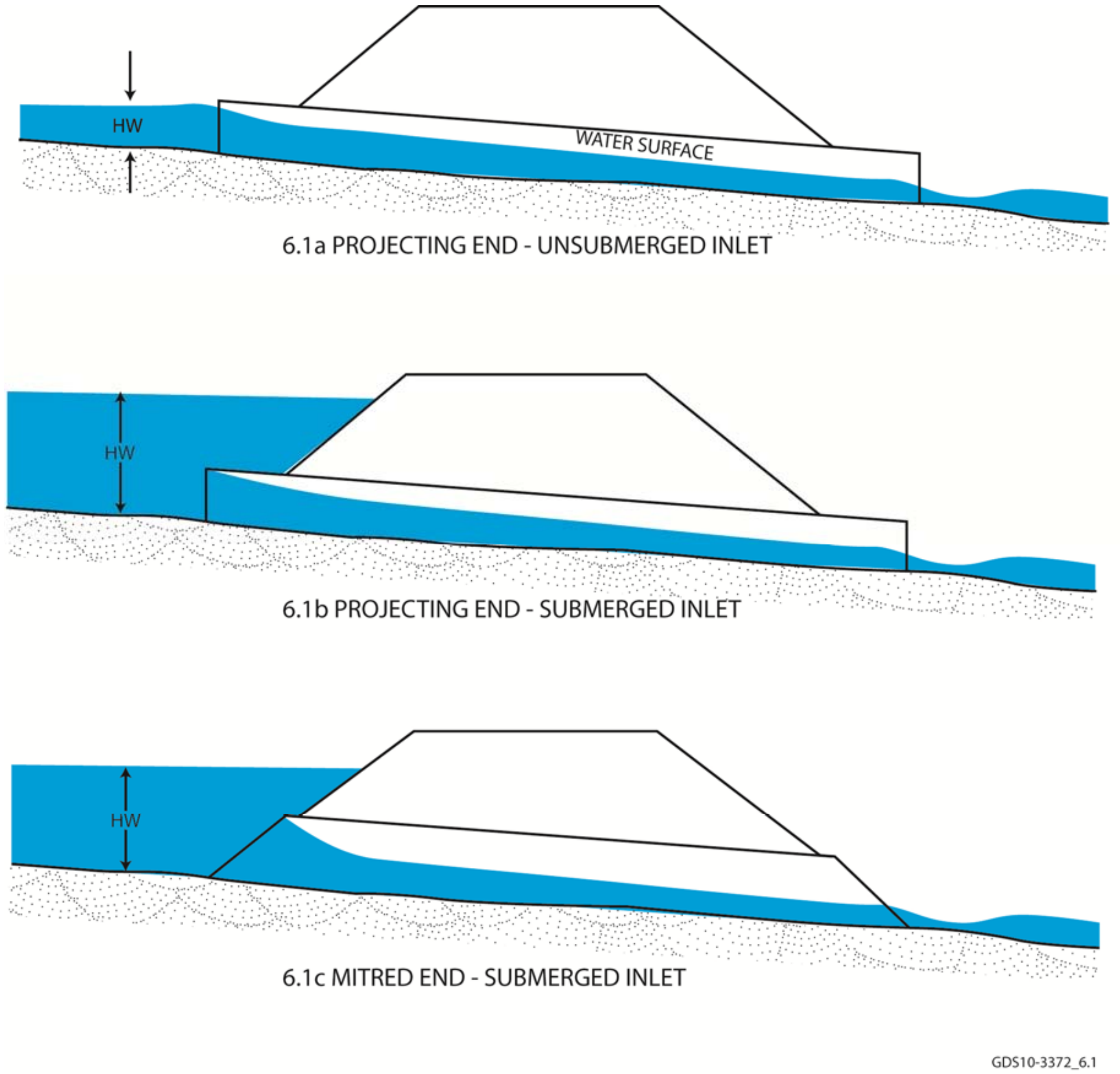


Figure 6.1 Flow profiles for culvert under inlet control

With culverts subject to inlet control, the important factors are the entrance conditions and the projection of the culvert into the headwater pond. Sketches of inlet control flow for both unsubmerged and submerged projecting entrances are shown on Figures 6.1a and 6.1b. Figure 6.1c shows a mitred entrance flowing submerged with inlet control. In inlet control the roughness, slope and length of culvert and the outlet conditions, including depth of tailwater, are not factors in determining culvert capacity.

Outlet control occurs (Figure 6.2) either in long culverts laid on flat grades and/or in culverts with high tailwater depths. Culverts with outlet control can flow with the culvert full or part full. If both the inlet and outlet are submerged the culvert flows full under pressure (Figure 6.2a). The culvert can also flow full over part of its length then part-full at the outlet (Figure 6.2c). If the culvert grade is flat enough, the culvert can flow under outlet control, part full along its entire length and with both inlet and outlet unsubmerged (Figure 6.2d).

In designing culverts, the type of control is determined by adopting the greater of the headwater depths calculated for both inlet and outlet control. For the two types of control, different factors and formulae are used to calculate the hydraulic capacity of the culvert. These are covered in the above references.

Culverts installed under high embankments may allow higher headwaters to be adopted. If deep ponding is envisaged, potential failure of the embankment should be investigated. The design standards of Section 4 and the design details of Section 7 should be adopted in this case.

Consideration must also be given to the potential for debris from the catchment to block the culvert inlet. This is discussed further in Section 7.

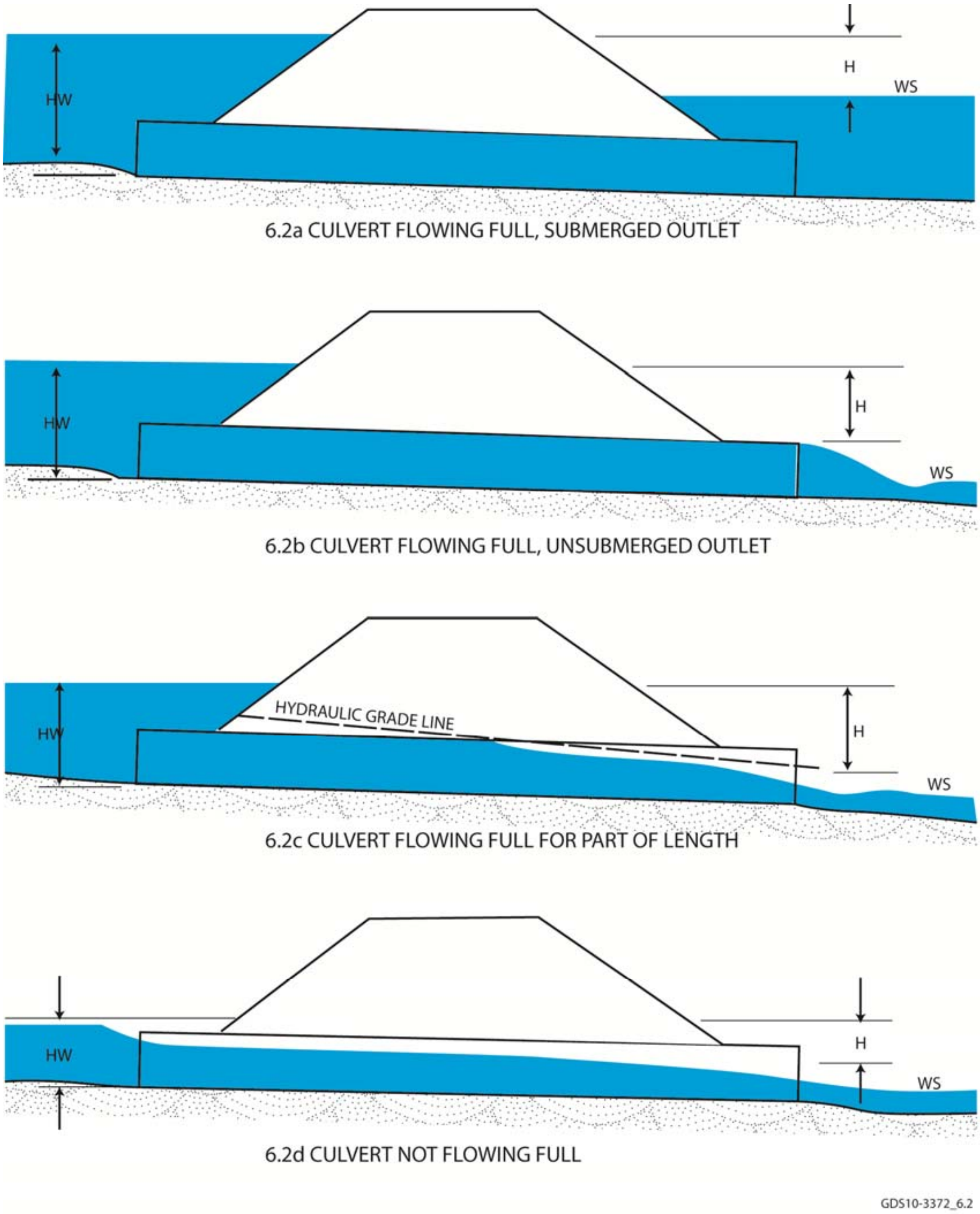


Figure 6.2 Flow profiles for culverts under outlet control

6.4.3 Bridges

Similar to culverts, bridges too have the potential for increasing upstream water levels and for increasing local velocities especially around piers and abutments. Figure 6.3 shows the typical effects that are possible. Backwater profiles can further be increased by build-up of debris on the bridge.

The design of bridges is well covered in Section 5 of Austroads (1994). Hydraulic factors that should be considered in the design of bridges are:

- Flow type – tranquil or rapid.
- Freeboard.
- Total length – the total length between abutments must be adequate to avoid excessive constriction to flow and interruption with river fluvial processes (meandering and sediment transport). Furthermore the length must be sufficient to avoid “choking” of the flow. To avoid critical flow and the formation of a hydraulic jump (with concomitant erosion potential) the Froude number must not exceed 0.9.
- Clear span length – the size of debris to be passed will influence clear span length in addition to backwater effects and economic factors.
- Scour.
- Buoyancy – can be caused by air entrapment or debris under the superstructure, and reduction of the effective weight if inundated. Therefore, adequate anchorage at the piers and abutments should be provided to resist both horizontal and vertical forces.
- Hydrodynamic loading – from the direct action of stream flow on a bridge structure.

The design of bridges to avoid or reduce their effects is complex and professional advice should be sought.

These effects are reduced by:

(a) increasing the waterway area by increasing the bridge length and height above water level.

(b) decreasing the size and number of piers and improving their shape.

(c) aligning the bridge perpendicular to the flow direction.

(d) reducing the eccentricity factor of bridges in flood plains.

Backwater calculations are well described in Austroads (1994) to determine water level rises upstream of bridge sites.

6.4.4 Service crossings

Many types of services cross watercourses including stormwater, water supply, effluent, gas, power and communications (e.g. fibre optic cables). Every attempt should be made to plan service networks so that crossings may be attached to existing bridges. Where this is not feasible then the crossings will either have to be designed to the standards required for bridges or constructed under the streambed and below likely scour depth. Particular care is required with effluent crossings to avoid fracture and consequent stream pollution.

6.5 Other modifications

Other modifications include channel realignments, stream diversions, erosion control devices, stormwater detention dams, soil conservation dams and stormwater collection and conveyance systems. Each of these will modify the normal water surface and velocities in a manner similar to culverts and bridges.

The effects of channel realignments, stream diversions and erosion controls (rock-lined channels, drop structures) can be assessed in the same manner as the normal water surface was determined in Section 6.3 by applying the modified channel geometrics and roughness's. In drop structures the added complication of hydraulic jumps will require consideration.

Stormwater detention dams and soil conservation dams will usually incorporate discharge conduits and can be treated similarly to culverts under inlet control under deep embankments. These dams also require adequate spillways to overcome blockages of outlets and provide for high return period flows. Stormwater collection and conveyance systems are described in the Concrete Pipe Association Hydraulic Design Manual (1997) and will also be covered by Local Authority Design Guides.

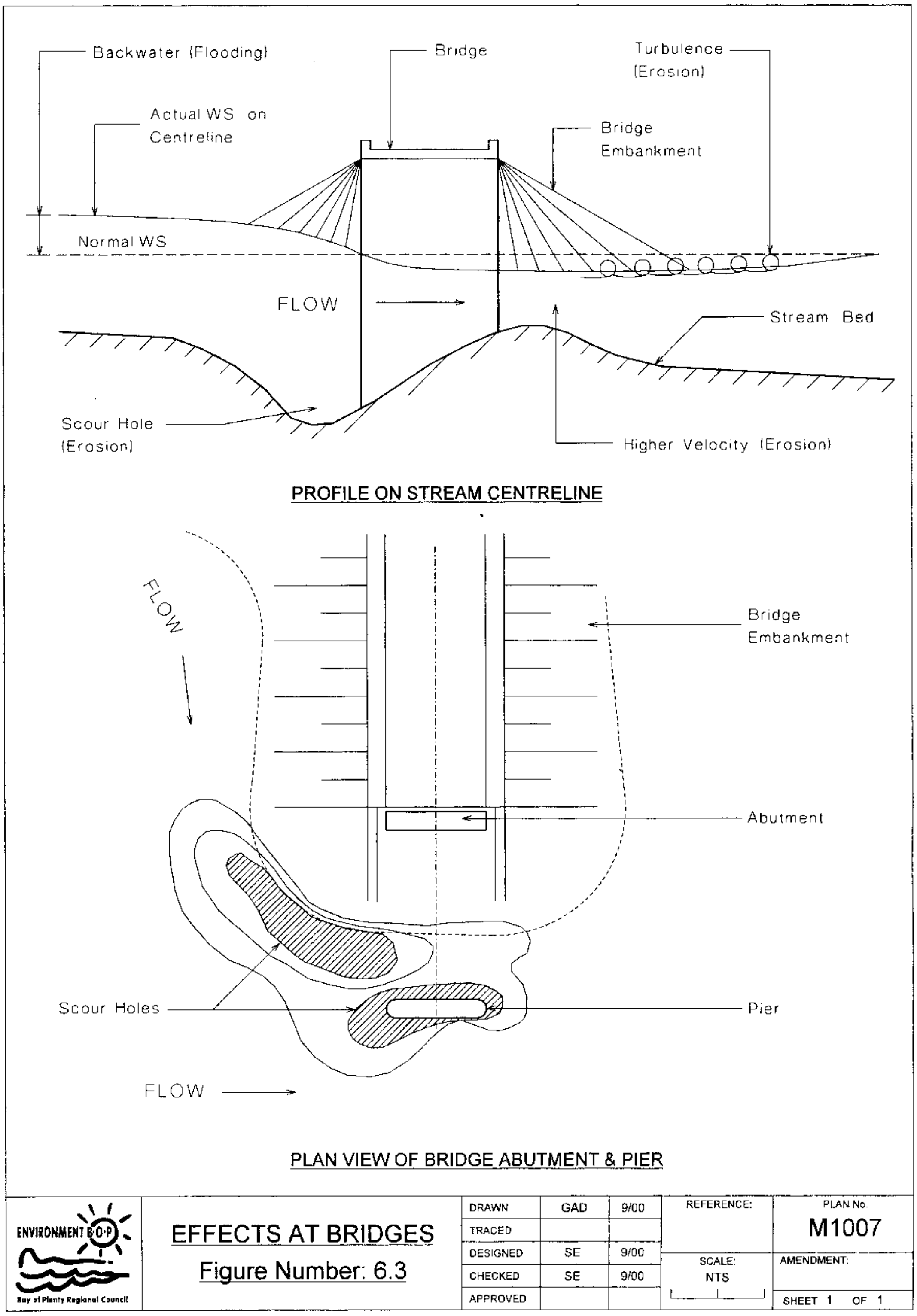


Figure 6.3 Effects at bridges

Part 7: Design details

7.1 Introduction

In addition to the general design procedures described in Section 6, there are design details that should be considered and implemented. The resolution of these will assist in the achievement of waterway modifications that avoid, remedy or mitigate the adverse effects listed in Section 1.

7.2 Culverts

The following adverse effects (in addition to raised upstream water levels) of culverts should be addressed and resolved using the techniques available in the references:

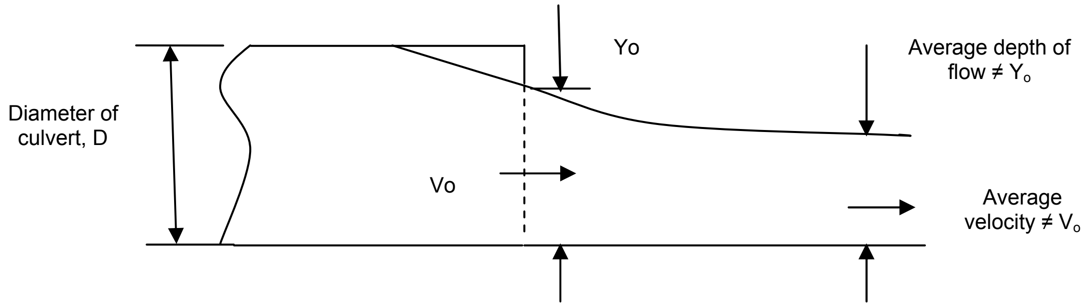
(a) Inlet scour – due to increased water velocities. Not as significant as outlet scour but should be considered and can easily be resolved by application of standard inlet designs. Will also improve the culvert's performance by reducing head losses and allow use of a smaller culvert or lower headwater. Use of wing walls, aprons, cut-off walls and embankment paving should be considered.

(b) Outlet scour – again due to increased water velocities and more destructive than inlet scour. Outlet protection should be considered to prevent erosion of the fill and adjacent channel and to prevent undermining of the culvert ends. Where necessary it can also combine to prevent seepage and piping through the bedding and backfill along the culvert barrel. Consideration should be given to the provision of wing walls, headwall, cut off and apron. Investigation of scour protection at similar culverts in the vicinity will provide valuable guidance. An important parameter in the selection of an appropriate energy dissipater is the Froude Number of the outlet flow. A Froude number less than unity (i.e.1) indicates flow is a fluvial motion (i.e. subcritical flow), and like a torrential flow motion (i.e. supercritical flow) when the ratio is greater than 1. The Froude number for culvert outlet discharges can be calculated as follows:

$Fr = V_o/(gY_o)^{0.5}$

Where Fr = Froude Number (dimensionless)

- V_o = velocity at culvert outlet (m/s)
- g = gravitational acceleration, 9.81m/s^2
- Y_o = brink depth at culvert outlet (m)



A range of Froude numbers normally associated with energy dissipaters are shown in the table below.

Fr	Dissipater
Less than 1.7	Simple apron structure, or flow expansion
Greater than 1.7 and less than 3	Rip-rap basin or horizontal roughness elements basis
Greater than 3	Hydraulic jump basin

Numerous technical publications exist that outline methods of reducing erosion at culvert outlets and the methods provided in Chapter 12 of the Council's Stormwater Guidelines are approved (2012). The Stormwater Guidelines cross-reference two recommended methods for designing erosion protection at culvert outlets. They are:

(a) Rip-rap basin and apron designs developed by US Department of Transportation (2006) in its technical publication titled Hydraulic Design of Energy Dissipaters for Culverts and Channels; and

(b) A simplified but more conservative approach for designing riprap aprons at culvert outlets. This simplified approach and method is outlined in the Council's Stormwater Guidelines in s12.4 (2012).

Streambed protection downstream of the culvert outlet can also be achieved by utilising the methods described in Section 7.5 below.

(a) Debris collection leading to failure of the drainage structure and overtopping and failure of the embankment. Where debris is a potential problem consideration should be given to the installation of debris control structures (Austroads Section 7.7) i.e. smooth inlets, increase culvert size, relief culvert, grills and gratings. These structures require regular maintenance to avoid blocking and consequent flooding.

(b) Embankment failure. Precautionary design and construction details should be used to prevent embankment failure while water is ponding upstream. These are described in Section 7.4 below.

(c) Siltation.

(d) Barriers to fish passage. High velocities and smooth inverts create barriers to fish. See Section 7.6 below.

(e) Safety – installation of grills at upstream ends.

7.3 Bridges

The following adverse effects, in addition to raised upstream water levels, of bridges should be addressed and resolved using the techniques available in the References:

(a) Scour: There are four types that require consideration, these being river degradation, general, local and constriction. Different materials scour at different rates. Loose granular soils are easily eroded while cohesive soils are more resistant although ultimate scour depths can be the same. There is ample discussion of these in Austroads (1994 Chapter 6). It is important to note that the problem of scour at bridges is complex hence appropriate factors of safety should be applied. The following points are made about each type of scour:

(i) River degradation – of the whole river system, can be brought about by the construction of a dam upstream or close downstream; gravel extraction; afforestation or deforestation of a large portion of the catchment.

(ii) General scour – due to river morphology, includes the unevenness of the natural river channel; deepening at bends; general bed motion during floods.

(iii) Constriction scour – should be calculated if the bridge works constrict the waterway area by more than 10%. Constriction can be caused by both piers and narrowing between abutments.

(iv) Local scour – at piers, abutments and embankments. If the upstream Froude number might approach unity, design of piers should be approached with extreme caution. Multi-piers can attract debris.

The design of scour protection is covered in Austroads (1994) Section 6.3 and should be undertaken bearing in mind the importance of the crossing and the consequences of failure. The following methods should be considered:

- abutments should be protected by the use of rock rip-rap (see Section 7.5 below).
- align piers with the direction of flood flows.
- use round piers and note that streamlined piers decrease turbulence and reduce the potential for debris accumulation.
- apply conservative factors of safety to foundation depths.

(b) Debris collection leading to increased backwater and flooding and structure failure: The minimum waterway freeboard clearances stated in Section 4 should be applied with some judgement as to their adequacy. Span length between piers or abutments should also be maximised to reduce this problem.

(c) Span Width: The minimum bridge span should be at least 60% of the “flow dominant” meander width. Meander width can be calculated using the Lacey formula as follows:

$W = 4.85 \times Q^{0.5}$

Where:

W = Flow Dominant (Lacey) meander width
Q = Mean annual flood

7.4 Embankments and small dams

For embankments and small dams, as defined in Section 4, the following practices should be considered:

(a) Preparation of the foundation surface to remove loose and permeable material.

(b) Use of low permeability soils in the embankment (clayey or silty gravel, clayey or silty sand, clay or silt).

(c) Batter slopes of three horizontal to one vertical (3H: IV).

(d) Minimum crest width of 4.0 m.

(e) Constructed in layers, 200 mm loose thickness, compacted to 95% of maximum dry density.

(f) Allowance for secondary flow paths (see Section 8).

(g) Use of filter collars and filter drains around conduits that pass through the embankment. Use of anti-seep collars is not permitted.

7.5 Channel erosion protection

Traditionally, structural integrity has been the single most important design criteria for bank protection works. However channel erosion protection works should reflect the full range of values including ecology, landscape, recreation, heritage, culture and drainage.

If channel erosion is required then a hierarchical approach to acceptable works should be adopted, which is outlined below in descending order of preference:

(a) bank re-grading

(b) waterway structural lining

Bank erosion treatments should involve the minimum amount of engineering intervention necessary. This approach is likely to be more sustainable from an environmental and economic perspective.

Alternative softer treatments like reinforced earth and geotextiles are recommended as they are more compatible with other values such as ecology and landscape. On exceptionally soft substrates the use of geotextiles should be considered.

In some locations, rock- lined channels provide an adequate solution for erosion protection. The rock lining could either be placed rip-rap, rock mattresses or flexible mats. Where necessary a permeable geotextile filter should be placed between the rock lining and the channel.

7.5.1 Bank re-grading

Where waterway capacity is more than adequate, planting on a slumped surface may provide sufficient stability to channel banks.

Reshaping the bank profile to a more gradual slope may be sufficient to stabilise a bank. A wide waterway corridor sufficient to allow for the gently sloping banks is required. For bank stability, firm silts can stand as steep as ¼ H: 1V but soft or loose material may need to be flatter than 3H: 1V. Banks steeper than 4H: 1V are difficult to machine mow although it is preferable to vegetate the banks with low maintenance planting.

7.5.2 Waterway structural lining

Waterway lining (concrete, timber etc.) is discouraged because it commits future generations to expensive replacement, encourages inappropriate development and filling located to close to the edge of the waterways and cannot support an ecologically diverse environment.

Structural lining commonly used and recommended in the region includes rock lining (i.e. rip-rap), rock mattresses (e.g. Gabions, Reno Mattresses) or flexible mats (e.g. Enkamat).

(a) Rock lined channels

In some locations, rock-lined channels provide an adequate solution for erosion protection. The rock lining could be placed rip-rap or rock mattresses. Permeable geotextile filter fabric should be placed between the rock lining and the channel. Grouted rip-rap will not be accepted.

(i) Low to moderate velocities

Where flow velocities are less than 4.5 m/s and batter slopes less than 1.5H: 1V, the class and thickness of rock to be used as placed rip-rap can be obtained from Tables 7.1 and 7.2. Note that velocities less than 2 m/s can still be erosive to stream banks – refer to Section 7.2 and Table 7.4 of the BoPRC Draft Stormwater Management Guidelines for the Bay of Plenty Region, for further details

Table 7.1 Design of rock slope protection (flow velocity<4.5 m/s)

Velocity (m/s)	**Class of Rock Protection W_c (tonne)**	**Chapter Thickness, *T* (m)**
<2	None	----
2.0-2.6	Facing	0.6
2.6-2.9	Light	0.85
2.9-3.9	¼	1.15
3.9-4.5	½	1.35

Table 7.2 Standard classes of rock slope protection (flow velocity<4.5m/s)

Rock Class	**Rock size* (m)**	**Rock mass (kg)**	**Minimum percentage of rock larger than**
Facing	0.40	100	0
	0.30	35	50
	0.15	2.5	90
Light	0.55	250	0
	0.40	100	50
	0.20	10	90
¼ tonne	0.75	500	0
	0.55	250	50
	0.30	35	90
½ tonne	0.90	1000	0
	0.70	450	50
	0.40	100	90

**Assuming a specific gravity of 2.65, spherical shape and batter slope of 1.5 horizontal to 1 vertical.*

Rock class, chapter thickness, rock mass and grading data provided in Tables 7.1 and 7.2 are derived from publications produced by California Department of Public Works (1970) and AusRoads (1994). These two publications can be used to design rip-rap placed downstream of culvert outlets where the mass of the critical stone, considered to represent the median stone diameter, d50 is based on equation below. It should be noted that the rock diameter referred to in rock sizing is the

$$W_{CRIT} = \frac{0.01135 v^6 \gamma}{\sin^3(\rho - \alpha) \times (\gamma - 1)^3}$$

geometric mean of the three principal axis dimensions.

Mass of Critical Stone	**W_{CRIT}**	**kg**
Factored Velocity	v	m/s
Specific Gravity of the Rock	γ	tonnes/m^3
Angle of Repose for material	ρ	deg
Embankment Slope	α	deg

Rock mattresses (Gabions, Renos) and flexible mats are proprietary products covered by manufacturers technical literature.

Note: – In determining the velocity the mean channel velocity in the design event should be modified as follows:

- Normal bends – Increase velocity by factor of 4/3.
- Very sharp bends and groyne-heads – Increase velocity by factor of 1.5.
- Culvert exits – Increase velocity by a factor of at least 1.1; unless hydraulic calculations indicate a greater increase – this might apply at unsubmerged outlets.

(i) Rock-lined channels - high velocities.

When flow velocity and depth comes close to critical flow then the velocity may be sufficient to create erosion problems, requiring protection.

For flows of greater than 4.5 m/s or batter slopes steeper than 1.5H: 1V then Figure 7.1 should be used in conjunction with more specific design such as the Californian Department of Transportation (1970) and Simons (1977).

Figure 7.1 is based on the Isbash formula and relies on close packed rock units mutually supporting each other and is satisfactory provided the extent of the rip-rap mat is carried clear of the high velocity zone (MOWD, 1978).

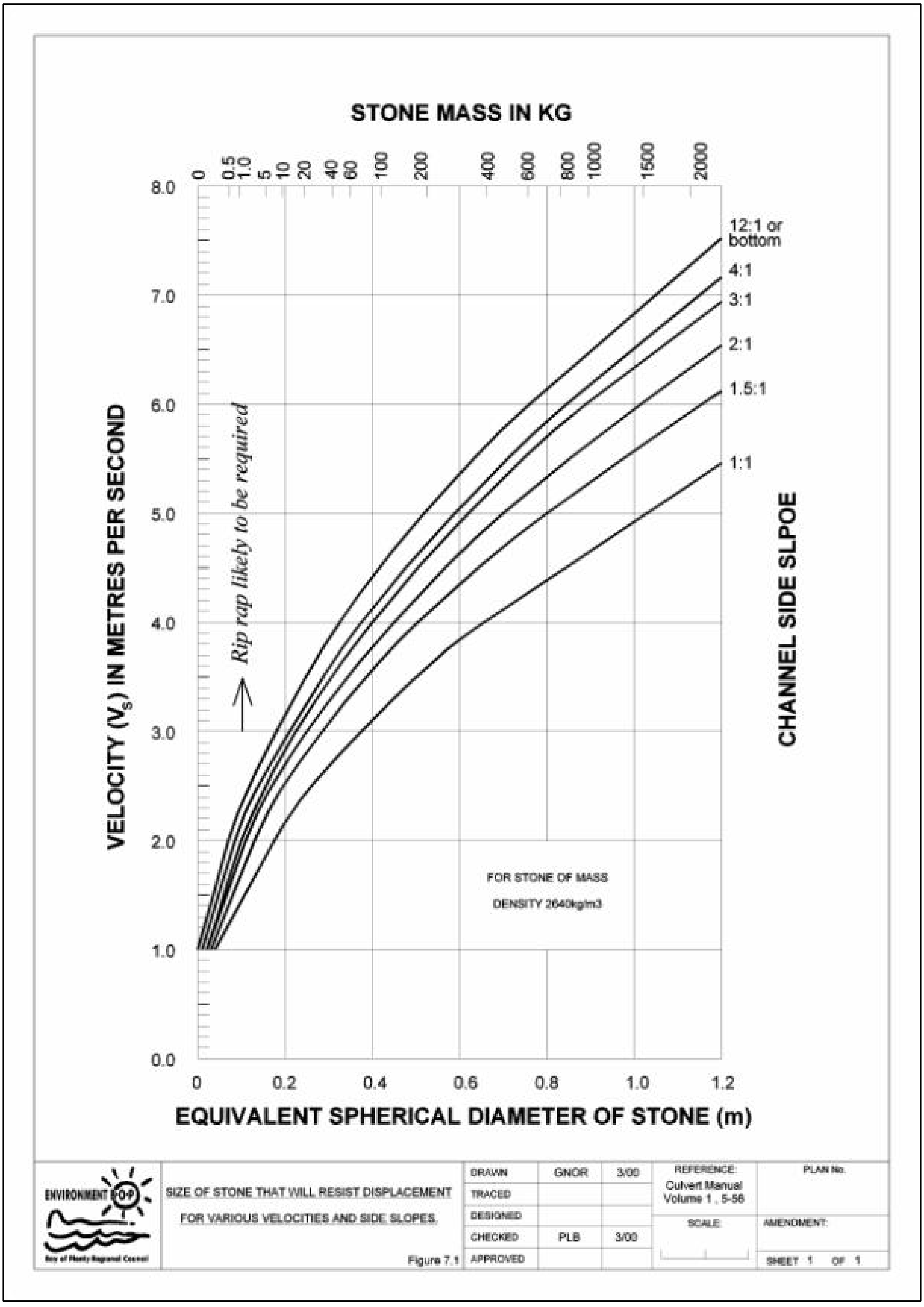


Figure 7.1   Rip-rap sizing diagram to determine d50

7.6 Fish passage at culverts

7.6.1 What this section covers

The Regional Water and Land Plan (RWLP, 2008) sets out the requirements for installing and replacing culverts. Fish passage is a condition of practically any resource consent involving culverts regardless of their activity status in the RWLP.

The intention of this section is to provide guidance on fish passage at culverts. It does not discuss methods for increasing the habitat value of culverts. Culverts provide very poor 'standalone habitat' for fish compared with open stream channel. As such the RWLP recognises culverts as "stream crossing structures" rather than as structures that can be used to infill streams.

7.6.2 The issue with culverts

Poorly situated and sized culverts can create significant barriers for fish and other aquatic fauna and, in doing so completely prevent fish from reaching upstream habitat. Culverts can create fish barriers in two ways:

(i) Flow velocity barriers

New Zealand freshwater fish species fish are typically small and it is most often juvenile forms that need to migrate through culverts to reach natal habitat. Sustained high velocities inside a culvert will quickly cause a fish to tire and be ejected from a culvert. Velocity barriers most often occur when a culvert is too small (relative to the stream flow) or when the culvert gradient is too steep. Fish passage may also be disrupted by turbulence at higher flows or that resulting from poorly designed baffles or culvert junctions.

(ii) Physical barriers

Physical barriers most often result where high exit velocities have caused scour below a culvert outlet causing the culvert to become perched and/or to project. Even native fish species with 'climbing' ability will be prevented from reaching a culvert outlet if it is both perched and projecting.

The most favourable road crossings are generally those that closely imitate natural stream flow conditions and stream channel conditions. Bridges and arch culverts are preferred over round culverts because they maintain the streambed intact and so provide the best opportunity for simulating natural stream flow and bed conditions. Box culverts are the next most preferred because, with the invert buried, they provide similar fish passage conditions to bridges and arches. Round culverts are the least preferred but can be configured and sized to provide adequate fish passage.

Please note that culverts can become impaired by debris or, when subject to significant storm events, can fail outright such that they no longer provide appropriate fish passage. While a resource consent is not required to maintain culverts[9], it is the responsibility of culvert owners to ensure the passage of fish is maintained or restored. Failing to do so may result in enforcement action being taken.

[9] Although you may to replace a culvert

7.6.3 **These Guidelines**

NIWA and DOC have published a document titled '*Fish Passage at Culverts' (Boubee et al, 1999)* that reviews how culverts can impact on migrating indigenous and exotic freshwater fish. The publication also discusses the applicability of several fish passage solutions within a New Zealand setting and using swimming performance specifications for New Zealand freshwater fish species. These guidelines borrow heavily from the '*Fish Passage at Culverts'* publication, listing the most important requirements for fish passage at culverts (these are reflected in the RWLP). Suggested approaches and figures with explanatory notes are added by the Bay of Plenty Regional Council in *italics.* However, you should refer also to the NIWA/DOC 'Fish Passage at Culverts' publication because it provides technical specifications and practicable (often illustrated) examples on how culverts should be configured to best provide fish passage.

For every new culvert or culvert replacement the following recommendations should be adopted as together, they will help you achieve fish passage through your culvert:

(i) The culvert should be positioned so that its gradient and alignment are the same as the stream.

(ii) Drop structures should only be used as temporary solutions and in ephemeral streams.

(iii) The culvert width should be equal to or greater than the average streambed width at the elevation the culvert intersects the streambed.

(iv) The culvert invert should be set well below the current streambed (minimum of 20% of culvert diameter at downstream end).

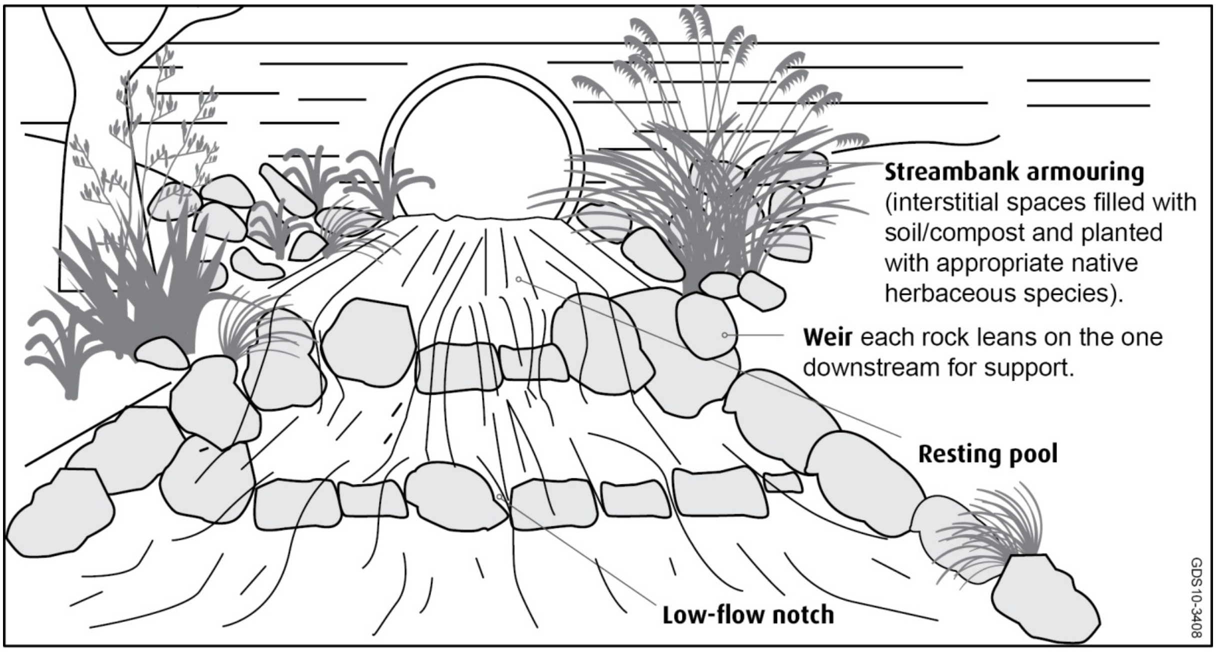
This will allow a modicum of natural bed material to accumulate in the bottom (invert) of the culvert so improving fish passage conditions for fish. The final culvert size will need to account for the fact 20% of the internal volume of the culvert will be buried.

Please note the RWLP requires that the culvert invert shall be installed a minimum of 0.1 m below the level of the bed of a river, stream, or lake. It would be preferable as an approach if instead 20% of culvert diameter was buried at the downstream end. This is because the 20% depth requirement is proportional to the size of the culvert unlike the RWLP rule where one depth standard applies across all culvert sizes and stream types. That is, at the very least you should ensure the invert is buried a minimum of 0.1 m below the streambed level but <u>ideally</u> one should embed 20% of the culvert diameter.

(v) Weirs should be notched and impermeable so that a pathway over the weir is present at all flows.

(vi) Bed material should be assessed to determine the potential for downstream erosion. If erosion is likely, a weir, or series of weirs, should be provided downstream of the outlet. Such weirs could also provide pools that serve as resting areas, reduce culvert velocities by backwatering, and eliminate elevated outlets.

Care should be taken to avoid installing weirs in a way that creates additional fish barriers downstream of the culvert. The weirs themselves could be constructed from something as simple as rocks placed carefully across the wetted channel (Refer Figure 7.2). Rocks would obviously need to be large enough to prevent dislodgement during storm events and placed so that fish can pick their way around the rocks or through a v-notch (Refer Figure 7.2).



A succession of weirs helps raise the water level so it creates an impoundment through the culvert.

Figure 7.2 Impermeable weirs suitable for fish passage

It is better to install a series of low weirs beginning well downstream of the culvert. This allows one to gradually increase the water level (at the same time creating resting pools), so that by the time the culvert is reached further upstream, the invert is well and truly submerged. Used in this way (i.e. so that the weir causes water to back-up through the bottom of the culvert), weirs are sometimes called tail water controls (Refer Figure 7.3). Resulting slow flow zones along the bottom of the culvert can greatly assist fish passage particularly if streambed material is also allowed to accumulate in the culvert invert. By increasing bed roughness, streambed material will slow flows further (Refer Figure 7.3).

Under no circumstances should reno mattresses and/or gabion baskets be installed in the beds of streams if there is any likelihood of:

- Flows percolating through rather than over these structures. A lack of surface flows could easily prevent fish passage particularly during summer low flows.
- Abrasive bedloads causing structural failure. In high flow environments, high bedloads can easily abrade reno mattresses and/or gabion baskets causing them to fail thus preventing fish passage.

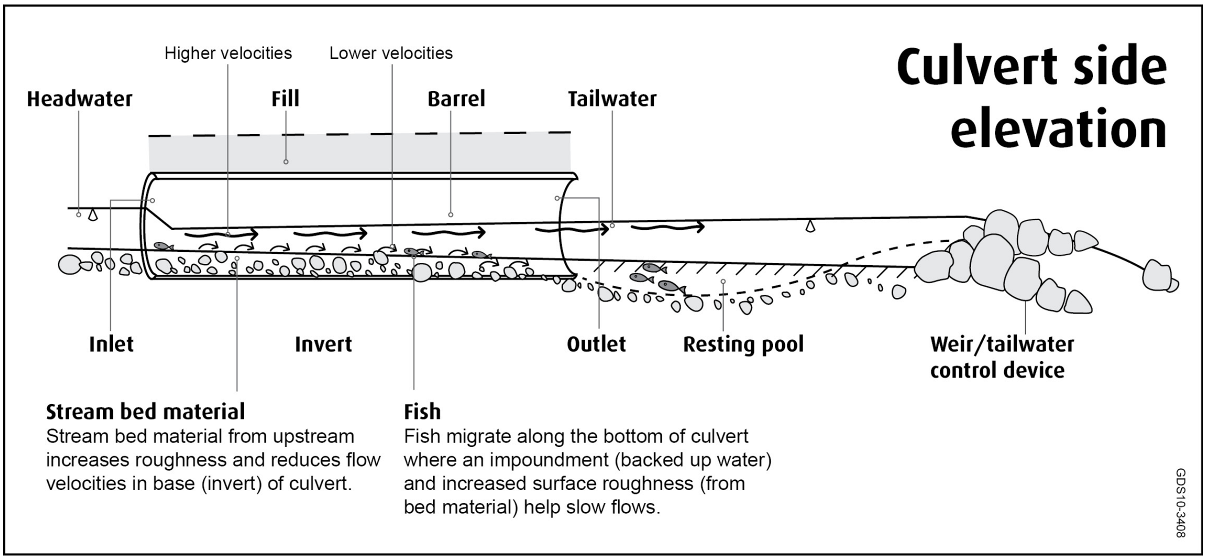


Figure 7.3 Culvert details suitable for fish passage

(i) Armouring of the banks at the outlet and inlet may be required to prevent erosion.

It is preferable that the armouring treatment chosen is one that can support vegetation. If for example, the preference is for rock rip-rap to be used, then it should include a matrix of rock and soil (Refer Figure 7.2). This would ideally then be hydro seeded with appropriate herbaceous species down to water level. As well as creating substantial benefits for aquatic fauna, the roots of established native and exotic grasses will help knit the whole structure together, increasing its lifespan.

(ii) The average barrel velocity should ideally be below 0.3 m/s; where this cannot be achieved, a 50-100 mm zone should be provided on either side of the culvert with velocities below 0.3 m/s.

Ultimately the goal should be to imitate flow conditions occurring upstream and downstream of the culvert. That is, it is acceptable to default to velocities exceeding 0.3 m/s, if these exceedances occur naturally (at base flows) upstream and downstream of the culvert. Under no circumstances should barrel velocities be greater than velocities occurring in stream channel up or downstream of the culvert at normal flows.

(iii) Where average barrel velocities are greater than 0.3 m/s [up or downstream of the culvert], smooth culverts provide a more suitable surface for climbing indigenous New Zealand species than ribbed ones (note, however, that ribbed culverts of the PolyfoTM type are useful for reducing barrel velocities while still providing resting areas for climbing species).

As stated above, it is recommended that 20% of the culvert be buried beneath the streambed. Only in exceptional instances (i.e. if allowed by consent) should a culvert be installed so that its invert remains exposed. If a box culvert is proposed and the invert cannot be buried, the preference is for the invert to be canted slightly to create a range of flow velocities and depths across the invert.

(iv) Spoilers are useful for reducing barrel velocities as well as for providing resting areas. Such structures should only be installed where they will not cause obstruction of the culvert through accumulation of debris, and where site and engineering restrictions leave no other options.

It is generally preferable to slow flows within a culvert using a tailwater control. These are situated beyond the culvert and so do no impede floodwater conveyance or accumulate debris. However, tailwater controls will not be feasible for streams where the bed drops steeply beyond the culvert outlet. In these instances, spoilers or baffles may be the most practicable solution. It is desirable to use spoilers and baffles in which the turbulence characteristics are known and can be demonstrated by their manufacturer. It may be acceptable for the spoilers/baffles to produce turbulence at high end flows, if the effect is to substantially increase fish passage at all other flows.

(v) Baffles are useful to ease passage of salmonids; but to ensure an uninterrupted pathway for indigenous species, they should not cut across the entire floor of the culvert.

See also above. Baffles work by disrupting / disturbing laminar flows through a culvert and, in that respect alone, baffles will generally be an improvement over an exposed culvert floor. However, the sizing and spacing of baffles may be critical for some species. I.e. baffles designed for salmonids may disadvantage smaller native species. Furthermore, some baffles may perform sub-optimally under certain flow conditions where for example, turbulence may be created during high flows (see also (ix) above). Baffles may improve fish passage conditions in culverts simply by retaining a modicum of stream bed material so simulating a streambed environment. There is some benefit in including gaps between banks of baffles as these gaps may develop into resting pools for migrating fish.

(vi) Where low flows (and therefore shallow water depths) are a feature of the site, the apron, weir, or barrel floor (for large and box culverts) should be dished or sloped to give greater water depth.

Please note that unless configured sensitively, aprons can create significant physical barriers to mainly non-climbing fish species. Scour can occur below aprons creating perches similar to those found below poorly configured culvert outlets. Installing a tailwater control below an apron may, by creating a pool below the apron, help dissipate flow energy and eliminate potential for scour and at the same time elevate the water level slightly. Aprons themselves can also be modified to include tail water controls to slow flows through culverts.

(vii) All junctions at the leading end of, and in between, the culvert components should be rounded to allow climbing species to pass.

(viii) Where the flow regime of the stream permits, in order to ensure the maintenance of a wetted margin the water depth should be no greater than 45% of the culvert height for the majority of the upstream migration period.

(ix) Drop structures (often used in conjunction with scruffy domes) are being used increasingly as energy dissipaters and as collection points for bed material at culvert intakes. The sumps and discontinuous piping that are characteristic of these drop structures provide very poor fish passage in much the same way as projecting and/or perched culvert outlets do. As such Bay of Plenty Regional Council requests that drop structures only be used where it can be shown with absolute certainty that there are no access requirements for fish and/or that no useable upstream fish habitat exists.

Users are advised to refer to this document further to gain a fuller understanding of its basis, applicability and limitations.

Part 8: Flows in excess of design flow

8.1 Introduction

In most of the situations for which these guidelines are intended to cover there will be an appreciable risk that the design flow will be exceeded during the lifetime of the waterway modification. For instance a culvert under an Access Track will likely be subjected to a flow exceeding the 10-year return period design flood sometime during its lifetime. It must therefore be accepted by all interested parties that the structure could fail and cause adverse effects downstream (sediment deposition, erosion etc.)

The purpose of this chapter is to provide some guidance on how to reduce the likelihood of these effects by the application of precautionary design measures.

8.2 Secondary flow paths

Secondary flow paths are routes upstream of the modification along which water will flow if water levels get high enough. Identification of secondary flow paths is recommended and they should be shown on the plans and cross-sections provided. This is where the stream or river will flow if the waterway modification is too small or has become blocked by debris and upstream water levels rise above the design headwater. This identification is a requirement in any situation where water may enter a building (see Section 4.8).

The secondary flow path might be a saddle in the catchment over into an adjoining catchment, or the overtopping of natural ground around the structure, or the overtopping of the structure itself. For major roads special floodway design might be appropriate (Austroads Section 9).

In all cases it is desirable to have the secondary flow path in non-erodible ground, ideally rock, but for lower velocity flows well- vegetated slopes might be suitable. Where high velocities occur, the slope and crest of the path should be protected in a similar manner as rock lined channels.

Where overtopping of small embankments is likely and no flow path exists in natural ground, the embankment can be designed for overtopping to some degree. This first of all requires adherence to the recommended practices in Section 7.4. In addition the crest of the embankment and the downstream slope can be protected with grass or dumped rock. The use of anything more substantial should be investigated and the cost weighed up against the adverse effects of failure and the resultant structural damage.

8.3 High velocities

Higher than design flows will invariably produce higher than design velocities and hence depth and length of scour and erosion patterns. The design of protective measures should therefore be conservative and should consider the inaccuracies and the ranges inherent in determining design flows. Construction of additional outlet scour protection on culverts should be carefully considered.

References

Rainfall Intensities

National Institute of Water and Atmospheric Research Limited (NIWA), HIRDS Software, 1993, PO Box 3047, Wellington.

Opus International Consultants Limited, High Intensity Rainfall in the Tauranga - Waihi – Te Puke Area, September 2000.

Tauranga City Council, Tauranga City High Intensity Rainfall Update, October 2005, by OPUS International Consultants, 10 October 2005.

Tauranga City Council, Tauranga City Rainfall Temporal Patterns, March 2006, by OPUS International Consultants, 13 March 2006.

Environment Bay of Plenty, Tauranga City Temporal Rainfall using the Average Variability Method, May 2006, by OPUS International Consultants, 10 May 2006.

Calculation of design flow

Environment B·O·P Hydro-Meteorological Data Summaries, Report to December 1995, McLarin and Stringfellow, Environmental Report No. 97/15, 31 July 1997.

McKerchar, A I; Pearson C P (2000), Factors Causing Flooding to be New Zealand's Number One Hazard. Tephra (in press).

National Institute of Water and Atmospheric Research Limited, Review of Probability Estimates of the Bay of Plenty July 1998 Floods, NIWA Client Report No. CHC98/61 (BOP90501), October 1998.

Pearson and McKerchar, Flood Frequency in New Zealand, Publication No. 20 of the Hydrology Centre, DSIR, Christchurch 1989.

Technical Memorandum 61, A Method of Estimating Design Peak Discharge, MWD, 1980.

Design Standards

Department of Building and Housing, Building Regulations, Schedule 1, Document E1 Surface Water, BIA 1992.

Department of Building and Housing, Compliance Document for New Zealand Building Code: Clause E1 Surface Water, 10 October 2011.

New Zealand Standard, NZS4404:2010: Land Development and Subdivision Infrastructure, Incorporating Amendment No.1

NZTA, Bridge Manual, Second Edition 2003 (includes amendments June 2004, September 2004 and July 2005), ISBN 0-478-04132-2, July 2005.

Transfund NZ, Integrated Stormwater Management Guidelines for the New Zealand Roading Network. Transfund New Zealand Research Report No. 260. by Kouvelis, B. Armstrong, W. (Editors), ISBN 0-478-25372-9 and ISSN 1174-0574, 2004.

General Hydraulic Design

Chow, Open-Channel Hydraulics, 1959.

Chow, Maidment and Mays, Applied Hydrology, 1988.

Henderson, Open Channel Flow, 1966.

Hicks, D M and Mason P D 1998, Roughness Characteristics of New Zealand Rivers, Water Resources Survey, DSIR Marine.

Maidment, Handbook of Hydrology,1993.

Viessman, W; Lewis, G.L. 1996, Introduction to Hydrology, 4th edition, Addison-Wesley, Boston.

Applied hydraulic design manuals, guidelines and codes

Austroads Waterway Design – A Guide to the Hydraulic Design of Bridges, Culverts and Floodways 1994.

BOPRC, Stormwater Management Guidelines for the Bay of Plenty Region, April 2012, by Aqua Terra International Ltd, 2012.

Christchurch City Council, Waterways, Wetlands and Drainage Guide, 24 August 2011

Concrete Pipe Association of Australasia (Humes Drainage Products Technical Manual), Hydraulic Design Manual NZ Edition, December 1997.

Environment BOP, Erosion Protection Works Guidelines for Tauranga Harbour, EBOP Guidelines No. 2002/02, by P H Herbst, AA Schuler and A Lawrie, 2002.

Environment BOP, Erosion and Sediment Control Guidelines for Land Disturbing Activities, EBOP Guidelines No. 2010/01, by D Clemens and Ridley Dunphy Environmental Ltd, June 2010.

Environment BOP, Erosion and Sediment Control Guidelines for Forestry Operations, EBOP Guidelines No. 2000/01, July 2000.

Ministry of Works and Development (MOWD), Culvert Manual, Volume 1, Civil Division Publication, CDP 706/A, Ministry of Works and Development, August 1978.

Ministry of Works and Development, Code of Practice for the Design of Bridge Waterways, Civil Division Publication 705/C, November 1979.

Tauranga City Council, Draft Stormwater Management Guidelines, July 2012.

Energy dissipation and rip-rap design

Sediment Transport Technology, D B Simons and F Senturk, Water Resources Publication, Colorado, 1977.

State of California Department of Public Works, Bank and Shore Protection in California Highway Practice, 1970.

US Department of Transportation, Hydraulic Design of Energy Dissipaters for Culverts and Channels, Hydraulic Circular No. 14 (2006).

Embankments and dams

Environment BOP, Guidelines for the Design, Construction, maintenance and safety of small flood detention dams, EBOP Guideline No. 2006/01, By S Everitt, December 2006.

NZ Society on Large Dams (NZSOLD) Dam Safety Guidelines November 2000.

US Bureau of Reclamations Design of Small Dams, 1960.

Fish passage

Boubee, J. Jowett, I. Nichols, S and Williams, E 1999, Fish Passage at Culverts, A Review with Possible Solutions for New Zealand Indigenous Species, A NIWA/DOC Publication ISBN 0-478-21871-0, Pages 58-59, December 1999.

Future sea levels

Environment B·O·P, Cyclones Fergus and Drena Storm Surge Report on the Magnitude of Storm Surges Recorded and Implications for Design Maximum Sea Levels, 1997.

Environment B·O·P. Chapter 32 Analysis, Bay of Plenty Draft Coastal Hazard Policy, CEARS, 1993.

Ministry for the Environment, Coastal Hazards and Climate Change: A Guidance Manual for Local Government in New Zealand Second Edition July 2008.

Ministry for the Environment, Preparing for Climate Change. A Guide for Local Government in New Zealand, Publication No. 891, July 2008.

NIWA, Extreme Sea Levels on the Mt Maunganui Shoreline (Moturiki Island), 1997, Environment B·O·P.

Tonkin & Taylor Limited, Tauranga District Council Storm Surge Inundation Study for Tauranga Harbour, Tauranga District Council, December 1999.

Regional Council Plans, Strategy and Policy Documents

Environment BOP, Operative Regional Water and Land Plan, Strategic Policy Publication No. 2008/06, ISSN 1176 4112, December 2008.

Environment BOP, Stormwater Strategy for the Bay of Plenty Region, Environmental Publication No. 2005/20, ISSN 1175 9372, October 2005.

Appendices

Appendix 1 – The erosion hazard zone

The following definition of the Erosion Hazard Zone is copied from the definitions section of the Regional Water and Land Plan (BOPRC 1 December 2008).

Erosion hazard zone – Land that has very severe to extreme erosion hazards. For the purposes of rules in Section 9.2 of this plan, the erosion hazard zone is:

(a) Any sand dune country; excluding sand dune country within urban areas or already developed sub-divisions that are on land between 50-150 m from the coastal marine area.

(b) Any land in the upper Rangitaiki River catchment above the confluence of the Otangimoana Stream and Rangitaiki River, including the Otamatea River catchment, in the following areas:

  (i) On the margins of erosion susceptible permanent streams and rivers; or

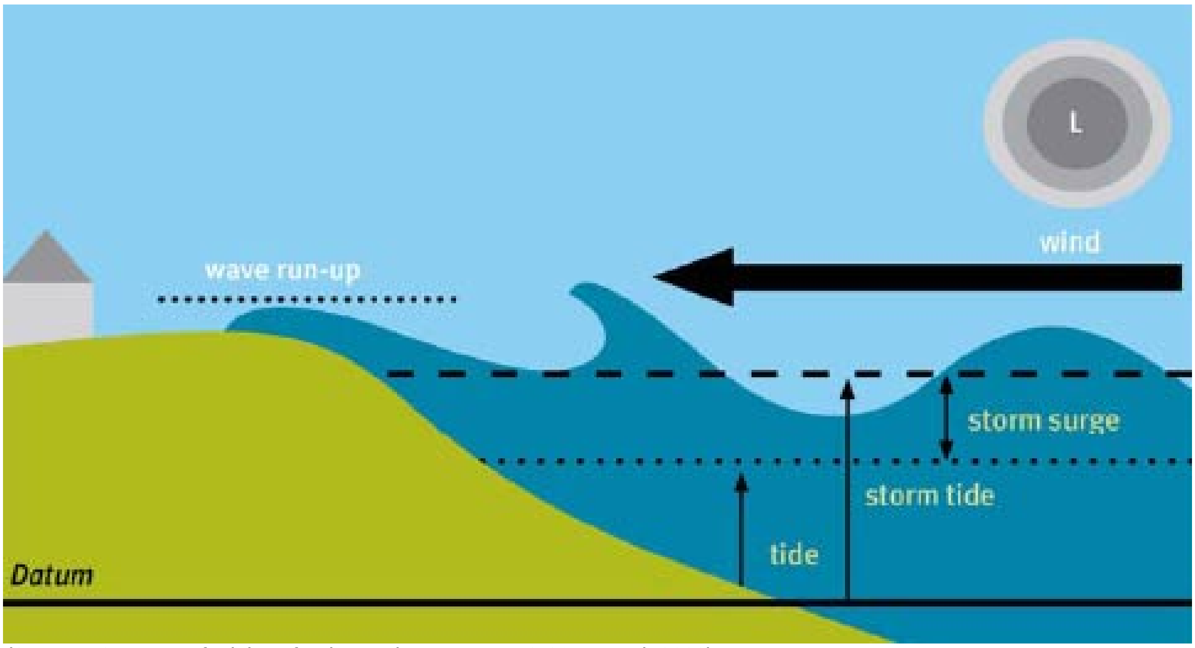
  (ii) In the beds and margins of ephemeral flow paths; or

  (iii) On steep terrace edges; as shown in Bay of Plenty Regional Council Plan Series M1009[1].

Note: The photomap plan series M1009 prepared by Bay of Plenty Regional Council at a scale of 1:25,000 shows the location of the beds and margins of the relevant land areas and ephemeral flow paths that are covered by definition points (b) (i) to (iii). These are the definitive maps used to assess compliance. Copies of these maps are available from or may be viewed at any Bay of Plenty Regional Council office.

Appendix 2 – Storm surge

Sea levels stated in Table 4.3 include storm tides and it's components are shown in the figure below. Storm tide represents the temporary increase in sea level offshore of the wave breaker zone. It comprises the mean sea level, the predicted tide at the time of the event and the storm surge height.



(Source : Courtesy of Ministry for the Environment, INFO 367, March 2009)

Mean sea level

The mean sea level is influenced by longer term climate fluctuations relating to seasonal effects namely the El Nino southern oscillation and Interdecadal Pacific Oscillation (IPO).

Predicted astonomical tide

At any given time there is a predicted tide level above a datum (e.g. chart datum or local vertical datum). The tide oscillates about the mean level of the sea. Tides used in Table 4.3 are the mean high water (MHW) level – **not** the mean high water spring (MHWS) level. Tides range between 0.76 m – and 0.89 m which may be further adjusted for differential levels within harbours and estuaries.

Storm surge

Storm surge is the increase in the regional ocean level (excluding the effects of waves). It arises from low barometric pressure (known as the inverse barometric effect) and winds blowing either on shore or alongshore (known as wind stress or wind set-up).

Wave run-up (swash)

At the shoreline, the maximum vertical elevation reached by the sea is a combination of wave set-up that is induced landward of the wave breaking zone and wave run-up (or swash). These act on top of the storm-tide level.

Wave run-up is excluded from values stated in Table 4.3. Wave run-up is highly variable even over a short length of beach, the beach slope, the backshore features and presence of any coastal defence structure.

Glossary of Terms

Average Recurrence Interval (ARI) – The long-term average number of years between the occurrence of a flood as big as (or larger than) the selected event. For example, floods with a discharge as great as (or greater than) the 20-year ARI design flood will occur on average once every 20-years. ARI is another way of expressing the likelihood of occurrence of a flood event. (see also annual exceedance probability).

Annual Exceedance Probability (AEP) – a statistical measurement of the annual chances (in %) of a flow of a specified size being equal or exceeded.

Cohesive Soil – a sticky soil, such as clay or silt; its shear strength equals about half its unconfined compressive strength.

Cumec – A cumec measures water flow. One cumec equals one cubic metres of water passing a given point every second (1 m^3/sec).

Development – Erecting a building, carrying out excavations, using land or a building, or subdividing land. *Infill* development refers to developing vacant blocks of land that are generally surrounded by developed properties. Greenfield development refers to developing properties in previously underdeveloped areas, e.g. the urban subdivision of an area previously used for rural purposes (see non-structural measures).

Rip-rap – Rock specifically designed and placed against water retaining banks in order to protect the slope from erosion.

Runoff – The amount of rainfall from a catchment that actually ends up as flowing water in the river or creek stage.

Time of concentration – the time taken for surface water run-off from the furthest point (in time) of the catchment to reach the design point.