Guide to Road Design Part 5: Drainage – General and Hydrology Considerations

Austroads
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Abstract
The Guide to Road Design Part 5: Drainage – General and Hydrology Considerations provides road designers and other practitioners with guidance on the design of drainage systems. This Guide needs to be used in conjunction with the other two Parts of the Guide to Road Design that relate to drainage design:

- Part 5A: Drainage – Road Surface, Network, Basins and Subsurface.
- Part 5B: Drainage – Open Channels, Culverts and Floodways.

This Guide provides information on the elements that need to be considered in the design of a drainage system. Guidance is provided on the safety aspects of stormwater flows, environmental considerations and water sensitive treatments within a drainage system. Drainage considerations are outlined covering the determination of the flood immunity, freeboard to be used for the design, types of structures, and operational and maintenance requirements. The hydrologic assessment provides guidance on rainfall intensities, run-off coefficients and determining the design discharges.

Keywords
safety, environment, climate change, fauna crossings, pollution control, water sensitive design, erosion, sediment, backwater, tidal water, drainage immunity, freeboard, operations, maintenance, hydrology, Rational Method, time of concentration, run-off coefficient

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Summary

The *Guide to Road Design Part 5: Drainage – General and Hydrology Considerations* provides road designers and other practitioners with guidance on the design of drainage systems. This Guide needs to be used in conjunction with the other two Parts of the *Guide to Road Design* that relate to drainage design:

- Part 5B: Drainage – Open Channels, Culverts and Floodways (Austroads 2013b).

The estimation of rainfall and run-off is discussed together with other drainage design considerations. However, as this subject is extensive and covered in detail in other references, in particular *Australian Rainfall and Runoff Volume 1* (Pilgrim 2001), it is discussed in this Guide only to the extent necessary to support the design guidance provided.

This Guide provides information on the elements that need to be considered in the design of a drainage system including the hydrology, safety and environmental aspects, and the maintenance and operations of these systems.

The design processes and formulae necessary to design effective drainage systems and infrastructure are included. It is supported by appendices containing design charts and worked examples that provide further information.

This Guide outlines good practice in relation to drainage design that will apply in most situations, rather than specifying mandatory practice. The reason for this is that there are many factors that influence the design of a road and drainage for a particular situation or site, and practitioners therefore need to exercise sound judgement in applying the information contained in the Guide.
Guide to Road Design Part 5: Drainage – General and Hydrology Considerations

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1. Introduction

1.1 Purpose

Austroads Guide to Road Design seeks to capture the contemporary road design practice of member organisations (Austroads Guide to Road Design Part 1: Introduction to Road Design (Austroads 2010a)). In doing so, it provides valuable guidance to designers in the production of safe, economical and efficient road designs.

The purpose of the Guide to Road Design Part 5: Drainage is to provide guidance to drainage designers for the design of an efficient and effective road drainage system. The Guide comprises three parts:

- Part 5: Drainage – General and Hydrology Considerations (AGRD Part 5)
- Part 5A: Drainage – Road Surface, Network, Basins and Subsurface (AGRD Part 5A) (Austroads 2013a)
- Part 5B: Drainage – Open Channels, Culverts and Floodways (AGRD Part 5B) (Austroads 2013b).

Designers should be aware of the other Guide to Road Design subject areas in the full range of Austroads publications that may also be relevant to drainage design and can be accessed through the Austroads website (www.austroads.com.au).

The Guide provides the guidance necessary for designers to:

- analyse and design key components such as open drains and channels, pit and pipe systems, culverts, floodways, small basins and subsurface drains
- recognise the interaction between these components within the overall wider drainage system
- appreciate and take into consideration important structural, construction, maintenance and operational design issues relating to drainage
- appreciate environmental issues and the broad consideration of these issues.

The Guide is one of eight parts of the Guide to Road Design and the design of road drainage systems will require designers to refer to other Parts of the Guide as illustrated in Figure 1.1. The figure indicates that outputs from the drainage design process must be considered in the broader context of the overall road design task as they may impact on other elements of the design. As one example, the hydraulic and structural requirements of road culverts (i.e. size and cover) have a direct impact on the vertical alignment of the roadway. Another major consideration in the structural design of the road formation and pavement is the ability to achieve and maintain a low and stable moisture content which is dependent on an effective drainage system.

Prior to commencing a road design, stakeholders who may have property or other infrastructure that is, or could be, affected by the impacts of stormwater run-off, including those beyond the limits of the road reserve, should be identified and consulted. This is particularly important when considering drainage as the impacts may occur well outside the construction boundary. See Section 3 – Environment.
1.2 Scope and Definitions

1.2.1 Scope

The Guide to Road Design series provides the designer with a framework that is intended to promote efficiency in design, construction and maintenance of a length of roadway. The Guide is concerned primarily with the hydraulic design of surface drainage systems within the road reserve.

Whilst Part 5 provides general guidance on environmental management matters, these facilities require specialist design capability and often the approvals of relevant drainage and environment authorities is required and so have not been discussed in detail.
Drainage design occurs within the broader context of road design and this Guide should be applied in conjunction with all other parts of the *Guide to Road Design*. For example, road safety is an important consideration that affects drainage design and designers should refer to the *Guide to Road Design Part 6: Roadside Design, Safety and Barriers (AGRD Part 6)* (Austroads 2010c), regarding the safe design of drainage structures (e.g. open drain cross-sections, culvert end walls and identification and treatment of roadside hazards).

The design of subsurface drainage systems is another specialised field that receives only generalised guidance within this Guide. Detailed information on the requirements for subsurface drainage materials, the design of pavement drains and their construction and maintenance may be found in the *Guide to Pavement Technology Part 10: Subsurface Drainage (AGPT Part 10)* (Austroads 2009a). *AGPT Part 10* provides advice on the general requirements for subsurface drainage materials, design of pavement drains and construction and maintenance considerations.

Additional guidance on construction of drainage systems is provided in the *Guide to Pavement Technology Part 8: Pavement Construction* (Austroads 2009d) and maintenance of drainage systems is discussed in the *Guide to Pavement Technology Part 7: Pavement Maintenance* (Austroads 2009e).

### 1.2.2 Definitions of Key Terms

The following definitions and terms relate directly to their use within the context of the three Parts of the Guide. For other definitions refer to the Austroads *Glossary of Terms* (Austroads 2010b).

**Afflux** – The difference between the normal water level and the water level due to a natural or artificial restriction/obstruction within the channel. Afflux is a measurement taken at a point and typically measured in millimetres. Afflux can be either positive or negative, with the highest positive afflux immediately upstream of the restriction or obstruction.

**Annual exceedance probability (AEP)** – The probability that a given rainfall total accumulated over a given duration will be exceeded in any one year (www.bom.vic.gov.au).

**Average recurrence interval (ARI)** – The average or expected duration of time between the occurrence of two storm or flood events having or exceeding a given magnitude. The period between exceedances is a random variate.

**Drainage authority** – The local authority responsible for the drainage, waterways, floodway and/or other water environments within a jurisdiction. This may consist of more than one agency/authority for a specific project area and may include: drainage authorities, catchment management authorities, local government, water authorities, etc.

**Environment protection agency/authority** – The agency responsible for protection of the jurisdiction environment and application of its statutory powers and described in the environment act/s.

**Gutter** – The gutter is drainage infrastructure along a road pavement and referred in some jurisdictions as channels. It can be used with kerb and described as kerb and gutter, with some jurisdictions describing this as kerb and channel. The various terms have been used through this Guide.

**Hydrograph** – A graph showing the rate of flow (usually m³/s) over time, measured at a specific point within a catchment. It demonstrates the relationship between catchment flow rate and rainfall. A hydrograph can be either synthetic (i.e. developed for a specified design storm series), or actual (as taken from a stream gauging station).

**Nomograph** – A two-dimensional diagram designed to allow the approximate graphical computation of a function.
1.3 Jurisdictional Considerations

1.3.1 Link to Jurisdictional Supplements

In using this Guide or other Austroads Guides, practitioners should refer to jurisdictional websites to determine whether a supplement exists. Supplements provide details of jurisdictional practices where they vary from that contained in Austroads Guides. The variation may provide:

- a statement that particular guidance does not apply to the jurisdiction
- details of technical differences that apply
- information on how some guidance is to be applied
- additional guidance on a relevant subject that may or may not be covered in an Austroads Guide.

1.3.2 Road Agency Policies and Guidelines

Jurisdictions may have specific policies that may vary from information contained in Austroads Guides. It is important that these are reviewed to establish the relevant criteria for the site-specific design within those jurisdictions. These documents may also be referred to in agency supplements.

1.3.3 Federal and State Legislation, Strategies and Guidelines

There are a number of Federal and State regulations, policies and guidelines that may apply to a road project, particularly if it receives additional funding from those sources. These documents may include community, environmental and social considerations that need to be considered or addressed in the design and delivery of a project.

1.3.4 Jurisdictional Responsibility

A number of road agencies also have responsibility for a number of other regulatory areas. These may include drainage, environment, catchment management, land use planning, etc. Where necessary, a road agency should ensure that it is involved in land use planning at a state and regional level to ensure that road/network requirements are included in land use and management decisions.

1.4 Inter-agency Relations

It is important that external government agencies or other authorities be identified early in the planning and design process to ensure that their statutory responsibilities and other requirements are understood and appropriate action is taken to address them. In some cases, road agencies or authorities may also have an overall drainage plan or strategy for a catchment, of which the roadwork is a part.

1.5 Management and Planning Framework

Overall planning for drainage is usually under the control of a drainage authority. Road agencies may have some powers with respect to drainage (varies significantly across Australia and New Zealand), and schemes for road drainage need to comply with the overall strategy and may be subject to the approval of the drainage authority.

The drainage authority or the environmental protection agency/authority (EPA) may set standards for water quality, and the EPA is responsible for inspection and policing of standards, including drainage during road construction or operation of the road network.
Once the need to locate the road within the drainage catchment has been established, the road drainage systems need to be developed as part of the overall drainage plan for each clearly defined drainage catchment. This Guide supports the concept of major and minor drainage networks, which together provide an integrated drainage system within a drainage catchment. For further detailed understanding of the concept of major/minor drainage, refer to AGRD Part 5A – Section 2.

There are a number of segments to a typical drainage plan:

- **Major/minor drainage networks**: Storm drainage systems for existing development should be shown together with systems for proposed or planned development, at least in outline. These networks should be based on the major/minor concept which recognises the dual requirements of the drainage system to provide convenience on a day-to-day basis, and protection for life and property in major storm events.

- **Land use development plan**: Plans of all existing and proposed developments should be shown and their relationships to the existing and proposed major/minor networks and present or planned run-off characteristics, both in quantity and quality.

- **Stormwater retention, infiltration and/or detention measures**: Should be identified for each sub-catchment and such measures should consider land use, drainage networks, terrain, and soil characteristics. Where appropriate, the retention, infiltration, and detention measures adopted in each sub-catchment should aim to retain as much incident storm rainfall as possible, e.g. capture and reuse/water harvesting.

- **Environmental management plan**: This plan should be matched to the particular land use, drainage and run-off quality planned to occur in the catchment in the course of future development. The purpose of the plan is to ensure that run-off entering receiving waters from the development meets acceptable water quality criteria.

- **Sediment and erosion control measures**: Should be incorporated into the planning, design, construction, and operational phases of a drainage scheme, to minimise soil loss and ensure minimum downstream environmental damage from water-borne sediment.

### 1.6 Principles and Objectives of Drainage

#### 1.6.1 Principles

The following principles should be used to guide designers in the planning and design of effective road drainage systems:

- minimising the effects of flooding on the road, its environs and the community
- enhancement of road safety for all road users
- protection of road assets
- mitigation of adverse environmental impacts.

#### 1.6.2 Objectives

In line with the above principles, the overarching objectives for the management of stormwater are:

- Water conveyed across the road reserve should be discharged in a manner that does not cause nuisance or damage to the adjacent landowners or occupiers.
- Water within the road reserve should be controlled so as to allow safe and efficient passage of all road users and prevent damage to the roadway and associated infrastructure.
- Land should not be seriously disturbed by making it more susceptible to erosion or significantly reducing the water quality downstream of the land.
It is recognised that a road requires a drainage system to deal with stormwater run-off. Therefore, the drainage system becomes an important and integral consideration in the planning and design of road infrastructure. In order to provide an appropriate and economical drainage system (i.e. whole-of-life costs), all road projects, irrespective of location, size, cost or complexity, must consider and address the following aspects:

- safety of all road users
- impact of flooding on adjacent properties (e.g. public and private property)
- potential damage to the road asset
- traffic delays or extra travel distance caused by road closures
- the design life of the road
- ultimate road configuration
- road maintenance and operator safety
- provision of an acceptable level of flood immunity and accessibility
- conveyance of stormwater through the road reserve at a development and environmental cost that is acceptable to the community as a whole
- pollutant discharge from the road reserve to receiving waters
- land degradation caused by erosion and sedimentation during road construction, operation and maintenance
- any impact on habitats and movement of terrestrial and aquatic fauna.

The original design intent must be reviewed and understood and the existing system needs to be assessed against the above aspects for performance, adequacy and continued durability. This requirement particularly applies to projects where it is proposed to integrate the proposed drainage system with the existing drainage infrastructure. The design must ensure that the original intent is restored, deficiencies are corrected and modifications/changes appropriately considered and detailed.

1.7 Geometry and Drainage Relationship

There is a strong relationship or link between road geometrics and road drainage. It is considered that road geometric design includes road drainage design as well as design speed assessment, horizontal alignment design, vertical alignment design, intersection design, etc.

Roads cut across the landscape between geographical locations, impeding the natural flow paths of stormwater run-off and resulting in a concentration of flows and/or redirection of flows. The road is geometrically designed and typically tied to a control line (single or multiple). Carriageways consist of a series of interconnecting surfaces which collect rainfall run-off and control the direction it flows. These surfaces include the traffic lanes, shoulders, verge, batter slopes, etc. The design of drainage infrastructure is often governed by these surfaces, such as:

- the location and grading of kerb and/or channel are based on the road surface levels and layout
- the location and grading of table drains are generally linked to the control line geometry
- the location, orientation and grading of culverts are influenced by both the horizontal and vertical alignments of the control line and batter interface points.

Another aspect highlighting the relationship between geometrics and drainage is the careful consideration of grade, crossfall and superelevation development in order to minimise the water film depth on the road and hence the potential for aquaplaning.
Therefore the geometric aspects and design of roads have a direct effect on stormwater run-off, and these effects must be considered in the geometric design of a road, such as:

- provision of adequate clearances for structures
- provision of adequate height to develop headwater to assist culvert flow
- adjusting the road grade to enable or control flows across the road surface and within table drains.

It is therefore critical that the geometric design and drainage design of a road must be undertaken in conjunction with each other. Generally, a preliminary alignment and cross-section is developed to enable drainage design, however, design iteration and adjustment of both aspects then commences. Changes to either element can impact the other; therefore it is essential that there is good communication between those undertaking the drainage design and those undertaking the geometric design. The drainage design should be verified prior to the finalisation of the geometric design.

1.8 Use of Software

Computer programs can be used at many stages in the management of stormwater and are in common use. The larger the project the more necessary it becomes to use computer modelling. It is not within the scope of this Guide to discuss specific proprietary software. However, in applying such software the designer should ensure that:

- the computer model is appropriate for the situation and that the user is suitably skilled in its use
- input parameters have been selected to represent the actual conditions
- predicted results are within a reasonable and expected range.

Some road agencies may list approved software programs for particular applications.

1.8.1 Validation of Software and Predetermined Criteria

Any spreadsheet or computer-based tool developed and then used to assist with drainage design should be checked and tested for applicability, accuracy of results, and compliance with current standards and methodologies as may be prescribed in road agency policies, contract requirements, guidelines, Standard Specifications and Standard Drawings. Certification of design is deemed to cover use of these spreadsheets/tools.
2. Safety in Design

2.1 Safe System

Roads and roadsides are to be designed to reduce the risk of crashes and this requires the designer to consider the operation of the road and in this case the drainage system and the impacts it may have on the road users. These considerations include the surface flow characteristics, location and orientation of structures such as end treatments and driver awareness of the conditions they face as they travel along the road.

2.1.1 Providing for a Safe System

Adopting a Safe System approach to road safety recognises that humans as road users are fallible and will continue to make mistakes, and that the community should not penalise people with death or serious injury when they do make mistakes. In a Safe System, therefore, roads (and vehicles) should be designed to reduce the incidence and severity of crashes when they occur.

The Safe System approach requires, in part (Australian Transport Council 2011):

- Roads and roadsides designed and maintained to reduce the risk of crashes occurring and to lessen the severity of injury if a crash does occur. Safe roads prevent unintended use through design and encourage safe behaviour by users.
- Provide forgiving environments that prevent serious injury or death when crashes occur.
- Speed limits complementing the road environment to manage crash impact forces to within human tolerance, and all road users complying with the speed limits.

2.2 Workplace Health and Safety Act and Standards

In the design and location of drainage infrastructure, designers must consider the hazards/risks which may exist at particular sites. Workplace health and safety requirements should be considered in drainage design, installation and subsequent maintenance. Some of the issues which should be addressed may include, but are not restricted to:

- construction work being undertaken in a flowing waterway under temporary stream control and dewatering works
- excavation/trenching stage
- geotechnical analysis and the need for shoring
- placement of materials including excavated material close to trench walls
- location of underground utilities
- placement of materials
- proximity of machinery to excavations
- crane capacity and reach
- proximity of machinery to overhead power lines and obstructions
- backfilling
- probability of wall collapse arising from vibration or traffic movement
- amount of time excavation will be open
• working within confined space
• anticipated weather conditions.

The cost of control measures to be used to ensure that the risks associated with the above issues are minimised should be included in the installation costs of the drainage structure.

Designers of drainage infrastructure need to consider each installation as a workplace during maintenance operations and incorporate provisions that permit maintenance work to be completed in an appropriate way that manages exposure to risk in accordance with the relevant workplace health and safety act. See Section 5 – Operations and Maintenance and road agency guidelines, such as Department of Natural Resources and Water (DNRW 2007) for further details.

It is recommended that project teams include or have access to personnel who are current in their knowledge, understanding and application of the relevant legislation and the functions and responsibilities of local authorities.

2.3 Life and Property

Heavy and/or prolonged rainfall may lead to a significant rise in water levels, which in turn may inundate property and pose a risk to people. The drainage system must provide for the removal and safe disposal of this water before it reaches a level that results in an unacceptable risk.

Drainage of water means that flows will be concentrated into defined channels and systems, with potentially high volumes and velocity. Assessment of the risks to life and property should be undertaken and suitable measures taken to control these risks. This includes minimising access to open channels and structures, provision for system overflow in extreme run-off events, and consideration of the effects of structural element failure.

Safety of pedestrians and cyclists using the road will require consideration of depth and width of ponded water, and the potential for drainage elements such as surface depressions or pit inlets, to ‘catch’ cycle wheels or impede pedestrians.

2.4 Road Safety

The Guide should be considered in the broad context of road safety and the contribution that the Guide can make to the design of safer roads.

Safer road user behaviour, safer speeds, safer roads and safer vehicles are the four key elements that make a Safe System. In relation to speed, the Australian Transport Council (2011) reported that the chances of surviving a particular crash decrease markedly above certain speeds, depending on the type of crash, as follows:

• pedestrian struck by vehicle 20 to 30 km/h
• motorcyclist struck by vehicle (or falling off) 20 to 30 km/h
• side impact vehicle striking a pole or tree 30 to 40 km/h
• side impact vehicle to vehicle crash 50 km/h
• head-on vehicle-to-vehicle (equal mass) crash 70 km/h.

In New Zealand, practical steps have been taken to give effect to similar guiding principles through a Safety Management Systems (SMS) approach.
Road designers should be aware of, and through the design process actively support, the philosophy and road safety objectives covered in the Guide to Road Safety.

2.5 On-road Safety

2.5.1 Protection for On-road Users

Drainage facilities should be designed to minimise their impacts on motor vehicles. Culvert end treatments have the potential to act as hazardous obstructions to errant vehicles. They should be designed to not present an obstruction; either through relocation of the feature outside of the clear zone or where this is not possible an assessment should be undertaken to establish whether the end treatments can be made traversable (see AGRD Part 5B – Section 3.14). If neither remedial treatment is possible then safety barriers should be considered.

Similar to culvert end treatments, basins also pose a risk to errant vehicles. This is due to the volume of run-off they may contain, or batter slopes that may not comply with the requirements for traversable slopes. Basins that contain as little as 300 mm depth of water for a one year ARI, one hour duration storm are deemed to present a drowning hazard for unconscious occupants of overturned vehicles.

Designers should refer to AGRD Part 6 or jurisdictional/drainage authority requirements for guidance on the identification and treatment of roadside hazards including those associated with bodies of water, drains and drainage facilities.

2.5.2 Floodways

In rural environments there is an on-road safety concern relating to the crossing of floodways. The main issue associated with safety at floodways is provision of adequate sight distance for drivers to ensure vehicles can stop before entering the floodwaters. Preferably, the floodway longitudinal profile should be horizontal so that the same depth of water exists over the entire floodway length. The floodway length should be limited and on a straight stretch of road where possible. Adequate permanent and temporary signing and delineation must be erected.

As floodwaters recede, silt and debris can be left on the road surface of a floodway and this can be a hazard to road users. The road agency should inspect each affected floodway as soon as possible after a flood event and clear the surface if required. Floodways are discussed in detail in AGRD Part 5B – Section 4.

2.5.3 Aquaplaning

Aquaplaning is discussed in detail in AGRD Part 5A – Section 4, and is the phenomena whereby vehicle tyres can become separated from the road surface by a film of water resulting in loss of control of the vehicle. The effect on vehicle handling is directional instability and can be as significant as causing complete loss of directional control. As contact with the road surface diminishes so too does the vehicle’s braking ability.

Whilst in theory aquaplaning can occur on all roads, it is generally considered an urban road drainage matter. The coarse surfacing types used in rural road construction usually permit surface run-off to permeate into the voids and minimise water film depths. In the urban environment the use of surfacing materials such as asphalt, together with wider sections of pavement contribute to a heightened risk of water film depths interrupting the contact between the road surface and the vehicle’s tyres.

See AGRD Part 5A – Section 4 for further details on preventing and managing aquaplaning.
2.5.4 Cyclists

As detailed in Section 4 – Drainage Considerations, it is common for road surface drainage to be designed for a 10 year ARI. To the general public such events are intense and it is not unreasonable to assume an absence of cycling or pedestrian traffic during such times.

On kerbed roads, run-off during the design storm event will accumulate in the channel\(^1\) generally accommodating the full width of sealed shoulder (or cycle lane where present) and depending on the requirements of the road agency even encroach into the adjacent lane. Given the anticipated absence of cycling traffic during high rainfall events, it is not necessary to maintain facilities for such road users during these events. However, the designer should undertake checks for more frequent events (AGRD Part 5A suggests a two year ARI) and ensure that cyclists are not forced into adjacent traffic lanes by encroaching run-off. Further information on the provision of drainage for on-road cycling and pedestrian facilities may be found in *AGRD Part 5A* – Section 5.2.6.

2.5.5 Pit Lids

Pit lids should be designed to ensure the safety of motor traffic, maintenance vehicles and plant, and pedestrians and cyclists. It is therefore important that pit lids are designed:

- to carry the appropriate motor traffic loading where the pit lid (and lintel where appropriate) is located either in the road pavement, or off the road surface but within a likely vehicle wheel path (see AS 3996/2006)
- so that above ground features are minimised and the pit does not constitute a safety hazard for road users
- so that they do not constitute a trip hazard for pedestrians
- so that they do not constitute a hazard by trapping bicycle wheels (due to design of grates or their surrounds).

For high speed highways, motorways and arterial roads it is essential that pit lids are not located within the traffic lanes. If they need to be located within the road corridor, preferably they should be located outside the clear zone.

On low speed roadways, it is desirable that pit lids are also located outside of traffic lanes as the lids can cause impacts, noise, come loose and cause safety problems and, of course, traffic will be disrupted with future access required for inspection and maintenance.

2.6 Off-road Safety

2.6.1 Pedestrians and Cyclists

Provision for pedestrians and cyclists can be made when sizing a waterway culvert where they may serve a dual function for shared paths. However, in order to be effective, the approach ramps must allow clear vision through the culvert cell. It should also be kept in mind that pedestrians will often prefer to cross over a road than under it for reasons of security.

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\(^1\) The channel component of the kerb and channel is also referred to as a gutter in this Guide.
Culverts for multi-use will require a clear distinction between wet and dry passageways. For pedestrian and cyclist movement the culverts will need to be designed for dry passage under a particular average recurrence interval (ARI) event. Typically, the pedestrian and cyclist path should be elevated above the water level for a chosen design storm, and in some cases may be protected by a flood wall to reduce the annual time of closure of the path. The size requirements for passage, as well as hydraulic requirements need to be considered and the culvert sized appropriately to meet both of these requirements. In this regard, designers are referred to the Austroads Guide to Road Design Part 6A: Pedestrian and Cyclist Paths (AGRD Part 6A) (Austroads 2009b).

Bicycle and pedestrian paths along the banks of watercourses may be subject to inundation during storms of a particular ARI. It is important that adequate sight distance is provided for cyclists to safely stop without entering the water and that temporary signage and barriers are erected to protect cyclists during and after the event. Following such an event, silt and debris can be left on the path surface and this can be a hazard to path users.

Debris may also encroach onto shared paths as a result of run-off from adjacent fill embankments. It is important that debris from all sources be cleaned from the path surface as soon as possible after the event. See AGRD Part 6A or the local road agency guidelines for further information about design and safety requirements.

2.6.2 Maintenance Access

The safety of maintenance personnel within drainage pits, including the requirements for step irons, ladders and landings, is discussed in AGRD Part 5A – Section 6.

Maintenance access should aim to bring trucks as close to a basin as possible to allow direct removal of detritus from the basin. Thus the access ramp should ideally have a maximum grade of 1:10, with an absolute maximum grade of 1:6.

If maintenance trucks are located too far from the basin during de-silting operations, the access track between the basin and the trucks can become contaminated with mud. This mud can make the track very slippery and dangerous if it is too steep.

2.7 Protection of General Public

2.7.1 Culverts and Stormwater Drains

Where long culverts and drainage inlets/outlets potentially provide a hazard to human safety (particularly in urban areas) preventative measures should be considered, such as fencing, swing gates and grates. Any safety device needs to ensure that it prevents both access to the culvert and trapping of a human against the grate. The effect of any proposed human safety measure on culvert capacity and efficiency needs to be checked.

2.7.2 Energy Dissipators

Energy dissipation may be necessary due to high flow velocities. Dissipation devices usually consist of large obstructions to the flow and result in a high degree of turbulence. For these reasons, energy dissipation structures should be avoided in urban areas where possible. Access should be limited by appropriate fencing. See AGRD Part 5B – Section 3.13 for further information on energy dissipators.
2.7.3 Drainage Basins

Drainage basins are used to either temporarily or permanently store stormwater run-off, typically to meet land development requirements and minimise impacts on downstream infrastructure. They may either detain run-off for later release, or retain run-off for infiltration, evaporation or other uses. Basins may be purpose built or have multi-use functions such as recreational areas.

Provisions should always be made to allow safe escape from drainage basins during wet weather, i.e. when the banks are wet and slippery. Clear warning signs should be displayed prominently and consideration should also be given to the placement of depth indicators. For example, the inclusion of a fence, when side batters down to the water’s edge are steeper than 1:5 and/or water depths are more than 350 mm. See local jurisdictional guidelines such as Melbourne Water’s *Constructed Waterways in Urban Developments Guidelines* (MW 2009) and The Royal Life Saving Society Australia’s (RLSSA) *Guidelines for Water Safety: Urban Water Developments* (RLSSA 2004) for further informational about safety requirements associated with wetlands and permanently wet basins.

A maximum depth of 1.2 m during an ARI 20 year flood is recommended for publicly accessible basins and all batters that are accessible to the public should have a maximum slope of 1:8. Fencing may be required for basins within, or designed as multi-use zones to prevent public accesses, especially during and after storm events. See jurisdictional guidelines such as Melbourne Water’s *Constructed Wetlands Guidelines* (MW 2010) and *Constructed Shallow Lake Systems Design Guidelines for Developers* (MW 2005a) for detailed information on safety issues related to water lakes and basins.

2.7.4 Fencing

Fencing may be required to protect the public (pedestrians and cyclists) from water in drainage infrastructure such as culvert and underground drainage inlets, and in some cases retarding basins.

As an example, where culvert inlets are located in areas likely to be accessed by people, particularly children, consideration must be given to appropriately fence the inlet to ensure someone is not swept into the culvert during a flood event. An example of this type of fencing is shown in Figure 2.1.

**Figure 2.1:** Fencing around pipe inlet

![Fencing around pipe inlet](Source: DTMR (2010b)).
A serious problem with fencing a culvert inlet is that the potential for blockage is greatly increased and designers need to check for and mitigate any possible adverse effects due to the blockage.

Guidance on the use and design considerations and general use of fences for drainage basins and pedestrian and cyclist safety, is provided in the Guide to Road Design Part 6B: Roadside Environment (AGRD Part 6B) (Austroads 2009c).

The use and design of fencing adjacent to bicycle paths is covered in the AGRD Part 6A (Austroads 2009b).

Generally a security fence should be installed where one of the following criteria is met (the distance is a straight line measurement). The location of a basin is within:

- 1.0 km of public facility, e.g. hotel
- 1.0 km of a school
- 0.5 km of a township
- 0.3 km of a local road
- 0.3 km of a stopping bay
- 0.1 km of property access.

Fencing of a temporary basin should be removed upon decommissioning of the basin.
3. Environment

3.1 General

A sustainable and environmentally aware transport system acknowledges not only the impact of the road network on the surrounding environment but takes into account the long-term impacts during land use planning stages as well as the planning, design, construction and maintenance of that road network.

Environmental issues associated with road infrastructure should be identified and assessed throughout the road planning and design process. Project-specific environmental assessment provides information about the condition of the existing environment, the proposed project area, associated environmental impacts of the proposal and the identification of any opportunities for environmental management.

Road agencies plan, manage and develop their road network and its use so that it is sustainable and meets the challenges of an ever growing transport system. It is imperative, in partnership with other agencies, that road agencies endeavour to ensure that the prosperity and liveability of their communities are maintained and improved. It is also necessary to continue to meet the needs of business and industry and help deliver sustainable transport strategies.

In the VicRoads Sustainability and Climate Change Strategy 2010–2015 (VicRoads 2010) a sustainable transport system is described as one that:

- contributes to meeting the social and economic needs of the present generation without compromising the capacity for future generations to meet their own social and economic needs
- ensures the short-term and long-term protection of the environment, locally and globally
- promotes and provides for transport options with a smaller carbon footprint
- is safe and supplies ongoing health and wellbeing
- provides for the future prosperity of the nation.

3.1.1 Scope

This section specifically addresses environmental aspects of the design of drainage requirements for the road network and those issues associated with the design and operation of the road network. While there is general information contained in this Guide regarding site management and pollution control, details during the construction phase are dealt with in a number of environmental protection authority, road agency and drainage authority regulations, policies and guidance manuals as well as road agency procedures and contractual requirements.

3.1.2 Environmental Considerations

A key environmental consideration related to drainage and road run-off (via stormwater, site water run-off, rainfall, litter and spills) is pollutant export and its resulting impact on water quality. Pollutants contained in run-off and drainage from road corridors have the potential to adversely affect the water quality and aquatic biota of receiving waters with short or long-term impacts.

For any given project, the significance and impact of pollutant export will depend upon:

- the relative sensitivity of the receiving environment
- traffic type and volume
- road project infrastructure type and form (e.g. off-ramp, traffic lights, bend in road, steep hill, etc.)
• climatic factors experienced in the locality.

Therefore, pollution control techniques must be established and implemented according to many factors, including the type, source, concentration of pollutant export and the risk of harm it may have on the receiving environment.

3.2 Climate Change

3.2.1 General

In a report prepared for the Intergovernmental Panel on Climate Change (IPCCC) (Pachauri & Reisinger 2007), it was suggested that there is scientific evidence that human induced climate change can be attributed to the burning of fossil fuels, which has increased the layer of greenhouse gases (GHG) in the atmosphere and resulted in elevated atmospheric temperatures.

The significant growth in greenhouse gases in recent times has led scientists to believe that things like sea level rise, changed weather patterns, and increasing temperatures are already in the system and will accelerate. While changes in climate are set to continue throughout this century and beyond, impacts are of concern in existing planning and design time-scales.

Climate change will have a significant impact on transportation, affecting the planning, design, construction, operation, and maintenance of infrastructure. Decisions taken today, particularly those related to the redesign and retrofitting of existing infrastructure, or the location and design of new infrastructure, will affect how well the network is able to adapt to climate change into the future. Consideration of the likely impacts within each project will help avoid costly future investment and disruption to operations.

Climate change, resulting in more extreme and frequent severe weather, may have an effect on a road agency’s infrastructure in a number of ways including the following:

• increased range of temperatures leading to increased thermal loading effects and degradation of structural material
• more severe and frequent flooding and/or flowing water volumes leading to increased hydraulic loading on structures and more severe scouring damage to foundations and road formation
• sea level rise near the coast and storms which may result in flooding of roads
• increased flow volumes and maximum water levels leading to the need for longer and higher bridges, larger capacity culverts or the need for overland flow paths
• more severe wind-loading and more frequent exposure to high wind loads
• higher tides and frequent storms in coastal areas leading to more severe exposure to saline environments and as a consequence, higher initial cost/resource usage in new structures and shorter life/higher maintenance costs in existing structures and road infrastructure
• reduced rainfall in dry areas leading to higher salinity in soils and groundwater, desiccation and shrinkage of clay soils, all of which lead to damaging effects on structures and road pavements.

It will be necessary for road agencies to include the consideration of climate change in decision making in order to better appreciate the risks that might be placed on existing and future infrastructure and to incorporate greater flexibility to accommodate change where appropriate to do so.
3.2.2 Sea-level Rise

In the past, the possible impacts of climate change on sea level and drainage near coastal areas (which is where most of the population in Australia resides) have not been considered during planning and design processes. Given that there is a large body of scientific work suggesting that these changes will become much more evident within the design and planning horizons that road agencies use for future infrastructure development, the planning and design of roads and bridges in coastal regions should now include consideration of an anticipated rise in sea level (MRWA 2012).

Predictions of the possible effect on sea level and other effects are given in the International Panel on Climate Change 4th Assessment Report, IPCC (Hennessy et al. 2007) and websites – for Australia, Department of Climate Change and Energy Efficiency (DCCEE 2012) and for New Zealand, Ministry for the Environment (ME 2012).

Designers should ensure they are familiar with the latest design/research information and should consult with the relevant local government and coastal management authorities.

3.2.3 Catchment Changes

Due to climate change, it is expected that a number of catchment changes will occur. In some catchments, warmer and wetter conditions are expected with resultant more vigorous vegetation growth. This will reduce run-off and may counter to some degree any increase in rainfall intensity resulting from a temperature rise expected to counter to some degree any increase in rainfall.

In other catchments, drier conditions are expected with climate change. In these catchments it can be expected that vegetation growth will reduce. With reduced vegetation, run-off can be expected to increase, especially from more extreme events.

3.2.4 Increase/Decrease in Rainfall Patterns

Rainfall patterns

The Rational Method (refer to Section 6 – Hydrology for further detail on the application and use of this method) assumes that rainfall intensity is uniform throughout the storm. This simplification is not necessarily true, and design rainfall patterns for different parts of Australia and for different durations of storms should be selected from Australian Rainfall and Run-off: A Guide to Flood Estimation \(^2\) Volume 2 (Pilgrim 2007) (refer to Section 6 – Hydrology for further information on AR&R Vol. 1 and Vol. 2). The total rainfall for the selected storm duration as read from the rainfall Intensity-Frequency-Duration table or chart should be distributed in time increments according to the percentages shown on the design rainfall pattern.

Rainfall excess

To convert rainfall into run-off, some kind of loss model must be assumed, as discussed in AR&R Vol. 2 (Pilgrim 2001).

The losses are deducted from the incoming rainfall pattern. If losses exceed rainfall, there is no run-off for that time interval. Where there is a rainfall excess, the estimated depth is multiplied by the contributing area to obtain the run-off volume in that time interval.

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\(^2\) Abbreviated to AR&R Vol.1 or Vol. 2 as appropriate through this Guide.
For further information on adjustments to rainfall intensities, refer in Australia to the Department of Climate Change and Energy Efficiency (DCCEE 2012) and in New Zealand to the Ministry for the Environment (ME 2012).

3.3 Fauna Passage/Crossings

3.3.1 General

Through environmental management, some of the possible impacts of road infrastructure can be reduced or eliminated.

Continuity of wildlife corridors may require consideration including the desirability of:

- extending vegetation cover that exists upstream and downstream under the bridge
- placing the bridge abutments away from the banks of the watercourse
- shielding road lighting where it may affect migration of nocturnal mammals
- fencing to guide fauna to crossings or exclude them from hazardous areas.

The passage of fauna and fish through culverts may require provision of larger culverts and/or special features at inlets or outlets, or within culvert cells which could affect the hydraulics of the culvert.

Fauna crossings are explained in more detail in AGRD Part 6B (Austroads 2009c).

Recognition of the impacts of road corridor development on fauna populations has led to modifications in the way that roads are now designed. A substantial amount of research has been undertaken to develop practices that facilitate fauna movement through the road corridor in a way that minimises mortalities. Much of the research has focussed on passages that are integral with drainage structures. As such, the provision of fauna passage is one of the key environmental factors which may influence the physical dimensions of a drainage structure.

This section provides an overview of what steps need to be taken when the project environmental assessment process has identified a need for fauna passage to be incorporated into drainage design.

3.3.2 Identifying Fauna Passage Criteria

When a project environmental assessment has identified fauna passage requirements, it is necessary to undertake the following steps:

- identify terrestrial and aquatic fauna pathways and areas of high mortality on the road
- identify the species groups of concern
- consult with the relevant authority
- identify criteria affecting drainage design (e.g. ensuring dry fauna passage at all times).

3.3.3 Identify Terrestrial and Aquatic Fauna Pathways

A review of the project environmental assessment should be undertaken to check for the presence of any significant terrestrial and/or aquatic fauna movement pathways which could be potentially affected by the proposal.
If fauna pathways have been identified or are likely to exist in the study area, proceed to identifying the species groups of concern. If not, document the outcomes of the environmental assessment review and continue identifying other relevant drainage design criteria. Bridges provide a good solution to maintaining terrestrial and/or aquatic fauna movement pathways as shown in Figure 3.1.

Figure 3.1: Fauna corridor under bridge

Source: DTMR (2010b).

3.3.4 Identify the Species Group

Where fauna pathways have been identified, identify the relevant species group from the list below:

- fishes
- amphibians (frogs)
- mammals: macropods
- mammals: arboreal species (e.g. possums, gliders, etc.)
- mammals: koalas
- mammals: platypus
- mammals: bats/flying foxes
- mammals: small-size (e.g. echidnas)
- marsupials: bandicoots
- birds (flying and ground-dwelling)
- reptiles (e.g. snakes, lizards and turtles)
- invertebrates (insects and spiders).

3.3.5 Consult with the Relevant Authority

During the planning and design stages it is necessary to consult with the relevant authorities to establish the type of fauna within the area of a project, their requirements and need to be designed into a specific project.
Many jurisdictions produce guidelines that outline the habitat and migratory requirements of particular species. This is particularly important where a road alignment will dissect a population of a species and there is a need to provide access across the alignment to ensure the population’s survival.

3.3.6 Identify Criteria Affecting Drainage Design

In consultation with the relevant authorities and from information provided in the environmental assessment (i.e. baseline fauna studies), determine the following:

- Specific characteristics and requirements for the species of concern (i.e. movement patterns, habitat range).
- Opportunities to facilitate safe passage of fauna through the drainage system (e.g. bridging options, culvert modifications). See Table 3.1 for an indication of culvert size for various sized fauna, and fauna sensitive road design guidelines, such as Queensland’s *Fauna Sensitive Road Design Manual Volume 1* (DMR 2000) and Volume 2 (DTMR 2010a) and VicRoads *Fauna Sensitive Road Design Guidelines* (VicRoads 2012).
- The economic and engineering feasibility of potential opportunities in consultation with the project manager and designer.
- The need for further preliminary design work (i.e. implications for cut-and-fill balance if a different culvert size is required, revised dimensions, etc.).
- Specific design criteria for drainage structures (i.e. allowable flow velocity, minimum culvert height, dry passage requirements, etc.).

### Table 3.1: Use of culverts or underpass types by fauna

<table>
<thead>
<tr>
<th>Fauna type</th>
<th>Small pipe &lt; 0.5 m dia</th>
<th>Large pipe &gt; 0.5 m dia</th>
<th>Small box culvert &lt; 1.2 m high</th>
<th>Large box culvert &gt; 1.2 m high</th>
<th>Bridge underpass</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small mammal</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Medium mammal</td>
<td>×</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Large mammal</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>•</td>
<td>•</td>
</tr>
<tr>
<td>Semi-arboreal mammal</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Arboreal mammal</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>•</td>
<td>✓ (large only)</td>
</tr>
<tr>
<td>Bats</td>
<td>×</td>
<td>×</td>
<td>×</td>
<td>✓ (adapted roof structure)</td>
<td>✓ (adapted roof structure)</td>
</tr>
<tr>
<td>Reptile</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Bird</td>
<td>×</td>
<td>×</td>
<td>•</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Amphibian</td>
<td>✓</td>
<td>×</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Introduced predator</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

*Caution: This table is based on preliminary research only. Although not confirmed at the time, fauna should pass through all the culverts larger than the minimum ones shown. Recommended minimum sizes for design are shown in DMR (2000).*

*Source: Adapted from DTMR (2010b).*
3.3.7 Fish Passage

**Barriers to fish passage**

Any structure placed across a stream can impede fish moving upstream or downstream, including dams, weirs, floodgates, roads, bridges, causeways and culverts. These structures can act as physical (e.g. dams and weirs), hydraulic (e.g. fast flowing water through pipes) and behavioural barriers (e.g. long dark tunnels) to fish movements, resulting in:

- restricted spawning migrations and movements to critical habitats
- reduced dispersal of juvenile fish
- large congregations of fish downstream of the structure that may lead to increased predation and disease
- genetically isolated populations upstream and downstream of the structure
- localised extinction of some species above the barrier (source: www.vicfishways.com.au).

In certain situations, culvert designs will have special requirements to allow the passage of fish and/or terrestrial fauna. While the requirements will differ depending on whether fish and/or terrestrial fauna passage is to be catered for, all will require a clear distinction between wet and dry passageways.

It is typically more efficient to determine size requirements for terrestrial fauna or fish passage prior to undertaking any hydraulic calculations, as the former may dictate culvert size.

Fish passage may be required for regular movement between permanent ponds on each side of a roadway (i.e. for feeding, shelter or breeding), or for seasonal migration for breeding. Fish migration is known to occur through both instream and floodplain culverts.

Fish generally migrate upstream in response to rain events and hence may swim upstream where:

- the road crosses a stream with permanent water
- there are pools of water upstream and downstream of the road which would be connected by intermittent flows through a road waterway
- significant rain events open upstream areas to fish habitat.

In fish migration areas, bridges are preferred, however, if a culvert is required, the following types in order of preference) may be used:

- arch
- buried (earth bed) box culvert
- box culvert with low flow channel
- pipe culvert.

To allow fish to swim upstream through a culvert, the culvert should:

- where practicable, reproduce the natural conditions of the watercourse bed and ideally be recessed below the natural bed levels (about 100 mm below, some guides recommending that the culvert be recessed 20% of the cell height and backfilled with bed material won on site)
- have a floor slope that does not exceed 1 in 100
- have a minimum water depth of 0.2 m to 0.5 m and a water velocity at periods of migration not greater than 0.3 m/s at 0.5 m water depth (noting that an additional cell or floor roughening to reduce velocity may be required).
Where possible, a low flow channel should have:

- a maximum flow velocity of 1.0 m/s
- a maximum flow velocity of 0.3 m/s at a depth of 0.2 m to 0.5 m
- where practicable, a minimum flow depth of 0.2 m to 0.5 m
- an absence of areas of large-scale turbulence (relative to flow depth, i.e. whirlpools).

However, the above flow conditions are usually difficult to achieve, especially if the culvert floor cannot be recessed below the natural bed level. In such cases the next preferred option is to size the culvert so that there is minimal change in the channel flow area at the culvert.

To assist in the development of suitable low flow conditions and to assist in the control of sediment flow, an inlet weir can be formed:

- to direct all base flows to one wet cell
- in front of the dry cells to a height of 0.3 m to 0.5 m
- and placed four times its height from the culvert entrance.

When fish passage is required, measures should be taken to ensure that light can enter the culvert, e.g. by providing stormwater drop inlets from a median above the culvert.

Drop inlets are not recommended at culvert entrances in regions where fish migration is required. In areas where fish passage occurs, high velocities may inhibit or prevent passage from occurring. For example, where fish passage is a requirement, there may be specific velocity criteria that apply to normal and/or low flow. These will typically be determined through an appropriate local process and use of a fish specialist. However, the velocity requirement may depend on the fish species requiring passage.

Further information on fish passage can be found in Why do Fish Need to Cross the Road? – Fish Passage Requirements for Waterway Crossings (Fairfull & Witheridge 2003).

### 3.3.8 Fauna Crossings – Design Criteria

To allow for fauna crossings through a culvert, a raised ledge against the cell wall may be required on either side of the watercourse, but aligned with the dominant movement of fauna. The ledge should be 0.3 m to 0.5 m wide, constructed of materials appropriate to the relevant fauna use, at least 100 mm above the base flow water level, and connected to a dry path on both sides.

Other features associated with a terrestrial passage include:

- separate wet and dry cells to cater for both terrestrial and aquatic movement
- a low flow channel instead of a ledge
- a lizard run (Figure 3.2 and Figure 3.3)
- rough side walls as an alternative to a lizard run.

As a general rule, culverts should not be modified to provide or promote habitats for birds and terrestrial animals as the habitats would not be natural and would be vulnerable to floods.
Table 3.2 is a guide to the appropriate culvert height to assist the passage of particular fauna.

**Table 3.2: Minimum desirable culvert cell height for terrestrial passage**

<table>
<thead>
<tr>
<th>Species</th>
<th>Culvert cell height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small mammals</td>
<td>0.5</td>
</tr>
<tr>
<td>Medium mammals</td>
<td>0.5</td>
</tr>
<tr>
<td>Large mammals</td>
<td>1.2</td>
</tr>
<tr>
<td>Semi-arboreal (e.g. Koala, Eastern Pygmy Possum)</td>
<td>&gt; 1.2</td>
</tr>
<tr>
<td>Microchiropteran bats</td>
<td>&gt; 1.2</td>
</tr>
<tr>
<td>Reptiles</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Source: DTMR (2010b).*
3.3.9 Riparian and Wildlife Corridors

Riparian land is any adjoining land to creeks and rivers or any land that influences the waterway bank itself and which may also be a wildlife corridor. Rather than just narrow strips of land adjoining these corridors, riparian land may be quite wide and diverse. The riparian zone provides natural buffer zones that protect the water quality and the watercourse banks and may extend across floodplains.

In some watercourses, management programs have been initiated to restore degraded land in the riparian zone. These issues need to be considered when locating bridge abutments and piers. They may also need to be considered when locating culverts.

In locations where terrestrial corridors are required under a bridge, and when suitable circumstances exist, vegetation cover that exists upstream and downstream of the bridge, should be extended under the bridge.

Bridge abutments should be moved away from watercourse banks in order to increase the opportunity for fauna passage that is usually required on both sides of the waterway. This also provides a benefit related to the risk of scour.

Lighting or gaps may be required in culverts to assist diurnal animals to move through the culvert. This is because they may not be willing to enter a dark entrance where the other end is not visible.

For guidelines on the use of wildlife exclusion or movement guide fencing, see the local jurisdiction’s fauna-sensitive road design guidelines.

Reference should be made to the project’s environmental assessment for any specific requirements that need to be considered in the location, orientation or geometry of the culvert/bridge.

3.4 Pollution Control and Water Quality

3.4.1 General

A significant amount of pollutants, ranging from litter to solid and soluble particles, accumulate on roads and car parks and are conveyed into stormwater drainage networks during storm events. Between run-off events, pollutants enter drainage networks as a result of ineffective street sweeping, wind action or vehicle movement.

On highways and freeways, gross pollutant management is slightly less important than in local streets and parking areas in typical commercial and industrial areas. For highways and freeways, the emphasis in stormwater quality management is on control of suspended solids and associated contaminants and control of turbidity.

Typical pollutant loads for various land uses and road operation are set out in Wong, Breen and Lloyd (2000). For the considerable amounts of erosion that occur during construction, estimates need to be made for the specific site and the months of the year that exposure occurs. Further detail on erosion and sediment are provided in Section 3.6 – Erosion and Sediment.

Relevant issues should be covered in dealings with other authorities throughout the planning and design process; it is suggested that designers should be aware of the initiatives and development work undertaken by eWater CRC (e.g. MUSIC, a Model for Urban Stormwater Improvement Conceptualisation model (eWater CRC 2012)).
3.4.2 Sources of Pollution from Roads

Road operation contributes a range of pollutants to the environment including litter, suspended solids, toxicants, oil and other hydrocarbons. These are usually washed off during rain events. The management and clean-up of road accidents/incidents can also have a significant impact on a receiving water environment. Table 3.3 lists a range of common sources of pollutants in road run-off.

Table 3.3: Common sources of pollutants in road run-off

<table>
<thead>
<tr>
<th>Source</th>
<th>Typical pollutants</th>
<th>Source details</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric deposition</td>
<td>P, N, S, heavy metals (Pb, Cd, Cu, Ni, Zn, Hg), hydrocarbons</td>
<td>Industrial activities, traffic air pollution, and agricultural activities (all deposited as particulates). Raindrops also absorb atmospheric pollutants.</td>
</tr>
<tr>
<td>Traffic – exhausts</td>
<td>Hydrocarbons including PAH (polycyclic aromatic hydrocarbons), MTBE (Methyl Tertiary Butyl Ether), Pb, Cd, Pt, Pd, Rh</td>
<td>Vehicle emissions, unburned fuel and particles from catalytic converters.</td>
</tr>
<tr>
<td>Traffic – wear and corrosion</td>
<td>Sediment, heavy metals (Pb, Cr, Cu, Ni, Zn, Cd, Mn)</td>
<td>Tyre wear, vehicle corrosion.</td>
</tr>
<tr>
<td>Leaks and spillages</td>
<td>Hydrocarbons, phosphates, heavy metals, glycol, alcohols</td>
<td>Engine leaks, hydraulic and de-icing fluids, refuelling and accidental spillage and lubricating oil can contain phosphates and metals.</td>
</tr>
<tr>
<td>Roofs</td>
<td>Heavy metals (Cu, Pb, Zn), bacteria, organic matter</td>
<td>Atmospheric deposition, corrosion of metal roofing or coatings such as tar can sometimes contain concentrations of heavy metals. Animal droppings and vegetation contribute organic matter and bacteria.</td>
</tr>
<tr>
<td>Litter/animal faeces</td>
<td>Bacteria, viruses, phosphorus, nitrogen</td>
<td>Drink cans, paper, food, cigarettes, animal excreta, plastic and glass. Some of this will break down and wash off urban surfaces. Dead animals on roads decompose and release pollutants such as bacteria and nutrients. Animals (including pets) leave faeces that wash into the drainage system.</td>
</tr>
<tr>
<td>Vegetation/landscape maintenance</td>
<td>P, N, herbicides, pesticides, fungicides, organic matter</td>
<td>Leaves and grass cuttings, herbicides and pesticides used in landscaped areas such as gardens, parks, recreation areas and golf courses can be a source.</td>
</tr>
<tr>
<td>Soil erosion</td>
<td>Sediment, P, N, herbicides, pesticides, fungicides</td>
<td>Run-off from poorly-detailed landscape or other areas can wash onto impervious surfaces and cause run-off pollution.</td>
</tr>
<tr>
<td>Cleaning activities</td>
<td>Sediment, organic matter, P, N and detergents</td>
<td>Washing vehicles, windows, bins or pressure washing hard surfaces transports pollutants into surface water drainage.</td>
</tr>
<tr>
<td>Illegal disposal of chemicals and oil</td>
<td>Hydrocarbons and various chemicals</td>
<td>Occurs at small (domestic) or large (industrial) scales.</td>
</tr>
<tr>
<td>Pavement</td>
<td>Sediment</td>
<td>Pavement wear from water, traffic, and so on.</td>
</tr>
</tbody>
</table>

Source: DTMR (2010b).

Tyre wear contributes to road grit and traces of the various metals used in tyre and brake manufacture. Vehicle parts are a source of iron, nickel, copper, cadmium and chromium. Traffic crashes result in plastic and glass fragments. Fuel combustion and additives produce lead, sulphur and nitrogen compounds. Fuel or lubricant spills may result in release of petrol or oils and hence release hydrocarbons into stormwater systems and/or groundwater.

Large car parks near regional shopping centres are sources of significant quantities of litter including plastic containers, packaging, and plastic bags. Car parks are also major sources of oil, grease and other hydrocarbons.
Residential streets, in addition to any pollutants from road operation, contribute vegetation from street plants or lawn cutting, animal faeces, detergents and waste materials from building sites. Unsealed residential streets add road materials and other sediments to the pollution load.

The relative contribution roads make to changes in catchment hydrology and pollutant load is directly related to their contribution to total catchment imperviousness. In urban catchments, the impervious local road network usually represents a significant proportion of the catchment compared to arterial roads and therefore has a greater impact on the catchment hydrology.

### 3.4.3 Spill Management

The design of drainage systems can play a role in spill management. The type of transport, traffic volume per day and receiving environment will need to be assessed on a site-by-site basis for each project. It may simply be more feasible to realign the road project or design a different and more robust drainage system that removes, or minimises the discharge of road run-off to very sensitive environments.

High volume road projects upstream of very sensitive waterways, such as Ramsar wetlands (Ramsar 2012), may require specific spill management elements to be designed into the drainage system. While it could be said that this should occur for all parts of a drainage system, this is neither practical, nor cost effective from a construction or ongoing maintenance perspective. Specific capture techniques can be installed to respond to high pollution locations, even retrospectively.

Occasionally trucks overturn, spilling their loads onto the road and in some cases into the drainage system. The most dramatic event is incineration of part of a drainage system by ignited fuel. However, even solids like fertilisers may be inadvertently hosed off the road and into the drains by emergency services personnel. These events could occur at any point on the arterial road system and the emergency services, in consultation with the environment protection agency, need to develop and use appropriate procedures for collection of spills of toxic and nuisance materials.

However, when any drain is upstream of a reservoir supplying potable water, arrangements should be made for interception and holding of effluents from the road reserve. This may take the form of a sedimentation or retention pond, with gate valves to shut off flow to the reservoir in the event of heavy pollution or a toxic spill. Toxic materials would then be pumped out and transferred to another site for treatment or disposal.

The issue of interception of contaminated water should be assessed on a project-by-project basis. However, provision should be made to cater for the interception of about 20 m³ of contaminated water. This amount will vary depending upon the traffic type using the road and, where B-double tankers are expected, the contaminated water volume could be doubled. The requirement should be confirmed with the relevant jurisdiction.

### 3.4.4 Typical Steps for Pollution Control and Treatment

Guidance on the treatment of pollution in stormwater is given in:

- Road Run-off and Drainage: Environmental Impacts and Management Options (Austroads 2001)
- Guidelines for the Treatment of Stormwater Run-off from the Road Infrastructure (Austroads 2003)
- The Collection and Discharge of Stormwater from the Road Infrastructure (Alderson 2006)
- WSUD Engineering Procedures – Stormwater (Melbourne Water 2005b)

A flow chart for pollution control and treatment for both construction and operational phases is shown in Figure 3.4.
Figure 3.4: Flow chart for design of pollution treatment train


Note: For more information on treatment trains, see Section 3.5.10 – Treatment Train. Source: Alderson (2006).
3.4.5 At-source vs Catchment-based Treatment for Roads

There has been a lot of discussion within the industry and agencies about whether it is best to treat pollutants at a source or at a catchment level. There is sufficient justification for implementing treatment elements at either location. The decision should reflect any high value ecosystems that may be impacted by road run-off in the long-term. The land use, a project’s location within a catchment and the site or project characteristics also need to be taken into account to properly assess the best option for a particular project. A jurisdiction’s regulations may indicate when and where pollution control measures should be implemented.

At-source controls

At-source controls are those treatment elements that are placed on site or near the source of the pollutant load. For road projects this means in road reserves and median strips and may include buffer strips, grass swales or biofiltration trenches where there is sufficient room. Other treatment elements may be used where available land, either on the roadside or belonging to others, may be utilised, e.g. for wetlands.

In-transit controls

In-transit controls are where treatment elements are constructed between the source and the receiving waters. Retarding basins (for flood control or vegetated for treatment) are a common example. This type of control is more likely to be undertaken by a drainage authority. At a localised point source of pollution, e.g. roundabouts, the local ‘roundabout’ drainage system can be separated from the area wide road drainage system and each system designed for the required pollution control.

End-of-pipe controls

End-of-pipe controls are treatment elements placed at the end or discharge point of a catchment area before it enters the receiving waters and may include treatment elements such as litter traps. There are various circumstances where end-of-pipe controls could be used on road projects, in particular projects in built-up urban areas, which discharge directly to a waterway.

Discharge of road run-off into creeks, rivers and wetlands should first be treated by passing through appropriate treatment devices so that discharge can be improved to a standard acceptable to the receiving environment. The proposed treatment should be adequate depending on the risk, quantity and type of pollutants and the environmental values of the receiving waters.

Catchment-based controls

Catchment-based controls are treatment elements/trains (i.e. a series of treatment elements designed to collectively meet the treatment objectives, see also Section 3.5.10 – Treatment Train), that are strategically placed to treat polluted stormwater from larger catchment areas and tend to be constructed by drainage authorities, including local government.

Wetlands are a common type of treatment used and suit areas where at-source controls are not feasible and where a better environmental outcome can be achieved further down the catchment or at the junction of several smaller catchments.

The jurisdictional responsibilities of a road agency may also impact on the decision made regarding the type of treatment train used and whether these are placed at source or are catchment based.

It is important to ensure that maintenance responsibility and requirements are identified and resourced as part of the planning and design process.
3.4.6 Debris Control

The performance of culverts or other drainage infrastructure can be compromised by the accumulation of debris at inlets. This accumulation can cause failure of the drainage structure, possibly resulting in over-topping of the roadway by floodwaters, with ensuing damage to the embankment or to the properties upstream and downstream of the culvert. Where there is likelihood of a culvert being so affected, consideration should be given to the installation of a debris control structure.

Measures to control debris range from design precautions to providing elaborate control structures. Design precautions include providing a smooth well designed inlet, avoiding multiple cells, and increasing the size of the culvert. If multiple cells are unavoidable, provision of a sloping cut on the upstream pier (wall) ends may help to align floating debris with the culvert entrance. More costly measures should be avoided unless there is a definite and serious potential hazard.

A relief culvert passing through the embankment at a higher level than the main culvert permits water to bypass the latter if it becomes blocked. The relief culvert could also be placed at a low level some distance away from the main culvert where it is not likely to be blocked. If this relief culvert is an additional requirement, the total cost of both culverts should be compared with that of a larger culvert that will be less subject to blockage.

Debris control structures can be costly both to construct and maintain. Details of the various types of debris control structures may be found in Bradley, Richards and Bahner (2005), which provides information on debris accumulation and the various debris control countermeasures available for culvert and bridge structures and:

- presents various problems associated with debris accumulation at culvert and bridge structures
- provides a procedure for estimating the potential of debris accumulating at a bridge structure
- provides general guidelines for analysing and modelling debris accumulation on a bridge structure to determine the impacts that the debris would have on the water surface profile through the bridge structure and the hydraulic loading on the structure.

Various types of debris countermeasures for culvert and bridge structures are also discussed.

The choice of structure type depends upon size, quantity and type of debris, the cost involved and the maintenance proposed. However, for existing culverts which are prone to debris clogging, it may be worthwhile to construct a debris control structure rather than replace or enlarge the culvert.

Experience has shown that the following combination of stream characteristics tends to produce the most serious debris problems:

- susceptibility of stream to flash floods, i.e. relatively impervious watersheds with moderate or steep gradients
- actively eroding banks bordered by trees or large shrubs
- relatively straight unobstructed stream channels with no sharp bends
- cleared land upstream with fallen trees on the ground.

3.5 Water Sensitive Design

Designers are encouraged to take a holistic approach to the management of water, particularly on larger projects which have a large catchment area and influence significant watercourses. In some cases road designers will be required to undertake water sensitive road design analysis and to use appropriate models to define the existing conditions and to estimate pollution loads prior to construction.
The purpose of the treatment of pollutants in road run-off is to bring the effluents within the range of standards prescribed by the relevant environmental authority having regard to the eventual use of the stormwater. The objective is to provide a long-term solution to stormwater run-off quality for road projects. In order to achieve this goal, drainage designers need to consider strategies to:

- reduce pollutants and sediments in the stormwater run-off from roads before discharge into receiving waters
- control the outflow of stormwater so as not to erode or cause siltation of the receiving waters
- reduce the volume of stormwater to be discharged.

Studies around the world have shown that run-off from roads can have a detrimental impact on receiving waters and the aquatic life they sustain. These contaminants include:

- particulate matter
- nutrients (nitrogen and phosphorus)
- heavy metals
- petroleum-based products
- organic compounds
- rubber products.

The treatment of road run-off is an important element of catchment management, owing to the expected high pollutant concentrations of metals and hydrocarbons generated from road surfaces. Treatment of road run-off is primarily directed at the removal of suspended solids and associated contaminants such as nutrients and heavy metals.

Roads and other transport-related surfaces can constitute up to 70% of the total impervious areas in an urban catchment and these road surfaces contribute considerably larger pollutant loads compared with other land uses. In many studies correlations have been made between the amount of pollutants generated and the road traffic volume (Wong, Breen & Lloyd 2000).

The contribution of a road’s impact on increases in stormwater run-off from a catchment varies depending on the road design, location and whether it traverses an urban or regional catchment.

### 3.5.1 Urban Stormwater Management Principles

Flood prevention and public safety remain fundamental objectives in the planning and design of any stormwater drainage system, and stormwater treatment elements should in no way compromise these objectives.

### 3.5.2 Water Sensitive Urban/Road Design

Water sensitive urban design (WSUD) is the integration of the water cycle into urban planning and the utilisation of best practice techniques to achieve sustainable water and ecological resource management. While WSUD was initially an additional requirement to the traditional conveyance approach to stormwater management, it is now an integral part of the drainage system. It seeks to minimise the extent of impervious surfaces and mitigate changes to the natural water balance.

The application of WSUD into the planning and design of road networks is referred to as water sensitive road design (WSRD). Whether building a new road or upgrading an existing road, it is necessary to improve the quality of the stormwater run-off from that road. This needs to be incorporated into a project at the scoping and design stages and requires best practice objectives to be met where practicable and feasible.
Roads may represent a relatively small proportion of the total catchment, but they sometimes contribute significantly to water quality concerns. This is especially the case of erosion during construction and on roads with high traffic volumes, where a number of different contaminants may be produced. Between rainfall events, contaminates can build up and then run off at a greater rate than normal into receiving waters.

A key element of WSUD is that urban stormwater should be managed both as a resource and for the protection of receiving water ecosystems. WSUD is a particular issue for urban planning and design, but the key principles of WSUD are also applicable to road infrastructure in the rural environment. These principles are:

- protect existing natural features and ecological processes
- maintain the natural hydrologic behaviour of catchments
- protect water quality of surface and groundwaters
- integrate water into the landscape to enhance visual, social, cultural and ecological values.

### 3.5.3 What is Best Practice?

Best practice is the application of sustainable planning and management practices in the management of stormwater run-off to protect the downstream environment. The aim is to manage or treat the stormwater run-off to meet the water industry’s performance objectives using the best available technology in a practical and feasible way. Best practice can be described as the best combination of techniques, methods, processes or technology used in an industry sector or activity that demonstrably minimises the environmental impact of that sector or activity.

Road agencies need to be aware of all water-related issues, not only in the road reserve, but also both upstream and downstream. In situations where the road crosses a waterway, or substantial fill is required for road construction in a floodplain, there may be significant impact on the conveyance of waterways or drains, with resultant damage to infrastructure including bridges and culverts. There are three primary principles that form the basis of stormwater management:

1. **Preservation**: Preserve existing values of stormwater systems, such as natural channels, wetlands and stream-side vegetation.
2. **Source control**: Limit changes to the quantity and quality of stormwater at or near the source.
3. **Structural control**: Use structural elements to improve water quality and control stream flow discharges.

These principles should be applied as part of an ordered framework to achieve best practice environmental management.

### 3.5.4 Performance Objectives

The *Urban Stormwater: Best Practice Environmental Management Guidelines* (BPEMG) (Victorian Stormwater Committee 1999) identifies and sets out the performance objectives that should be used to assess treatment elements when designing water sensitive road design (WRSD) for a project. Performance is measured by the WSRD elements/treatment trains ability to meet annual pollutant load reduction targets, from typical urban loads. These objectives are:

- 45% reduction in total nitrogen (TN) of a typical urban load
- 45% reduction of total phosphorus (TP) of a typical urban load
- 80% reduction in total suspended solids (TSS) of a typical urban load.

Note: These objectives may vary between jurisdictions or even region to region and should be used as a guide only.
3.5.5 MUSIC Model

By using a computer model, e.g. MUSIC (eWater CRC 2012), or specially designed graphs it is possible to estimate if the WSRD elements designed for a project will meet these performance objectives. These are further discussed in VSC (1999) and Melbourne Water (2005b). It must be noted that this model cannot be used to undertake hydraulic design.

3.5.6 Key Design References

There are a number of key references available across Australia and New Zealand to a designer for the planning, selection, design, construction and maintenance of treatment elements and include:


Auckland Regional Council (ARC) 2003, *Stormwater management devices: design guidelines manual* (TP10), Auckland Regional Council, NZ.


Western Australia Department of Water 2004, *Stormwater management manual for Western Australia 2004–2007*, Western Australia Department of Water, Perth, WA.


Wong THF (ed) 2006, *Australian runoff quality: a guide to water sensitive urban design*, Engineers Media, Crows Nest, NSW.

There are a number of consultancy organisations and experts available to undertake, or advise on the planning, design and construction of WSRD elements.

3.5.7 Selecting Treatment Elements

Selection of appropriate treatment elements depends on the site situation, constraints and treatment requirements. Jurisdiction guidelines, VSC (1999) and Melbourne Water (2005b) describe in detail suitable elements for road projects and should be referred to in selecting treatment elements. As a general rule, site conditions and target pollutant characteristics influence the selection of the appropriate type of treatment measure while climatic conditions (amount of rainfall) influence the hydrologic design standard (size) for the treatment measure. There are a number of treatment options suitable for the treatment of road run-off and managing peak flows.

Below is a brief summary of the main types used and their advantages and disadvantages. Further details and the application, suitability, design and maintenance of these treatment options can be found in Melbourne Water (2005b), VSC (1999) and local and state road agency and environment protection authority, guidelines and manuals.
Assess potential pollutant control devices

Each potential pollutant control device needs to be assessed to determine if it is suitable for the site conditions. Each pollutant control device can be accepted or rejected on the basis of screening criteria to provide a short list. Table 3.4 provides a means of assessing common design elements in order to determine if a particular control device is suitable for a specific site condition.

Table 3.4: Design elements associated with treatment devices

<table>
<thead>
<tr>
<th>Pollutant control device</th>
<th>Area served (ha)</th>
<th>Head requirement</th>
<th>Capital cost</th>
<th>Maintenance cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oil grit separators</td>
<td>&lt; 1</td>
<td>Low</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
<tr>
<td>Gross pollutant trap</td>
<td>5–5000</td>
<td>High</td>
<td>High</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Trash rack</td>
<td>20–500</td>
<td>Low to moderate</td>
<td>Moderate</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>Downward inclined screen</td>
<td>5–500</td>
<td>High</td>
<td>Moderate to high</td>
<td>Low to moderate</td>
</tr>
<tr>
<td>Extended detention basin</td>
<td>&gt; 5</td>
<td>Low</td>
<td>Low to moderate</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Sand filter</td>
<td>1–50</td>
<td>High</td>
<td>Moderate to high</td>
<td>Moderate to high</td>
</tr>
<tr>
<td>Filter strips</td>
<td>&lt; 2</td>
<td>Low</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>Grassed swales</td>
<td>&lt; 5</td>
<td>Low</td>
<td>Low</td>
<td>Low</td>
</tr>
<tr>
<td>Constructed wetlands</td>
<td>&gt; 2</td>
<td>Low to moderate</td>
<td>High</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

Notes:
From 0–5% slope preferred but the range can be extended beyond 5%. Buffer zones should only be extended beyond 5% with careful design. Source: Derived from EPA (1997) and Mudgway et al. (1997).

Buffer strips

A buffer strip is a densely vegetated area separating the road from the receiving stream (generally by at least 20 m), through which stormwater passes as overland flow. The length of buffer strip, slope, vegetative characteristics, catchment characteristics and run-off velocity are all factors that may affect the pollutant removal efficiency of buffer strips. The method is not so effective when slopes are steep, since steep slopes lead to the development of rills and scour and subsequent erosion of the slope.

A buffer strip can achieve removal of most of the coarse sediment, regardless of the initial sediment load. However, it is not so effective with respect to fine particles. The removal of total phosphorus decreases significantly with increasing sediment input load and higher flow rates. Buffer strips are not suitable for removal of heavy metals, polycyclic aromatic hydrocarbons and nutrients.

Buffer or vegetative filter length must be adequate and flow characteristics acceptable or water quality performance can be severely limited. Furthermore, because the filtering occurs as water flows overland, and no pond is involved, smaller particles may not settle during normal flows and re-suspension of sediment may occur during high flows.

Swales

The location and side slopes of swales should be designed in accordance with AGRD Part 3 (Austroads 2010c). Longitudinal gradient should ideally be between 2% and 4% to promote uniform flow conditions. The foreslope and backslope of the drain should comply with guidance given in AGRD Part 6 (Austroads 2010d). The width of the swale base should be not more than 2.5 m, unless measures are used to ensure uniform spread of flow.
Maximum flow velocity should be less than the scouring velocity of the soil shown in Table 3.5 or less than 0.5 m/s (whichever is the smaller) for the one year ARI event and not more than 1.0 m/s for the 100 year ARI event.

Worked examples can be found in Austroads (2001), Melbourne Water (2005b) and Wong (2006).

**Litter traps**

Litter traps in drainage systems capture large objects such as cans, bottles, plastic bags, packaging and rotting vegetation that are generally described as gross pollutants. Litter traps are the first stage of a water quality treatment sequence. It is important that there be a systematic maintenance program, or both the trapping and the hydraulic functions may fail.

A gross pollutant trap (GPT) is a permanent structure situated within a drain as a primary water treatment upstream of other facilities. The GPT is designed to collect litter by means of a trash rack, and to settle coarse sediment in a sediment basin. Gross pollutant traps generally have negligible effect on silt and clay particles or dissolved chemicals. Some GPTs are designed specifically to capture oils, but many are not.

**Table 3.5: Maximum velocity over surface**

<table>
<thead>
<tr>
<th>Soil description</th>
<th>Maximum velocity (m/s)</th>
<th>Soil description</th>
<th>Maximum velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare earth surfaces</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>0.4</td>
<td>Sandy loam</td>
<td>0.5</td>
</tr>
<tr>
<td>Silty loam</td>
<td>0.6</td>
<td>Stiff clay</td>
<td>1.0</td>
</tr>
<tr>
<td>Fine gravel</td>
<td>0.8</td>
<td>Coarse gravel</td>
<td>1.0</td>
</tr>
<tr>
<td>Vegetative grass covered surfaces</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Grass type</td>
<td>Slope (%)</td>
<td></td>
<td>Maximum velocity (m/s)</td>
</tr>
<tr>
<td>Grass mixtures</td>
<td>0–5</td>
<td>1.5</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>5–10</td>
<td>1.0</td>
<td>0.8</td>
</tr>
<tr>
<td>Kikuyu</td>
<td>0–5</td>
<td>2.5</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>5–10</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>&gt; 10</td>
<td>2.0</td>
<td>1.8</td>
</tr>
<tr>
<td>Couch, bent or fescue</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>0–5</td>
<td>2.2</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>5–10</td>
<td>2.0</td>
<td>1.8</td>
</tr>
<tr>
<td></td>
<td>&gt; 10</td>
<td>1.8</td>
<td>1.5</td>
</tr>
</tbody>
</table>


**Sedimentation basins**

Sedimentation basins may be installed temporarily during road construction, or may be designed as part of a permanent water treatment process. A single basin is used for temporary works, but a permanent basin is more efficient if partitioned into two segments, one for coarse sediment and the other for the fine particles. See AGRD Part 5A – Section 7 for further details on the design of permanent basins within the road reserve. Further details can also be found in Melbourne Water (2005b). Sedimentation basins are designed to remove suspended solids from run-off. As a number of pollutants are often attached to soil particles, a basin will also help to remove pollution. A typical layout is shown in Figure 3.5.
Figure 3.5: Typical sedimentation basin

Melbourne Water (2005b) outlines a process for the design of a sedimentation basin and explains the steps involved in producing an efficient layout. A number of factors can reduce the efficiency of a sedimentation basin and these are also described.

The following factors reduce the efficiency of a basin:

- short circuiting of the flow from inlet to outlet
- turbulence
- bottom scour
- poor outlet design
- temperature currents
- wind currents.

Of these, turbulence has the greatest detrimental effect, and several smaller basins may be more effective than a very large basin. Turbulence may be reduced by attention to the following details:

- maintaining a low flow velocity through the basin
- eliminating unnecessary changes in the direction of the flow
- keeping surface areas to a practical minimum, and shielding from wind.

Sedimentation basins are designed to catch the sediment and will fill up with sediment over time. Consideration should be given to the need to access each basin for cleaning, and care is needed to ensure that the access road itself does not become a source of sediment. Depending upon the slope of the batters, cleaning equipment may not need any special access provisions. At each permanent sedimentation basin, the access track should allow safe all year round access.

**Infiltration basins**

An infiltration basin is an impoundment that is designed to infiltrate stormwater into the groundwater. The sides of the impoundment are prepared with batter slopes that remain stable in the presence of standing water. Typically, an infiltration basin will be dry for most of the time and will only fill during periods of rainfall. An infiltration basin associated with road reserves has two purposes:

- disposal of stormwater run-off from the vicinity of roads to prevent flooding
- a method of treating stormwater so that it can be used in a number of environmentally beneficial practices.

There are four approaches to the design of infiltration basins:

- for a storm event with a specific ARI
- to catch a certain amount of run-off
- to attenuate channel (stream) discharge
- for risk management.

Typically, on-site detention and infiltration basins are designed with an ARI of between 2 to 10 years as applicable to a minor drainage element. This would satisfy requirements for an off-line stormwater management device, with excess flows from larger storm events bypassing the basin. The size of the overflow facilities will depend upon the design ARI selected. The overflow facilities should be checked for the appropriate major design event.
Infiltration is believed to have high pollutant removal efficiency and can also help recharge the groundwater, thus restoring low flows to stream systems. Infiltration basins can be challenging to apply on many sites because of soil requirements.

Permeable pavements have also been used but this form of construction requires good drainage conditions to ensure adequate subgrade bearing capacity and some road agencies may have reservations about the structural capacity of these pavements for use on major roads.

The time required to clean the pre-treatment facilities needs to be compared against the time required to clean the infiltration basin. In some situations it may be just as easy to clean out the basin rather than having to clean out a trash rack and the sediments from the basin.

Design considerations for infiltration basins are contained in Melbourne Water (2005b). A worked example for an infiltration basin is described in Appendix A.

Further information on drainage basins can be found in AGRD Part 5A – Section 7.

3.5.8 Bio-retention Systems

A schematic diagram of a bio-retention treatment system is shown in Figure 3.6.

**Design principles**

The primary design objective is to maximise the detention time in the filter medium. A minimum detention of 12 hours is recommended.

If the water table is too high or the filter medium has a high hydraulic conductivity, consideration should be given to placing a layer of material with a lower hydraulic conductivity at the base of the filter zone to ensure that the water quality meets the target values.

The depth of the soil filter and the provision of above-ground detention storage are the two principal design considerations when specifying the dimensions of the bio-retention zone. Three equations define the operation of this zone:

- the hydraulic residence time
- the surface detention storage
- the maximum infiltration rate.

The hydraulic residence time ($T_{HRT}$) in the bio-retention zone is made up of detention above the filter zone and the time taken through the filter medium. Hydraulic residence time should be checked for:

- maximum depth of inundation
- minimum depth of inundation.

For further information designers are also referred to Schueler (1987) and the Guideline Specifications for Soil Media in Bio-retention Systems developed by the Facility for Advancing Water Biofiltration (FAWB 2009).
3.5.9 Wetlands

A wetland is a permanent, shallow water body with extensive emergent vegetation. A schematic diagram of a wetland system is shown on Figure 3.7. An ephemeral wetland does not have a permanent water body, but is wet often enough to support aquatic vegetation. Wetlands improve water quality by settling the finer sediments, binding and adsorbing heavy metals and by taking up nutrients such as nitrogen and phosphorus in growing plants. A guide to layout, sizing and planting of constructed wetlands may be found in Melbourne Water (2005a) and the Western Australia Department of Water (2007).

The use of constructed wetlands varies from between jurisdictions and depends on the requirement to treat the road run-off specifically or greater parts of the catchment. Wetlands require continual/regular inflow to maintain minimum water levels to ensure the survival of aquatic vegetation and the level of treatment to be maintained, and therefore catchments that are purely based on road run-off rarely provide enough run-offs to maintain these water levels. Some areas of Australia and New Zealand which receive regular rainfall may provide the continual inflow required.

The lower chart in Figure 3.8 shows that if excess nitrogen is not dealt with at source, the area of wetland required to attenuate it is much greater than for total suspended solids (the upper chart) and the removal efficiency is quite low.

Where grated pits are provided in wetlands, bar spacings of about 60 mm may be suitable in remote locations. Where the pit may be traversed by pedestrians and bicycles, a bar spacing of 20 mm with the bars aligned at right angles to the direction of traffic may be suitable.
Figure 3.7: Schematic plan and profile view of a typical constructed wetland

Note: Image supplied courtesy of Melbourne Water.
3.5.10 Treatment Train

A treatment train is a series of treatment elements designed to collectively meet the treatment objectives, e.g. a gross pollutant trap used in conjunction with a biofiltration trench. Implementing a treatment train using structural treatment elements allows various types of pollutants and particulate sizes to be trapped.
For a road project this may involve designing a treatment train that includes grass buffers, vegetated swales or biofiltration trenches which then discharge to a traditional drainage system or a waterway. At each stage different size particles and contaminants may be removed. Comprehensive software has been developed by the Cooperative Research Centre for Catchment Hydrology at Monash University for the modelling of catchments and development of stormwater treatment strategies which can provide detailed information, e.g. MUSIC (eWater 2012).

**Obtain hydrological data**

Prior to commencing the process illustrated in Figure 3.4, the designer will need to obtain hydrological data. The pollution concentration at a point depends upon cumulative pollution transport in a series of frequent rainfall/run-off events, so rainfall records for the catchment are required, or must be simulated. While rainfall of low average recurrence interval such as a one year ARI may be used for preliminary sizing of a treatment device, a single event does not satisfactorily define the effect of that device.

**Monitor existing water quality**

For major road projects it may be desirable to monitor water quality in affected streams for up to a year before commencement of construction. Tests commonly included are for suspended solids, dissolved solids, pH, and turbidity.

Where the road receives effluent from farmland, tests may have to be made for components of fertilizers, pesticides, insecticides and animal faeces. In such cases, negotiations should be made with the environmental authority to have these pollutants dealt with at the source rather than at the expense of the road agency.

**Estimate pollution loads during construction**

Construction activity, especially when the earthworks have no vegetative cover or other protection, may result in significant soil erosion. The soil loss may be estimated using the universal soil loss equation, or a similar method found in Book 2, Appendix E3 of the International Erosion Control Association (IECA 2008), or can be calculated using a computer-assisted method such as SOILOSS (Rosewell & Keats 1993).

**Estimate pollution loads during operation**

Pollution loads from road operation are partly a function of traffic volume. Estimates of pollution loads may be obtained from Wong, Breen and Lloyd (2000) or Wong (2006). On major roads, consideration should be given to the possibility and treatment of toxic spills.

**Project-specific water quality standards**

On major road projects, or where projects are close to significant water sources, the relevant water resource management agency, catchment management authority or other relevant authority may require adoption of specific water quality standards, taking into account the amenity and proposed use of the run-off. Elsewhere, generic standards may apply.

Catchment management authorities may be concerned about the levels of sediment, phosphorus and nitrogen in road run-off. The concentrations of heavy metals are not easy to test for, but it is assumed that they are adsorbed to the sediments, and that if both large and small sediment particles are trapped then the heavy metals are also satisfactorily removed from the effluent. Computer simulation of a proposed basic pollution treatment train typically shows that extra devices are necessary for nitrogen removal. An example of pollution reduction targets is provided in Section 3.5.4 – Performance Objectives.
Consider treatment options

The method of treatment and its location depends on the scale of the pollution load and the availability of land for treatment devices. Where new subdivisions are being laid out, some land should be specifically reserved for drainage swales and treatment areas. Ideally, pollutants should be captured and removed at or near their source. They may otherwise be captured in-transit or at the end of the drainage path. Typical treatment devices are listed in Table 3.6.

Table 3.6: Locations and types of treatments

<table>
<thead>
<tr>
<th>Location</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Source controls</td>
<td>Community awareness</td>
</tr>
<tr>
<td></td>
<td>Land use planning and regulation</td>
</tr>
<tr>
<td></td>
<td>Street cleaning</td>
</tr>
<tr>
<td></td>
<td>Isolation of sources of high pollution</td>
</tr>
<tr>
<td></td>
<td>Buffer strips</td>
</tr>
<tr>
<td></td>
<td>Litter traps</td>
</tr>
<tr>
<td></td>
<td>On-site detention basins</td>
</tr>
<tr>
<td></td>
<td>Permeable pavements</td>
</tr>
<tr>
<td></td>
<td>Stormwater infiltration systems</td>
</tr>
<tr>
<td>In-transit controls</td>
<td>Swale drains</td>
</tr>
<tr>
<td></td>
<td>Bio-retention filters</td>
</tr>
<tr>
<td></td>
<td>Gross pollutant traps</td>
</tr>
<tr>
<td></td>
<td>Separators</td>
</tr>
<tr>
<td></td>
<td>Sedimentation basins</td>
</tr>
<tr>
<td></td>
<td>Wetlands and ponds</td>
</tr>
<tr>
<td>Outlet controls</td>
<td>Gross pollutant traps</td>
</tr>
<tr>
<td></td>
<td>Floating booms</td>
</tr>
<tr>
<td></td>
<td>Flood detention basins</td>
</tr>
<tr>
<td></td>
<td>Wetlands and ponds</td>
</tr>
</tbody>
</table>

In relation to source controls it should be noted that:

- Community awareness programs promote the responsible disposal of pollutants rather than tipping them into the drainage system.
- Land use planning can isolate point sources of pollution from other sensitive areas, and also provide the space for storage and treatment of pollutants.
- In rural and semi-rural areas, consideration should be given to diverting the road alignment, if practical, to avoid or minimise encroachment on environmentally sensitive areas or water resources.
- A regular street cleaning program can prevent significant amounts of sediment, road grit and vegetation from entering the stormwater system.
- A bund could be placed around fuel or oil storage tanks to retain any spillage.
- Buffer strips are areas of 20 m to 100 m in width with thick ground cover and other vegetation placed between a run-off source and the receiving water (see also Section 3.5.7).
Select the treatment train

There is no single device that can economically treat stormwater for all pollutants. Road run-off requires several treatment stages, depending on the nature of the pollutant load and the water quality standard to be achieved for the end use of the water.

A typical comprehensive treatment sequence would be:

- capture of litter and large fragments from traffic crashes
- trapping of coarse sediments
- settlement of finer sediments
- adsorption of a proportion of dissolved solids in wetlands
- aeration of water in large ponds to remove the biological oxygen demand (BOD).

Not all of these steps may be taken in practice, or they may occur at different locations in the system. Selection of the appropriate treatment for a specific location will require consideration of a range of factors including capital and maintenance costs, and the need for fencing to restrict access of persons to bodies of water.

Typical treatment measures include vegetated buffer strips, swales, litter traps, sediment basins, wetlands and regional lakes. For effective operation of devices a regular maintenance program is essential. Also, they should be inspected and maintained after large storm events.

It must be recognised that there may be extensive land requirements involved in providing treatment solutions for effective pollutant removal, together with ongoing and possibly substantial maintenance costs. These should be explicitly considered in the decision making process.

Where control of the peak flow of water is required as well as control of pollution, a detention basin may be added. Provision of a separate detention basin is now rare, because volume can often be added above a wetland for this function. However, there are many types of wetlands and this approach may not be available if the wetland is environmentally sensitive.

Preliminary sizing of devices

Before proceeding to computer simulation of the effect of the selected treatment train, it is necessary to estimate preliminary sizes and ensure that the devices can fit in the area available. If not, the components selected for the treatment train need to be reviewed. Methods for design of various devices are set out in Wong, Breen and Lloyd (2000) and Alderson (2006). At this stage, consider the requirements of the municipality, the landscape architects and other stakeholders.

Analyse effects of treatment

Some catchment management authorities will accept the design of devices in accordance with Austroads (2003), provided that they are sized to meet the specified water quality objectives. Others will require a computer simulation to demonstrate that the proposed treatment train is likely to achieve the required results.

A typical program, MUSIC, is available from eWater website (eWater CRC 2012). The program simulates pollution generation and transport, but cannot undertake hydraulic design.
**Review treatment train**

Should the treatment train not achieve the required results, some components may have to be increased in size, or different devices added to the train, or a totally new treatment train devised. The process is then repeated until a satisfactory result is obtained.

In cases where satisfactory results are not possible, further negotiations with the local Environment Protection Agency and the Catchment Management Authority will be required.

### 3.5.11 Maintenance and Disposal

The costs associated with maintenance and disposal of collected material are an important aspect of the type and sizing of the treatment device/s used. If periodic maintenance and cleaning requirements require specialist machinery and/or are expensive, they are unlikely to be undertaken regularly, if at all. This will result in the device failing to operate properly and may cause blockages in the system.

Devices designed to collect sediments may contain a high level of contaminants and pollutants. The regulatory requirements for the disposal of this material will vary between jurisdictions.

Jurisdictions should ensure that proper maintenance procedures are developed and undertaken for all treatment devices constructed and that these are written into maintenance requirements. See the reference list cited in Section 3.5.6 – Key Design References and Section 5 – Operations and Maintenance, for further maintenance considerations.

### 3.6 Erosion and Sediment

#### 3.6.1 General

Soil characteristics are a major determinant of soil erosion risk, and hence should be collected during the early stages of the planning and design process. Field inspections can quickly identify existing erosion and sediment issues along an existing or proposed road corridor. Investigations can be completed to ascertain the causes of the erosion by:

- discussing the problems with maintenance personnel, local residents and land owners
- reviewing maintenance procedures
- completing site measurements, sampling and testing soil properties
- reviewing hydraulic and drainage calculations for the existing designs to determine flows and velocities.

The activities that cause erosion include:

- stripping of vegetation and loosening of soil, followed by heavy rains and strong winds
- diversion and concentration of streams or drains that increase flow velocity
- construction work in or near streams
- discharges from construction sites
- release of groundwater flows.

Soils data is collected for drainage planning and design purposes from catchment locations, drainage channels, eroded areas and the road alignment to obtain soil properties that can be used to determine the suitability and limitations of the soils in service in the following situations:

- flow limitations for inlets and outlets to cross drainage
- open channel permissible velocities
• stability or the need for lining of longitudinal drainage
• slopes to cut and fill batters.

Soils information can be obtained from:
• published soil maps and reports
• government departments responsible for environment and resource management, and CSIRO soil databases
• field investigations at each site.

It is important to note that in the majority of cases, soils data should be collected for both surface and subsurface layers, as the upper topsoil layer is usually removed and stockpiled prior to construction.

3.6.2 Scour

Scour at inlets

A culvert normally constricts the natural channel, forcing the flow through a reduced opening. As the flow contracts at the entrance to a culvert, vortices and areas of high velocity flow impinge against the upstream slopes of the embankment adjacent to the culvert. Scour can occur just upstream of the culvert entrance as a result of the acceleration of the flow into the entrance. Upstream wingwalls, aprons, cut-off walls and embankment paving assist in protecting the embankment and streambed at the upstream end of a culvert.

The formation of vortices at the culvert entry may also considerably reduce the capacity of culverts by entraining air either continuously or intermittently. Vortices are known to form at low heads and in situations when there is a dead body of water present above the culvert entry. Vortices also pose a safety problem and their formation should be eliminated if at all possible. Elimination of vortices is undertaken through streamlining and the construction of a hood and/or headwalls sufficiently high to prevent circulation of water above the culvert entry. Where these devices are employed, the use of a physical model to confirm their hydraulic performance is recommended.

Scour at outlets

If the flow emerging from a culvert has a sufficiently high velocity and the channel is erodible, the jet will scour a hole in the bed immediately downstream and back eddies will erode the stream banks to form an elongated circular scour hole. Coarse material scoured from the hole will be deposited immediately downstream, often forming a bar across the stream, while finer material will be carried further downstream. The provision of wingwalls, headwalls, cut-off walls and aprons is generally all the protection that is required at culvert outlets.

If an unacceptable scour hole does develop, a decision must be made as to which type of scour protection is suitable for the site, and consideration given to:
• protection of the natural stream bed material
• provision of a flow expansion structure
• provision of an energy dissipating structure.

For discussion on the selection of an energy dissipator, see Section 3.6.10 – Energy Dissipators.

Stream bed protection can be achieved with a concrete apron, rock riprap, or rock mattresses, or concrete filled mattresses. Mattresses should be selected on the basis of information provided in other sections dealing with construction materials. It is important that mattresses are anchored to the cut-off wall or apron at the culvert outlet, to stop them moving downstream. A geotextile filter is usually provided under the mattresses and may also be required under the rock riprap.
Relevant road agency guidelines provide additional information on the use and design of energy dissipators.

The judgement of designers is required to determine the need for scour protection. As an aid in evaluating the need for protection, culvert velocities should be computed and compared with the velocities occurring in the natural stream.

When comparing velocities, it should be noted that in many streams the maximum velocity in the main channel is considerably higher than the mean velocity for the whole channel cross-section. Investigation of scour and outlet protection at similar culverts in the vicinity of the culvert being designed will provide guidance on whether further protection is required. Periodic site visits and inspection after major flood events will also confirm whether the protection is adequate or further protection is required.

Further information on the flow velocities and culvert outlet protection is contained in AGRD Part SB – Section 3.13.

### 3.6.3 Erosion Estimates

An assessment needs to be undertaken to estimate the susceptibility of the soils to erosion. Soil maps will provide some guidance on what to expect at the site. However, a survey of the soils at the proposed site is essential.

It should be realised that a soil with a high potential for erosion will not erode if it is not exposed to water flow or wind action. It is necessary to examine those layers, which will be exposed, either during construction, or at the completion of the project.

The presence of vegetative cover protects the soil from the erosive forces of wind and rain and greatly reduces the run-off volume (retention), promotes infiltration and decreases flow velocities (retardation). The level of erosion protection varies depending on the vegetative cover provided (Table 3.7). Protection can also be provided using various other coverings (Table 3.8).

<table>
<thead>
<tr>
<th>Description</th>
<th>Protection provided</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well established and stable vegetative cover</td>
<td>High</td>
</tr>
<tr>
<td>Grassed areas with or without intermittent taller growth</td>
<td>Intermediate to high</td>
</tr>
<tr>
<td>Lightly timbered area with some grass cover</td>
<td>Intermediate</td>
</tr>
<tr>
<td>Re-establishing growth with soil visible in places</td>
<td>Low to intermediate</td>
</tr>
<tr>
<td>Bare earth with little or no vegetative cover</td>
<td>Low</td>
</tr>
</tbody>
</table>

*Source: Alderson (2006).*
Table 3.8: Cover and management factor

<table>
<thead>
<tr>
<th>Description</th>
<th>Factor (C)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No cover</td>
<td>1.00</td>
</tr>
<tr>
<td>Straw secured with netting</td>
<td>0.20</td>
</tr>
<tr>
<td>Mulch</td>
<td>0.10</td>
</tr>
<tr>
<td>Fibre glass rovings</td>
<td>0.05</td>
</tr>
<tr>
<td>Bitumen emulsion</td>
<td>0.02</td>
</tr>
<tr>
<td>Grass (few trees)</td>
<td>0.04</td>
</tr>
<tr>
<td>Shrubs</td>
<td>0.01</td>
</tr>
<tr>
<td>Trees (few shrubs)</td>
<td>0.01</td>
</tr>
</tbody>
</table>


Erosion estimates are required for the design of erosion protection measures, such as temporary sediment basins during construction (see also Section 3.6.12 – Sediment Transport and Melbourne Water 2005b). The size of the basin may be determined by the volume to be stored between specified clean-out dates rather than by the minimum area required for the sedimentation process.

The method currently used is the universal soil loss equation, or the more recently released revised universal soil loss equation. For a detailed discussion on the use of this approach, see Hu et al. (2001). Both methods require an estimation of a number of factors such as:

\[ R = 164.74 \times 1.118 \times I^{0.644} \]

where \( I \) is the two year, six hour ARI (mm/h).

\[ K \]

Soil erodibility factor found from nomographs relating particle size, soil structure, organic content and permeability. Typical values range from 0.01 to 1.0 t/ha/metric EI units. Low \( K \) values (low erosion) are associated with sandy soils of high permeability and high organic matter. As organic matter declines, \( K \) increases, and hence erosion increases. Decreasing permeability increases \( K \) and granular structured soils have lower \( K \) values than massive or blocky structured soils.

\[ LS \]

Slope length factor, slope factor multiplied by a steepness factor which are usually tabulated against a range of soil types. Typical values range from 0.05 (for swelling clays on short (≤ 12 m), shallow (≤ 2%) slopes) up to 95 (for any soil apart from swelling clays on long (≥ 500 m) steep (≥ 60%) slopes). The slope factor is given by:

\[ S = 16.8 \times \sin(\text{slope as a decimal}) – 0.5. \]

\[ P \]

Support practice factor which compensates for surface treatment practices. Taken as 1.0 for construction sites.

\[ C \]

Cover management factor which allows for erosion protection practices such as provision of vegetation or other erosion protective measures (e.g. mulching) (see Table 3.8 for some common practices).

A detailed method may be found in Book 2, Appendix E3 (IECA 2008) or can be calculated using a computer-assisted method called SOILLOSS (Rosewell & Keats 1993).

Basic calculations will readily show that the area cleared for works should be kept to a minimum. High, steep batters are very vulnerable to erosion. Where possible, earthworks should be advanced in stages, each stage being protected quickly by processes such as hydromulching and in the longer term by vegetation. Typical values are given for erosion losses in Table 3.9.
### Table 3.9: Typical erosion estimates for erodible soils ($A_s$)

<table>
<thead>
<tr>
<th>Description</th>
<th>Annual average loss (tonne/ha/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthworks with 1:2 batter slopes and high cuts and fills (more than 2 m)</td>
<td>300</td>
</tr>
<tr>
<td>Earthworks with low cuts and fills (less than 2 m)</td>
<td>150</td>
</tr>
<tr>
<td>Frontage road earthworks</td>
<td>15</td>
</tr>
<tr>
<td>Steep pasture</td>
<td>5</td>
</tr>
<tr>
<td>Flat pasture</td>
<td>0.5</td>
</tr>
</tbody>
</table>

*Source: VicRoads (2003).*

#### 3.6.4 Erosion and Scour Protection Measures

In all aspects of road construction and operation there is a need to minimise erosion and scour as a result of stormwater. This can be accomplished by ensuring flow velocities are maintained below critical values or by providing some form of protection to the underlying surface where these critical velocities are exceeded.

A culvert usually has less waterway area than a natural channel and is designed to flow faster. However, the velocity in the barrel needs to be kept within limits so that the change in velocity at the outlet will not damage the channel downstream. Table 3.10 shows advisable maximum culvert velocities (column two) and typical maximum velocities before scour will occur in unprotected soils of various types. However, if the channel has good vegetative cover (i.e. is only intermittently inundated) the permissible velocity can be increased.

As may be seen by comparing columns two and three in Table 3.10, there is usually some distance downstream of the culvert where the flow is decelerating and the streambed and banks require protection by rock beaching. Where for some reason, such as a steep culvert slope, the culvert velocity exceeds the allowable value in Table 3.10 column two, an energy dissipator will be required (see Section 3.6.10 – Energy Dissipators).

#### Table 3.10: Desirable maximum flow velocities in culverts or unprotected stream beds

<table>
<thead>
<tr>
<th>Stream bed soil type</th>
<th>Maximum advisable culvert velocity (m/s)</th>
<th>Maximum allowable stream velocity (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silt</td>
<td>1.0–1.5</td>
<td>&lt; 0.3</td>
</tr>
<tr>
<td>Clay, soft</td>
<td>1.0–1.5</td>
<td>0.3–0.6</td>
</tr>
<tr>
<td>Clay, stiff</td>
<td>1.2–2.0</td>
<td>1.0–1.2</td>
</tr>
<tr>
<td>Clay, hard</td>
<td>1.2–2.0</td>
<td>1.5–2.0</td>
</tr>
<tr>
<td>Sand, fine</td>
<td>1.0–1.5</td>
<td>&lt; 0.3</td>
</tr>
<tr>
<td>Sand, coarse</td>
<td>1.0–1.5</td>
<td>0.4–0.6</td>
</tr>
<tr>
<td>Gravel, 6 mm</td>
<td>1.0–1.5</td>
<td>0.6–0.9</td>
</tr>
<tr>
<td>Gravel, 25 mm</td>
<td>1.2–2.0</td>
<td>1.3–1.5</td>
</tr>
<tr>
<td>Gravel, 100 mm</td>
<td>2.5</td>
<td>2.0–3.0</td>
</tr>
<tr>
<td>Rocks, 150 mm</td>
<td>3.5</td>
<td>2.5–3.0</td>
</tr>
<tr>
<td>Rocks, 300 mm</td>
<td>3.5</td>
<td>4.0–5.0</td>
</tr>
</tbody>
</table>

*Source: Derived from DTMR (2010b) and VicRoads (2003).*
3.6.5 Rock Protection

Rock beaching (Figure 3.9) (or other form of protection) should be used whenever the flow velocity is likely to erode an exposed surface. Beaching is most often applied to channel beds and banks but is equally applicable to spillways, levees and the like. The nominal size of rock protection (beaching) shown on drawings may be $d_{50}$, the average diameter of stone, or may be the rock class specified by weight. Standard classes and thickness of rock slope protection are set out in Table 3.11 and Table 3.12. The stone should be reasonably well graded throughout the thickness. The stone must be hard, dense, durable and resistant to weathering.

Where necessary, a filter layer should be placed between the embankment fill and the rock protection to prevent fine material from being washed out through the voids of the face stones. Geotextile fabrics have generally replaced sand/gravel filters in road works.

Figure 3.9: Detail of rip rap protection (major channels and culverts) and beaching (channels and culverts less than 6 m² in cross-sectional area)

On culverts where the waterway cross-sectional area is less than 6 m², smaller-sized stones known as beaching are used to protect the culvert entrance, and the stream bed and banks. For larger waterway areas, specialist hydraulic engineering advice should be obtained.

The average size of stone used in beaching may be estimated from (Equation 1):

$$
d_{50} = 8550 \frac{(Q \times S_o^{2.16} \times R)^{0.4}}{p^{0.4}}
$$

where

- $d_{50}$ = 50th percentile rock size (mm)
- $Q$ = Design discharge (m³/s)
- $S_o$ = Channel bed slope (m/m)
\[ R = \text{Channel hydraulic radius (m)} \]
\[ P = \text{Channel wetted perimeter (m)} \]

The design discharge is based on the ARI used in the design of the particular drainage element (e.g. culvert, levee bank, channel, etc.). The beaching should be a densely graded mixture of stone with the specified average size. This calculation does not apply to grouted beaching, or to stones used to fill gabions.

A preliminary guide to length of beaching is (Equation 2):

\[ X = 5 (V_o - V_a) \]

where

\[ X = \text{Length of beaching downstream (m)} \]
\[ V_o = \text{Average outlet velocity (m/s)} \]
\[ V_a = \text{Maximum non-scouring velocity from Table 3.10, Column 3 (m/s)} \]

A more accurate method is to calculate a backwater curve and to apply beaching or geotextile reinforced vegetation wherever the velocity exceeds the allowable non-scouring velocity for the soil type in the streambed, \( V_a \).

**Table 3.11: Design of rock slope protection**

<table>
<thead>
<tr>
<th>Velocity (m/s)</th>
<th>Class of rock protection (tonne)</th>
<th>Section thickness, ( T ) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>(&lt; 2)</td>
<td>None</td>
<td>(_)</td>
</tr>
<tr>
<td>2.0–2.6</td>
<td>Facing</td>
<td>0.50</td>
</tr>
<tr>
<td>2.6–2.9</td>
<td>Light</td>
<td>0.75</td>
</tr>
<tr>
<td>2.9–3.9</td>
<td>(\frac{1}{4})</td>
<td>1.00</td>
</tr>
<tr>
<td>3.9–4.5</td>
<td>(\frac{1}{2})</td>
<td>1.25</td>
</tr>
<tr>
<td>4.5–5.1</td>
<td>1.0</td>
<td>1.60</td>
</tr>
<tr>
<td>5.1–5.7</td>
<td>2.0</td>
<td>2.00</td>
</tr>
<tr>
<td>5.7–6.4</td>
<td>4.0</td>
<td>2.50</td>
</tr>
<tr>
<td>(&gt; 6.4)</td>
<td>Special</td>
<td>(_)</td>
</tr>
</tbody>
</table>

*Source: MRWA (2006).*
Table 3.12: Standard classes of rock slope protection

<table>
<thead>
<tr>
<th>Rock class</th>
<th>Rock size(^{(1)}) (m)</th>
<th>Rock mass (kg)</th>
<th>Minimum percentage of rock larger than</th>
</tr>
</thead>
<tbody>
<tr>
<td>Facing</td>
<td>0.40</td>
<td>100</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>35</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.15</td>
<td>2.5</td>
<td>90</td>
</tr>
<tr>
<td>Light</td>
<td>0.55</td>
<td>250</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.40</td>
<td>100</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>10</td>
<td>90</td>
</tr>
<tr>
<td>¼ tonne</td>
<td>0.75</td>
<td>500</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.55</td>
<td>250</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>35</td>
<td>90</td>
</tr>
<tr>
<td>½ tonne</td>
<td>0.90</td>
<td>1000</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.70</td>
<td>450</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.40</td>
<td>100</td>
<td>90</td>
</tr>
<tr>
<td>1 tonne</td>
<td>1.15</td>
<td>2000</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>0.60</td>
<td>1000</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.55</td>
<td>250</td>
<td>90</td>
</tr>
<tr>
<td>2 tonne</td>
<td>1.45</td>
<td>4000</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1.15</td>
<td>2000</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.75</td>
<td>500</td>
<td>90</td>
</tr>
<tr>
<td>4 tonne</td>
<td>1.80</td>
<td>8000</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1.45</td>
<td>4000</td>
<td>50</td>
</tr>
<tr>
<td></td>
<td>0.90</td>
<td>100</td>
<td>90</td>
</tr>
</tbody>
</table>

\(^{(1)}\) Refer to relevant jurisdictions for specific requirements. Source: MRWA (2006).

3.6.6 Gabions

Gabions are rectangular wire cages used to retain rockfill to serve as retaining structures or surface protection against erosion and scour (Figure 3.10). The baskets can be filled with smaller rocks than would be required for mass rock retaining walls or rip rap protection. Typically, the gabion baskets are 0.5 or 1.0 m thick.

The flexible wire mesh allows the use of gabions on soft ground where more rigid structures would require soil stabilisation or stronger substructures. The highly permeable rockfill releases hydrostatic pressure behind gabion retaining walls and avoids uplift caused by turbulent flows.

Where used as retaining walls, gabions should be designed to be stable as gravity structures.

Where gabions are to be used in polluted, corrosive or marine environments, the specification should require the wires to be coated with a layer of ultra-violet stabilised polyvinylchloride.
3.6.7 Rock Mattresses

Rock filled wire baskets with a thickness of 150 mm to 300 mm are known as rock mattresses. They are used at culvert outlets, to line waterway channels and on embankment slopes. They protect against scour and erosion, and depending upon the soil type, can contend with water velocities of between 2 and 6 m/s. It is usual to specify a geotextile filter between the mattress and the soil.

3.6.8 Geotextiles

A wide variety of geotextiles may be used as filter layers or for slope protection. The use of filter fabrics in subsurface drains is covered in AGRD Part 5A – Section 8.

Some organic textiles such as jute are intended to break down soon after vegetation is established on the slope. Other types, e.g. polypropylene netting, are designed to retain soil and seed and to last through several seasons until shrubs and cover plants are fully established. A woven polyester mattress can provide protection to stream banks or seawalls. It is placed in position then filled with grout, providing flexibility in shape and is easier to construct than rock rip rap.

Geotextiles placed on slopes must be anchored top and bottom and fastened to the slope at regular intervals. Care needs to be taken during construction so that the geotextile is not punctured or torn.

3.6.9 Ground Cover

Establishment of ground cover is essential for erosion protection. The type of ground cover selected should be appropriate to the environment, and should also take into account maintenance requirements. Usually landscape architects will be asked to investigate and advise on native and endemic plants for each project. Slopes near the road should be gentle and grassed. Steep slopes or inaccessible areas should be mass planted with compatible mixtures of hardy native plants.

Ideally, a grass should be easy to establish, drought tolerant, have a low nutrient requirement, a low long-term growth rate, and inhibit weed invasion. Usually, grass mixes for roadwork contain some rye grass, which grows quickly then dies off as other types such as fescues become established. Research is continuing into the use of native grasses for roadwork.
For selection of water-tolerant or wet-dry species such as those required for wetlands, see references such as Melbourne Water (2005b).

### 3.6.10 Energy Dissipators

Where the outflow velocity is greater than can be accommodated by the receiving waters, a form of energy dissipator will be required. Energy dissipators are most often associated with culverts. Some types of dissipators are shown in Figure 3.11.

The erosive forces existing in a natural drainage network may be increased by road construction. This results from the concentration of overland flows, the increase in flow velocities, increased run-off caused by building impervious road pavements and finally the discharging of culverts and spillways into locations susceptible to erosion damage.

Energy dissipators are installed to allow a return to the existing drainage conditions as quickly as possible. Energy dissipators should not be considered in isolation but as part of a larger design system which includes the culvert, channel protection requirements and perhaps a debris control structure. The choice of dissipator also needs to consider tail water depth/critical depth.

The operation of any hydraulic energy dissipator depends largely on expending a part of the energy of the high-velocity flow by some combination of the following methods:

- external friction between water and device
- friction between the water and air
- internal friction and turbulence.

The energy dissipators shown in Figure 3.11 show the general arrangement of four types and are based on dissipators which incorporate methods of expending energy in their operation. Care should be exercised in the application of dissipator types as each site has its own particular peculiarities and problems. Designers need to be flexible in their approach to ensure that the dissipator operates as intended.

Each type of dissipator requires a certain set of conditions in order to operate efficiently, and the final selection should account for such circumstances. Table 3.13 lists a few of the design parameters which need to be considered. The table also indicates several other dissipator types available to the designer. The four types shown have a wide applicability in most situations, relative simplicity of design and general performance. However, very little reliable data are available regarding the performance of any dissipator types, and it is better for the designer to err on the side of caution than to treat the calculated results as sacrosanct.

Energy dissipators require specialist and sometimes complicated maintenance regimes due to clogging from debris, causing system failure, and require increased safety design.

An important parameter in the selection of an appropriate energy dissipator is the Froude Number (Fr) of the outlet flow. Where an outlet has $F_r < 1.7$, a simple apron structure, riprap, or a flow expansion structure will suffice. Where $1.7 < F_r < 3$, riprap basins or horizontal roughness elements basins are appropriate. Where $F_r > 3$, a hydraulic jump basin will be required.

Further discussions on Froude Number can be found in AGRD Part 5B – Section 2.3.

Stream bed protection can be achieved with a concrete apron, rock riprap, or rock mattresses, or concrete filled mattresses. Mattresses should be selected on the basis of information provided in other sections dealing with construction materials. It is important that mattresses are anchored to the cut-off wall or apron at the culvert outlet, to stop them moving downstream. A geotextile filter is usually provided under the mattresses and may also be required under the rock riprap.
Relevant road agency guidelines provide additional information on the use and design of energy dissipators. Further information on energy dissipators in relation to culverts is supplied in AGRD Part 5B – Section 3.13. Specialist technical knowledge should be sought in the design of energy dissipators.

Figure 3.11: Energy dissipator types

Type A (riprap) energy dissipator

Type B (horizontal roughness element) energy dissipator

Type C (forced jump) energy dissipator

Type D (impact) energy dissipator

### Table 3.13: Dissipator limitations

<table>
<thead>
<tr>
<th>Dissipator type</th>
<th>Froude number(7)</th>
<th>Allowable Debris(1)</th>
<th>Tail water (TW)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Silt/sand</td>
<td>Boulders</td>
</tr>
<tr>
<td>Flow transitions</td>
<td>na</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Scour hole</td>
<td>na</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Hydraulic jump</td>
<td>&gt; 1</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Tumbling flow(2)</td>
<td>&gt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Increased resistance(3)</td>
<td>na</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>UABR type IX baffled apron</td>
<td>&lt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Broken-back culvert</td>
<td>&gt; 1</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Outlet weir</td>
<td>2–7</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Outlet drop/weir</td>
<td>3.5–6</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>USBR type III stilling basin</td>
<td>4.5–17</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>USBR type IV stilling basin</td>
<td>2.5–4.5</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>SAF stilling basin</td>
<td>1.7–17</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>CSU rigid boundary basin</td>
<td>&lt; 3</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Contra Costa basin</td>
<td>&lt; 3</td>
<td>H</td>
<td>M</td>
</tr>
<tr>
<td>Hook basin</td>
<td>1.8–3</td>
<td>H</td>
<td>M</td>
</tr>
<tr>
<td>USBR type VI impact basin(4)</td>
<td>na</td>
<td>M</td>
<td>L</td>
</tr>
<tr>
<td>Riprap basin</td>
<td>&lt; 3</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Riprap apron(8)</td>
<td>na</td>
<td>H</td>
<td>H</td>
</tr>
<tr>
<td>Straight drop structure(5)</td>
<td>&lt; 1</td>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>Box inlet drop structure(6)</td>
<td>&lt; 1</td>
<td>H</td>
<td>L</td>
</tr>
<tr>
<td>USACE stilling well</td>
<td>na</td>
<td>M</td>
<td>L</td>
</tr>
</tbody>
</table>

1 Debris notes: N = none, L = low, M = moderate, H = heavy.
2 Bed slope must be in the range 4% < So < 25%.
3 Check headwater for outlet control.
4 Discharge, Q < 11 m³/s and Velocity, V < 15 m/s.
5 Drop < 4.6 m.
6 Drop < 3.7 m.
7 At release point from culvert or channel.
8 Culvert rise less than or equal to 1500 mm.
Note: na = not applicable.

#### 3.6.11 Minimum Energy Loss Structures

Minimum energy loss structures are designed to pass water through a bridge site or a culvert with a low expenditure of energy. They can be described as ‘minimum energy’, ‘constant energy’ or ‘minimum energy loss’ structures. Minimum energy loss structures are very rarely used by road agencies these days as less expensive and more effective options are available.
3.6.12 Sediment Transport

Construction of road drainage infrastructure has the potential to cause environmental harm and subsequent damage to infrastructure if not properly managed. One of the most common environmental impacts of road construction is the erosion of soil and subsequent sedimentation of watercourses, roads or drainage infrastructure. Therefore control of erosion and sediment transport on and from project sites is a responsibility of site managers, project and contract managers, designers and contractors.

Areas subject to erosive flow velocities should be identified and protected by the appropriate control measures. Erosion may be controlled by drainage control measures (Table 3.14) or velocity control measures and devices (Table 3.15). Most of the latter group rely on roughening the flow surface or decreasing the longitudinal gradient of the flow path.

Table 3.14: Measures for drainage control

<table>
<thead>
<tr>
<th>Practice</th>
<th>Velocity range</th>
<th>Application</th>
<th>Intended duration of usage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Channels with impervious linings</td>
<td>High</td>
<td>Protection against high velocity</td>
<td>Permanent</td>
</tr>
<tr>
<td>Chutes</td>
<td>High</td>
<td>Passage of flow down steep slopes or embankments</td>
<td>Permanent</td>
</tr>
<tr>
<td>Diversion banks</td>
<td>Low – medium</td>
<td>Diversion of flow away from disturbed areas</td>
<td>Permanent</td>
</tr>
<tr>
<td>Diversion channel/drain</td>
<td>Medium</td>
<td>Large drainage areas</td>
<td>Permanent</td>
</tr>
<tr>
<td>Erosion control mats/geotextiles</td>
<td>Medium – high</td>
<td>Used to line channels where medium to high velocities are expected</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Outlet protection</td>
<td></td>
<td>Control of erosion at outlets. Reduces velocity and dissipates energy</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Reinforced turf lined channels</td>
<td>Medium – high</td>
<td>Alternative to hard channel linings in urban environment</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Rock lined channels</td>
<td>Medium</td>
<td>Temporary and permanent channels</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Rock mattress channels</td>
<td>High/turbulent</td>
<td>Areas with turbulent flow or high velocity. Can be used as a permanent measure</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Slope drains</td>
<td>High</td>
<td>Conveyance of flow down long or irregular grades</td>
<td>Temporary</td>
</tr>
<tr>
<td>Temporary watercourse crossings</td>
<td>Low – high</td>
<td>Most types of watercourses subject to environmental impacts</td>
<td>Temporary</td>
</tr>
<tr>
<td>Turf lined channels</td>
<td>Low – medium</td>
<td>Alternative to hard channel linings in urban environment</td>
<td>Permanent or temporary</td>
</tr>
</tbody>
</table>

Source: DTMR (2010b).
Table 3.15: Measures for velocity reduction

<table>
<thead>
<tr>
<th>Practice</th>
<th>Application</th>
<th>Intended duration of usage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Check dams:</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• rock</td>
<td>Act as a physical barrier and reduce gradient through ponding.</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>• recessed rock</td>
<td></td>
<td></td>
</tr>
<tr>
<td>• gravel/sand bag</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drop structures</td>
<td>Can be created using rock, timber or geotextile. Reduce gradient of channel,</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td></td>
<td>but require stabilisation at base of drop.</td>
<td></td>
</tr>
<tr>
<td>Lengthen flow path</td>
<td>Where sufficient room exists, velocities may be reduced with subsequent</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td></td>
<td>lowering of invert grade, or through lengthening of the flow path. The</td>
<td></td>
</tr>
<tr>
<td></td>
<td>latter may be achieved using bunds or baffles.</td>
<td></td>
</tr>
<tr>
<td>Surface roughening</td>
<td>Increases roughness through vegetation, gravel or other barriers to shallow</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td></td>
<td>flow. However, this will not always be cost effective.</td>
<td></td>
</tr>
</tbody>
</table>

Source: DTMR (2010b).

Erosion control practices can protect exposed soil or reduce the duration of soil exposure. Sediment-trapping controls may be effective for preventing medium-to-coarse sediment like sand and silt from entering watercourses and drainage lines. However, erosion controls are designed to prevent or minimise the amount of sand, silt, clays and fine sediment from being removed or transported from a site. Hence, erosion control should be used to prevent on-site degradation and the occurrence of sedimentation, whilst sediment controls are more useful for the prevention of off-site transport and impacts associated with deposition of already-eroded, coarser particles.

Logically, sedimentation is a product of erosion, therefore minimising erosion will ideally assist in the efficacy of sediment controls. Therefore, erosion prevention or minimisation should be a primary focus of on-site drainage. Measures for controlling erosion are outlined in Table 3.16.

Sedimentation management devices and controls should be installed to retain mobilised sediment on site and to prevent sediment from entering waterways. Hence, sediment controls are selected only after first choosing effective drainage, velocity and erosion controls. Sedimentation management and control measures are outlined in Table 3.17.
Table 3.16: Measures for erosion control

<table>
<thead>
<tr>
<th>Practice</th>
<th>Application</th>
<th>Intended duration of usage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chemical surface stabilisers(1)</td>
<td>Dust control on large areas such as haul roads and access tracks. Some products can be used as an alternative to erosion control blankets.</td>
<td>Permanent or temporary depending on construction materials</td>
</tr>
<tr>
<td>Geocellular confinement system</td>
<td>Three-dimensional honeycomb-shaped mesh that protects unbound or exposed steep cut and fill slopes or earth channels from hydraulic flows, erosion and downward slip of unbound materials.</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Erosion control blankets</td>
<td>Protect against raindrop impact erosion and sheet flow down batters.</td>
<td>Temporary</td>
</tr>
<tr>
<td>Mulching – timber, rock, straw, hydro, bitumen emulsion</td>
<td>Protection against raindrop impact erosion usually as a channel liner when in conjunction with revegetation. Stabilises exposed surfaces. Also reduces moisture loss from the soil.</td>
<td>Left in situ after application</td>
</tr>
<tr>
<td>Revegetation for erosion control</td>
<td>Used on disturbed, cleared or graded areas and stockpiles. Provides surface protection and soil stability.</td>
<td>Temporary or permanent</td>
</tr>
<tr>
<td>Surface roughening</td>
<td>Increases surface infiltration, minimises rutting and reduced wind erosion.</td>
<td>Temporary</td>
</tr>
</tbody>
</table>

1   Chemical stabilisers should be applied with care to prevent any impact on receiving water bodies.
Source: DTMR (2010b).

Table 3.17: Measures for sedimentation control

<table>
<thead>
<tr>
<th>Practice</th>
<th>Application</th>
<th>Intended duration of use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction exits</td>
<td>Removal and trapping of sediment from vehicles leaving site.</td>
<td>Temporary</td>
</tr>
<tr>
<td>Drop inlet and pipe inlet protection</td>
<td>Used to trap entrained sediment prior to entering drain inlet.</td>
<td>Temporary</td>
</tr>
<tr>
<td>Vegetative buffer zone</td>
<td>Includes buffer zones and grassed filter strips. Involves filtering and trapping of sediment run-off in areas of sheet flow. Most effective on sandy soils.</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Rock sediment trap</td>
<td>Placed in gullies and other flow paths to trap entrained sediment. Use to form small sediment ponds.</td>
<td>Permanent or temporary</td>
</tr>
<tr>
<td>Sediment basins</td>
<td>Used to pond water and settle sediment-laden run-off.</td>
<td>Permanent or temporary.</td>
</tr>
<tr>
<td>Sediment fences</td>
<td>Used to reduce velocity and pond run-off to promote settlement of entrained sediment. Not to be used in concentrated flow paths.</td>
<td>Temporary</td>
</tr>
<tr>
<td>Filter cloth or filter media</td>
<td>Can be placed at a point downstream of sediment source or construction site to trap sediment.</td>
<td>Temporary</td>
</tr>
</tbody>
</table>

Source: Adapted from DTMR (2010b).

In addition to these sediment controls, the techniques shown in Table 3.18 and Table 3.19 may be used for de-watering and in-stream sediment control.
With all in-stream sediment controls, amounts of sediment that exceed regional water quality objectives or parameters must be prevented from entering the main body of the watercourse. Clean and dirty water must not be allowed to mix.

### Table 3.18: Dewatering sediment control techniques

<table>
<thead>
<tr>
<th>Practice</th>
<th>Description</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Filter bags and filter tubes</td>
<td>Large reinforced, non-woven, geotextile filter bags attached to the end of a hose or pump.</td>
<td>Only suitable for low to medium flow rates depending on the sediment content of the water and the total surface area of the filter.</td>
</tr>
<tr>
<td>Sediment basin</td>
<td>Typically a temporary above-ground pond formed by an earthen embankment, however can be below ground effectively making it a sediment basin.</td>
<td>The primary action is that of gravitational settlement of the sediment, thus these ponds are usually significantly larger than filter ponds. They are normally used for de-watering large volumes of water at the beginning of each day.</td>
</tr>
</tbody>
</table>

*Source: DTMR (2010b).*

In stream sediment controls must be designed so that no more than 50% of the channel width is obstructed by in-stream sediment controls at any one time. Unpolluted water flows are not to be obstructed.

### Table 3.19: In-stream sediment control techniques

<table>
<thead>
<tr>
<th>Practice</th>
<th>Description and application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floating silt curtain</td>
<td>Geotextile filter fabric (silt curtain) suspended vertically in a water body between contaminated and uncontaminated water.</td>
</tr>
<tr>
<td>Rock sediment trap</td>
<td>A rock dam constructed across a drainage channel or small watercourse. The dam is formed primarily from coarse rock with a filter layer on the upstream face. The upstream filter layer may consist of a geotextile filter cloth or a layer of clean aggregate.</td>
</tr>
<tr>
<td>Staked sediment fence</td>
<td>A standard U-shaped sediment fence staked across a channel. A wire mesh backing is normally placed behind the fabric to provide additional support. Only used as a precaution in minor drains when flow is not expected but sediment flow may still occur due to non-stormwater flow.</td>
</tr>
</tbody>
</table>

*Source: DTMR (2010b).*

### 3.6.13 Managing Sediment within Culverts

Sediment deposits within culverts, especially multi-cell culverts, can cause significant operational and maintenance problems.

Occasionally sediment traps (basins) are constructed upstream of culverts. In these cases, an access ramp for maintenance must be provided to allow de-silting of the trap.

In critical areas, or for long culverts where maintenance is extremely difficult, a small sediment trap/weir can be constructed at the inlet to divert low flows to just one or two culvert cells. This will allow the flow to enter the remaining cells only during high flows. These sediment weirs should be designed to be fully submerged during major flood events so that no adverse backwater effects occur.

It is important that sediment traps are designed so that maintenance activities are able to be undertaken in a practical and economic manner. Factors that should be considered include access to the basin for the removal of sediment and debris, access for low loaders delivering plant, and the type of plant that will be necessary (e.g. the reach of a backhoe compared to an excavator).
3.6.14 Installation of Control Measures

The installation of control measures should be coordinated with the project’s construction works. Measures should be installed prior to and during clearing and grubbing, topsoil stripping and stockpiling, installation of access tracks and compounds, and cut and fill operations.

A summary of the control techniques is contained in Appendix C.

3.6.15 Maintenance of Control Measures

At all times during their required operational life, erosion and sediment control devices must be maintained in proper working order.

**Regular clean out**

The frequency of maintaining and/or replacing control measures should be based on the type of measure, type of soil, type of vegetation used and expected run-off characteristics. The accessibility of a control measure after a rainfall event must also be taken into account when determining the frequency of cleaning and hence the required size of the control. As a general rule, control measures should be cleaned out when 60% full of sediment.

**Repair and replacement**

The second key maintenance requirement includes the repair and replacement of deteriorated materials within the control measure (e.g. sediment fence fabrics). Controls that are continually damaged may indicate the need for additional or alternative controls. If a control measure is to be replaced, its replacement should be chosen based on the compatibility of the objectives.

3.6.16 Removal of Temporary Control Measures

Design documentation should specify that before the removal of all temporary control measures, the site should be inspected and stabilised. Generic guidelines should require that:

- a check for evidence of erosion is undertaken prior to removal to ensure erosion will not occur through the removal, and after the removal of control measures
- removal and disposal of any deposited sediment is carried out in an appropriate manner
- the removal process (e.g. machinery used) does not cause excessive ground disturbance
- any revegetation of the area has stabilised prior to the removal
- tall areas disturbed during the removal process are stabilised.

3.7 Miscellaneous

3.7.1 Backwater

Backwater is the accumulation of water in a stream that can be caused by:

- an obstruction limiting downstream channel capacity (that causes afflux in the stream)
- tidal flow (Section 3.7.2 – Tidal Waters)
- a downstream body of water such as a weir, dam or lake
- a higher-than-normal flow stage in a downstream connecting stream.
The presence of backwater typically causes an upstream section or body of water to have a level that is increased above normal depth. The cause of backwater can be a significant distance away from, but still affect, the site being assessed. Therefore designers should check the downstream conditions of planned sites or designed infrastructure for possible backwater influence.

Calculating the effect of backwater on normal stream flows can be difficult and is beyond the scope of this Guide. If it is considered that backwater could be affecting stream flow, advice and/or assistance should be sought from a hydraulics specialist.

### 3.7.2 Tidal Waters

Backwater as a result of tidal flow can affect stream flow and in turn drainage infrastructure design. When determining backwater effects on tail water levels for streams discharging into tidal waters, three factors may influence the final design level:

- tide levels
- storm surges
- climate change.

#### Sea and tide levels

Tide tables are published by local organisations. This data is used to estimate backwater and tail water levels for drainage designs.

The mean high water spring (MHWS) tide is often applied as a tail water level since this is a reasonable high tide level, but not overly conservative. Spring tides occur approximately once a fortnight at full and new moon. The definition of different tidal levels is provided in Figure 3.12, reproduced from the tide tables.

Storm tides are an increase in ocean levels above the expected tidal levels caused by low pressure and wind effects. Storm tides can occur at any stage of the tidal cycle.

As with other effects, the selection of a storm tide needs to consider the combined risk of occurrence of the storm tide and the flood from the waterway. The probability of a catchment flood at the same time as a storm tide is the product of the probabilities of the separate events, resulting in a much rarer combined recurrence interval than that of the separate events.

Storm tides may have been analysed by relevant local authorities and these levels may be available for usage. It is important to note that:

- mean sea level given for the location may be different to Australian Height Datum (AHD) which is the average mean sea level of 42 locations around Australia
- tide levels are given at or very close to the coastline
- tide levels are often specified in terms of low water datum.

It is very difficult to estimate the tide level at a location in a stream some distance from the coastline. The time for the tide to rise along the creek and to then flow back in the opposite direction is one factor. Another is the existence of local sand bars or raised areas of the creek bed closer to the mouth of the stream, which may prevent a tide from reaching or falling below a certain level at a particular road crossing. The transient nature of sand bars adds to the difficulty in estimating the tidal reach.

Although precise tide levels are not usually necessary at an upstream road or bridge crossing, tide levels at the mouth of the tidal stream are not sufficient to give tide design parameters at the road or bridge.
The absolute minimum survey requirements at a road or bridge site are the times and levels for successive low-high-low tides. Alternatively, successive high-low-high tide information may be found. The more tidal cycles measured the better.

This information when compared with tide levels at the mouth of the creek will allow experienced hydraulic engineers to predict approximate design tides such as MHWS and MLWS (mean low water springs) at the project site.

**Figure 3.12: Tidal planes**

**Notes:**
- **MSL (Mean Sea-Level)** The average level of the sea over a long period (preferably 18.6 years) or the average level which would exist in the absence of tides.
- **HAT (Highest Astronomical Tide)/LAT (Lowest Astronomical Tide)** These are the highest and lowest levels which can be predicted to occur under average meteorological conditions and any combination of astronomical conditions. These levels will not be reached every year. HAT and LAT are not the extreme levels which can be reached, as storm surges may cause considerably higher and lower levels to occur.
- **MHWS (Mean High Water Springs)** Long term average of the heights of two successive high waters during these periods of 24 hours (approximately once a fortnight) when the range of tide is greatest, at full and new moon.
- **MLWN (Mean Low Water Neaps)** Long term average of the heights of two successive low waters when the range of tide is the least, at the first and last quarter of the moon.
- **MLWS (Mean Low Water Springs)** The long term average value of two successive low waters over the same periods are defined for MHWN.
- **AHD (Australia Height Datum)** This Datum has been adopted by the National Mapping Council as the datum to which all vertical control for mapping is to be referred.

Source: Adapted from DTMR (2010b).
3.7.3 Storm Surge

A storm surge is the rise (or fall) of open coast water levels relative to the normal water level and is due to the action of wind stress and atmospheric pressure on the water surface. Storm surges occur as part of major storms such as cyclones where there are low atmospheric pressures and the wind blows over reaches of the ocean.

Some predicted surge heights have been produced by local authorities and this data should be referenced if available. There is no correlation with tide levels, nor are there any predictions for wave break set-up and wave run-up on the land. These factors will need consideration for any design with storm surge as a factor.

Storm surges would need to be considered with respect to coastal developments, the protection of coastal roads, route immunity for evacuation purposes, and for major coastal drainage designs. Local government may also have specific requirements or data in relation to storm surge.

There is a low likelihood of occurrence of storm surge coinciding with flood events, so calculation of design floods does not need to consider both together, but the two effects should be considered independently. The designer should review analysis of meteorological data to ensure the appropriate coincident conditions or the extreme water level conditions are used in backwater analysis.

**Downstream tributary**

If the crossing is located on a stream which joins another watercourse (larger or smaller) downstream, other issues need to be considered.

As the two open channels have different catchment sizes, they will peak at different times. The combined flow at their junction needs to be assessed.

In this case, two situations need to be considered:

- major flood on tributary with limited flow in the main stream
- major flood on the main stream and limited flow in the tributary.

Both cases need to be analysed to provide an understanding of the potential flood conditions at the road. The risk of coincidental flooding in the two streams needs to be considered to determine the combined risk of flooding. Depending on the relative sizes of the two streams, it may not be realistic to expect floods to occur together in the two streams.

3.7.4 Tail Water Levels

Tail water level refers to the normal water level, for a given flow, in a channel immediately downstream of a drainage structure. Details on tail water levels, tail water effects and design levels for tail water are provided in *AGRD Part 5B – Section 2.6*.

3.7.5 Acid Sulphate Soils

Government departments responsible for environment and resource management will be able to provide information and advice on areas dominated by actual and potential acid sulphate soil deposits (e.g. plotted on maps) which may include presence or depth to actual acid sulphate soil horizons and presence or depth to potential acid sulphate soil.

Many road construction activities may cause potential acid sulphate soils to be exposed to air, including:

- stripping of topsoil
- excavation of soil for underground drains and table drains
- excavation of sediment retention basins
- dewatering of cuts
- construction of culverts
- construction of bridge foundations.

Actual acid sulphate soils produce sulphuric acid which dissolves heavy metals in soils, causing damage to both the natural environment and engineering works. Engineering works are affected by sulphuric acid leaching from actual acid sulphate soils, which corrodes iron, steel and aluminium and attacks concrete, causing the concrete to expand, weaken and spall and subsequently exposing reinforcement to corrosion. Engineering works are also affected by the breakdown of soil structure in clays as heavy metals are dissolved in sulphuric acid. Changes in soil structure may reduce the shear strength of the soil and allow subsidence and erosion to occur.

**Management of acid sulphate soils**

When undertaking road construction activities at sites where either potential or actual acid sulphate soils (ASS) have been identified, the preferred management option is to avoid any disturbance to the existing ground surface and groundwater regime. Design of proposed road construction activities should be undertaken based on these restrictions.

If the exposure, excavation or dewatering of acid sulphate soils during road construction activities is unavoidable, either neutralisation, containment or sulphide removing techniques must be adopted to prevent damage to the natural environment and engineering works. Neutralisation is undertaken by the addition of a neutralising agent (usually quicklime) to exposed acid sulphate soils, which removes excess hydrogen ions beyond the capacity of the soil to neutralise the acidity. Containment can be achieved by excavating acid sulphate soils and reburying immediately (to minimise exposure to air) in impermeable excavations below the water table. Sulphide removal techniques involve excavating the acid sulphate soil, removing the iron sulphide using a hydrocyclone, replacing the clean soil and containing the remaining iron sulphides in a suitable manner.

Further information on design and construction through acid sulphate soils can be found on jurisdictional websites such as the DTMR (http://www.tmr.qld.gov.au), and VicRoads *Acid Sulphate Soils* (VicRoads 2006). Other road or environment protection agencies may have their own local requirements.

**Design and construction considerations through acid sulphate soils**

Acid sulphate soils (ASS) occur naturally over extensive low-lying coastal areas, predominantly below an elevation of 5 m AHD. In areas where the elevation is close to sea level, these soils may be found close to the natural ground level but at higher elevations they may also be found at depth in the soil profile.

In areas that have a high probability of containing ASS, local government planning strategies should, as far as practical, give preference to land uses that avoid or minimise the disturbance of ASS. Land uses such as extractive industries, golf courses, marinas, canal estates, and land uses with car parking or storage areas below ground level which are likely to result in significant amounts of excavation, filling (or even de-watering), should be avoided in high probability areas. However, where the ASS occur at significant depth, the previously mentioned land uses may be appropriate if they are unlikely to result in the disturbance of ASS layers. Jurisdictional planning policy requirements should be investigated during the planning and design stages to ensure regulatory requirements are included.
Alternative uses such as open space or wildlife corridors may be allocated to areas with high sulphide concentration. It is preferable to maintain groundwater levels in a steady state. Works to be avoided include:

- construction of drains or canals which unnecessarily lower the groundwater table, either during normal operation or during maintenance works such as de-silting
- construction of drains or canals that may cause significant water level fluctuations during dry periods
- construction of water storages, or sediment/nutrient ponds in acid sulphate soils.

All new drainage works in coastal areas should be investigated, designed and managed to avoid potential adverse effects on the natural and built environment (including infrastructure) and human health from ASS where such works may:

- disturb the groundwater hydrology or surface drainage patterns below 5 m AHD
- disturb subsoils or sediments below 5 m AHD where the natural ground level of the land exceeds 5 m AHD (but is below 20 m AHD).

In situations where the ASS investigation has identified high levels of sulphides in the soil, the design of new drainage works must incorporate appropriate management principles, such as:

- disturbance of ASS to be avoided wherever possible
- where disturbance of ASS is unavoidable, preferred management strategies are:
  - minimisation of disturbance
  - neutralisation
  - hydraulic separation of sulphides either on its own or in conjunction with dredging
  - strategic re-burial (reinterment).

Other management measures may be considered but must not pose unacceptably high risks, such as:

- Appropriate consideration of alternative development sites, and/or alternative sites to locate drains, roads, pipelines and other underground services.
- In situations where avoidance of all ASS is not possible, drainage should be designed so that areas with the highest levels of sulphide are either not disturbed (preferred), or minimally disturbed (where non-disturbance is not practical) and overall ASS disturbance is minimised.
- Wherever practical, drains are designed so that they do not penetrate the acid sulphate soil layers, and preferably the acid sulphate soils are at least 0.5 m below the channel invert.
- Drainage designs must allow construction and ongoing maintenance works to be performed in accordance with best practice environmental management as defined in documents such as the Queensland Acid Sulphate Soil Technical Manual: Soil Management Guidelines (Dear et al. 2002) and jurisdictional regulations requirements.
- Neutralising agents may need to be incorporated into the lining of constructed drainage channels to aid the neutralisation of acidic stormwater run-off, and to neutralise acidic water entering from acidified groundwater inflows. It is inappropriate to apply neutralising agents into natural watercourses or water bodies unless carefully planned and approved. This is particularly important for waters where pH-sensitive wildlife may be present such as in naturally acidic coastal wetlands.
- Drainage designs should not rely on receiving marine, estuarine, brackish or fresh waters as a primary means of diluting and/or neutralising ASS or associated contaminated waters.
- Larger drainage works (e.g. greater than 5000 tonnes of soil disturbance) should be staged to ensure that the disturbance is manageable.
In addition, the design of drainage systems in new urban development should give preference to:

- firstly, open channels with inverts at least 0.5 metres above ASS layers
- secondly, open channels with inverts above ASS layers
- thirdly, piped drainage systems that discharge directly to open waterways that will allow maximum dilution of acid waters
- fourthly, piped drainage systems that discharge directly to existing open channels or waterways, i.e. do not require the construction of new open channels that may intersect ASS layers or cause a lowering of surrounding groundwater levels.

The application of WSUD into potential ASS regions should give preference to systems that:

- maintain natural stormwater infiltration into the soil
- maintain or increase local groundwater levels
- avoid the need for groundwater to be used as a source of non-potable water.

A detailed management plan will be required:

- for disturbances greater than 1000 tonnes (note: amount may vary between jurisdictions)
- where the proposed works are likely to alter the groundwater table of the area or where the site is close to an environmentally sensitive area (even if less than five tonnes of lime treatment are required).

Environmental management plans (EMP) may be requested by the local government or environmental protection agency to support a drainage proposal, or prepared by a proponent who wishes to demonstrate their general environmental duty effectively.

### 3.7.6 Roadside Stops

In rural areas, litter concentrations occur near facilities such as rest areas, truck-parking bays, and wayside stops. Where kerbs and channels and underground drainage are provided on these facilities, grated pits or grated side entry pits may be used as litter traps.

There is a need to consider water sensitive road design where run-off from the paved area is ‘treated’ prior to discharge to the surrounding environment. For rural roadside stops this may simply include allowing the run-off to sheet flow off the paved area over vegetated/grassed land surrounding the rest area. Broken/slotted kerb or fencing may be installed if delineation is required. Management of litter may need to be addressed. See *AGRD Part 6B* (Austroads 2009c) for further design considerations of roadside rest areas.

The use of recycled water, rainwater tanks and water harvesting may be designed into the toilet facilities provided at a roadside stop.

### 3.7.7 Water Harvesting

Stormwater pollution is a major threat to urban rivers and creeks whereby excess nutrients can lead to the demise of animals, plants and fish living in waterways. Furthermore, heavy rainfall and associated flooding can cause erosion that also threatens plants and animals. This situation is exacerbated by the significant growth of many large cities and the resulting increase in run-off from rainfall.

Stormwater harvesting can assist in managing the adverse effects of storm run-off on waterways by reducing the volume and speed of flow of water and the amount of pollution reaching waterways. Water from the harvesting process may require treatment to remove pollutants before it is used for appropriate purposes.
Urban stormwater harvesting schemes can be defined as the collection, treatment, storage and use of stormwater run-off from urban areas. Figure 3.13 illustrates the harvesting process; however, it should be noted that whilst schemes will have common elements their characteristics will vary.

Storage associated with water harvesting may include wetlands, retarding basins, dams or lakes. Consequently, the effects on the catchment and waterways upstream and downstream of the harvesting scheme are a most important consideration.

**Figure 3.13: Conceptual illustration of the stages of a stormwater harvesting scheme**

Note: Image supplied courtesy of Melbourne Water.

Stormwater proposals must be considered in the context of the entire catchment, recognising the intrinsic value of rainfall in replenishing surface water flows and groundwater. They must also give consideration to the environmental and social/visual values of urban waterways.

The availability and quality of water for harvesting will vary from catchment to catchment depending on a range of issues including:

- catchment characteristics (level of urbanisation, environmental significance of waterways within the catchment)
- receiving environment (bay, estuary, freshwater system)
- existing demands (urban and rural).

The planning and development of stormwater harvesting schemes will therefore require the involvement of specialist designers. It will also involve a knowledge and understanding of the requirements of the environmental protection authority.
Road designers are likely to become involved in the planning and design activities where harvesting schemes are associated with major road projects, as coordination of the road design and road drainage with the harvesting facility is essential.

To better understand stormwater harvesting and to obtain further details regarding the design and establishment of a stormwater harvesting scheme, designers should refer to national and local drainage authority guidelines, including links to other sources, such as National Water Quality Management Strategy, Australian Guidelines for Water Recycling: Stormwater Harvesting and Reuse (Natural Resource Management Ministerial Council et al. 2009) and Guidelines for Stormwater Harvesting (Melbourne Water 2012b).

### 3.7.8 Drainage Shadow

On open, gently sloping terrain, the inclusion of a raised road embankment can lead to the creation of a drainage shadow (Figure 3.14). The disruption of drainage sheet flow (i.e. creation of drainage shadows) can adversely affect the local ecology, leading to significant tree and (potentially) animal habitation loss. The road embankment effectively intercepts drainage that would normally flow across the affected area. Increasingly, large side drains or table drains have been typically used to carry the water down to low points on the land profile to cross under the road in a concentrated fashion via large culverts.

However, by placing smaller culverts at closer spacing with associated short levee banks, rather than accumulating the flow and taking it via table drains to larger (less frequent) culverts, the surface water is better able to regain sheet flow characteristics on the downstream side of the road. As a consequence, the ground flows more closely replicate traditional water flow patterns and better preserve the local vegetation (Figure 3.15). This approach also serves to reduce scour potential.

Additional treatments may be used at culvert outlets to shorten the distance required to re-establish sheet flows. Figure 3.16 illustrates the concept of a sill drain. Discharge is redirected parallel to the road within a section of an open drain before overtopping as sheet flow. The need for such treatments will depend on the significance and proximity of downstream vegetation.

Figure 3.14: Typical drainage
Figure 3.15: Modified drainage

Figure 3.16: Section A of a sill drain

★ Actual depth below culvert outlet invert to be decided on a case to case basis
4. Drainage Considerations

4.1 General Considerations

Any drainage design requires consideration of:

- the appropriate ARI for various parts of the system
- the quantities of water to be managed
- how stormwater run-off can be most efficiently captured
- the environmental consequences of road operation and drainage design options
- the natural water balance of the site and the desire to maintain this balance where possible (i.e. at source infiltration)
- capacity, condition and durability of existing systems where present
- what external constraints affect the design choices
- how the drainage design impacts on other aspects of the project
- the consequences of failure of the system
- occupational health and safety issues in construction, operation and maintenance
- future operational costs and requirements of the drainage system.

Each design will have different attributes that will affect these considerations. For example, the considerations influencing the design of an urban drainage system can be substantially different to those influencing the design of a rural drainage system.

Notwithstanding these differences, the fundamental input for all drainage design is the determination of the peak run-off from a catchment, for an accepted ARI. This requires analysis of the catchment characteristics and the range of rainfall events that can be expected across the catchment. The definitive Australian work on this subject is AR&R Vol. 1 (Pilgrim 2001) which forms the basis of most drainage design manuals developed by road agencies and others. Catchment run-off/peak flow is covered in further detail in Section 6 – Hydrology.

4.1.1 Australian Rainfall and Run-off (AR&R)

AR&R Vol. 1 (Pilgrim 2001) comprises a series of books dealing with various aspects of the design process and the range and interrelationships of the building blocks of drainage design. Depending on the project, some or all of these topics need to be addressed:

- introduction
- design rainfall
- choice of flow estimation methods
- estimation of design peak discharges
- estimation of design hydrographs
- estimation of large and extreme floods for medium and large catchments
- aspects of hydraulic calculations
- urban drainage systems.
4.2 Road User Considerations

For road user safety, rain falling on trafficable lanes should be removed as quickly and directly as possible (to reduce the risk of aquaplaning), primarily through appropriately designed road geometry. Further to this requirement, all surface flows either conveyed to or emanating from the roadside should be confined to gutters and shoulders as applicable. Contour plans of the road surface can be used to estimate the flow paths and identify the potential for aquaplaning. See AGRD Part 5A – Section 4 for further details.

On a typical kerbed roadway, road surface run-off begins to accumulate within the gutter. The depth and width of flow needs to be assessed to verify that the encroachment of the channel flow into the roadway does not pose a safety hazard to road users travelling within the roadway or create a nuisance impact for those general public (off-road users) travelling alongside the roadway (i.e. splashing of pedestrians). Further, the depth and velocity of the flow need to be assessed to ensure that they fall within safe limits and minimise the impacts on all road users.

The location of drainage pits and other drainage structures is also an important consideration in road user safety and level of service. Care should be taken to ensure that:

- pit covers do not pose a hazard to cyclists or pedestrians (i.e. metal covers in tight radius curves creating a slip hazard for cyclists or covers in front of pedestrian ramps). Refer also to Section 2 – Safety in Design
- on-grade inlet spacing is such that encroachment of gutter flow into trafficable lanes is within acceptable limits and that a minimum width of carriageway remains open during severe storms for the particular class of road
- inlets are placed at appropriate, fixed locations to ensure road safety, such as prior to the development of superelevation to prevent the gutter flow sheeting back across the carriageway
- location of infrastructure takes into consideration future maintenance operations, the safety of workers and the road users around them
- inlets are placed to ensure an appropriate flow width is achieved adjacent to kerbs to minimise splashing of pedestrians, maintain access at bus stops, ensure access to pedestrian crossings and allow access to parked vehicles.

The safety of road users in other situations is discussed further in Section 2 – Safety in Design.

Further information on road surface flow is provided in AGRD Part 5A – Section 3.2.

4.3 Design Considerations

4.3.1 Identifying Design Considerations

The construction of new or upgraded road drainage infrastructure may lead to changes in the existing road and external environments. Problems associated with erosion and sedimentation, flooding (changes in peak water levels) and water quality are of concern to road and drainage authorities, adjacent land owners, road users and the local community. The occurrence of these problems, particularly after a project is completed, can be costly to remedy and may lead to reduced amenity.

Development of the most appropriate drainage solution for a project and avoidance or minimisation of adverse impacts requires effective planning and design which may involve a multi-disciplinary team on large or complex projects.
A generic set of design considerations that may apply to projects is summarised in Table 4.1. Designers should use the table as a check list for design considerations within the following categories:

- geometric
- geographic
- environmental
- crossing type
- drainage system risks, including the consequence of failure
- drainage structures
- maintenance
- safety
- staged construction of roads.

It should be noted that identified design considerations may present several options when being addressed. It is possible that upon further consideration or review, some design considerations may no longer be part of a project while others develop into key design controls.

Table 4.1: Summary of design considerations

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Consideration</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geometric considerations</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stream geometry</td>
<td>Stream longitudinal alignment</td>
<td>• Usually only one alignment for all flows, but it is possible to have different alignments for a low and high flow in the same stream.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Preferable to maintain or preserve the existing stream alignment as changes will affect the existing flow parameters (velocity, depth of flow and energy).</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• It is possible to alter the alignment of existing streams to improve the hydraulic performance of the road-stream crossing.</td>
</tr>
<tr>
<td></td>
<td>Stream gradient</td>
<td>• Has a significant influence on flow velocity which has a significant effect on sediment transport and scour potential.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Changes to stream gradient will also affect flow parameters.</td>
</tr>
<tr>
<td></td>
<td>Channel shape</td>
<td>• Tends to dictate the size and configuration of drainage structures.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Altering the channel shape to accommodate a drainage structure will affect flow parameters and could increase the risk of erosion.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• It is preferable to maintain or preserve the existing channel shape and culvert structures should be designed to &quot;fit&quot; the shape of the stream.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Some channels may not contain all of the design storm run-off and overtopping of the banks will occur. Multiple culvert installations for the one catchment will be required and in this instance, specialist advice/design will be required.</td>
</tr>
<tr>
<td>Road geometry</td>
<td>Horizontal alignment</td>
<td>• Minimising the skew angle between the road alignment and drainage structure (or eliminating it altogether) is most desirable as it reduces costs and construction difficulty.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Usually highly recommended to preserve stream alignment and hence high skew angles may be unavoidable.</td>
</tr>
<tr>
<td></td>
<td>Vertical alignment</td>
<td>• An initial vertical alignment design would be used to undertake the initial drainage design of various structures. It may be necessary to adjust the vertical alignment of the road in order to achieve the most efficient and effective drainage designs (considering allowable headwater levels, afflux and minimum cover requirements for structures) provided that appropriate values for road design parameters can be achieved.</td>
</tr>
<tr>
<td>Aspect</td>
<td>Consideration</td>
<td>Comment</td>
</tr>
<tr>
<td>--------</td>
<td>------------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td></td>
<td></td>
<td><strong>Stormwater run-off from road</strong></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Minimum and maximum grade requirements for longitudinal drainage channels (such as table drains) should be achieved so that effective flow is achieved but higher erosive flow velocities are avoided.</td>
</tr>
<tr>
<td>Stormwater run-off from road</td>
<td>This aspect is critical as water flow (and depth) on the road surface relates to aquaplaning. Any identified problems should be solved and mitigated through amended geometric road design (combination of horizontal, vertical, cross-section elements). A drainage solution to aquaplaning should be only considered as a ‘last resort’ option, for which specialist advice is highly recommended.</td>
<td></td>
</tr>
<tr>
<td>Stormwater across road</td>
<td>Where there is a possibility of stormwater crossing over the road (whether intentional or unintentional), adequate stopping sight distance must be provided.</td>
<td></td>
</tr>
<tr>
<td>Geographic</td>
<td><strong>Flood immunity</strong></td>
<td><strong>All regions</strong></td>
</tr>
<tr>
<td>Urban regions</td>
<td>Intense level of development and afflux which is usually of more concern in urban areas</td>
<td>Provision for higher peak flows arising from uncontrolled upstream development (many local authorities now require flow increases to be mitigated). Assess requirements of any catchment management plan or stormwater management plan prepared for the watercourse. Consider need for pollution control measures. Interaction of road drainage provisions with existing services. Minimisation of ground disturbance during construction as urban environments often have limited space for large control measures such as sediment basins. Consideration and control of afflux effects. There is often a requirement that negligible afflux increases be generated upstream/downstream of the proposed drainage structure. With respect to possible changed water levels, it is important that each case is assessed fully in keeping with a risk management approach.</td>
</tr>
<tr>
<td>Coastal regions</td>
<td>Tidal inundation, a corrosive environment and sandy soils (i.e. soils with little cohesion). Coastal environments are also often highly sensitive to pollution.</td>
<td>Legal requirements with respect to the protection of marine environments (e.g. protection of fish habitats and marine plants) must be met. Natural flow systems (e.g. tidal exchange) should be properly assessed and not be compromised. Corrosion resistant materials should be used. Potential acid sulphate soils (typically below five metres AHD) should be identified and managed appropriately. Storm tide influence upon peak water levels and greenhouse related sea level rise should be considered. Designs should allow for the presence of highly erodible or mobile materials such as sand. Consideration should be given to directing drainage to natural channels or swales rather than to hard structures. Bi-directional flows (due to tidal effects) must be identified and appropriately considered.</td>
</tr>
<tr>
<td>Aspect</td>
<td>Consideration</td>
<td>Comment</td>
</tr>
<tr>
<td>--------------------</td>
<td>-----------------------------------</td>
<td>--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Flat terrain</td>
<td>Nature and impact of water flow</td>
<td>• Flow velocities in flat areas are usually low so larger structures are needed to convey the flow.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Flow may be widespread and/or shallow so minor obstructions to the flow may divert flow significantly. These minor obstructions include levees and other floodplain works. Even the road itself may cause major diversions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• It is often difficult to determine the catchment areas accurately because of minimal relief in terrain and the presence of minor obstructions.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Poorly defined flow paths also mean that it is sometimes difficult to place culverts in the most suitable locations.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• In flat terrain the impacts of the road on flood levels may extend for significant distances upstream of the road. Where afflux is a concern, this impact may often be critical.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• There is usually an increased risk of erosion at culvert outlets because flow will be concentrated by drainage structures, particularly where there are poorly defined flow paths and/or most flow occurs across the floodplain.</td>
</tr>
<tr>
<td>Mountainous terrain</td>
<td>Steep terrain</td>
<td>• Control of velocities in roadside drains and culvert outlets.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Collect and discharge of water from the upslope side of the road to the downslope side.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Prevent erosion at outlets onto steep areas.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Consider need for small scale drop structures, weirs or drop manholes.</td>
</tr>
<tr>
<td>Environmental</td>
<td>Scour, erosion and sediment</td>
<td>• The risk of scour/erosion and sediment movement caused by the concentration of flows that typically occurs with drainage structures is of particular concern.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Causal factors including changes in flood flow patterns and changes in peak water levels should also be checked.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• In some instances, a new road embankment could lead to long-term ponding of water which in turn could have adverse environmental impacts.</td>
</tr>
<tr>
<td></td>
<td>Fauna habitat and passage</td>
<td>• Where there is a need, provide for fauna passage and the maintenance (or improvement) of water quality.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Streams are usually riparian corridors and all drainage works on roads should minimise any adverse impact on these important corridors. Therefore they can provide a corridor for fauna movement as well as providing a habitat for terrestrial and aquatic fauna.</td>
</tr>
<tr>
<td>Broader context</td>
<td></td>
<td>• Careful review of any relevant environmental assessment documentation, including any recommended management strategies, needs to be undertaken as some of these strategies may become design requirements or criteria.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• The recommended management strategies are generally based on the requirements of relevant legislation, policy, codes, guidelines and current best practice within the industry.</td>
</tr>
<tr>
<td>Other</td>
<td>Safety</td>
<td>• The safety of the public, road users and maintenance personnel must be considered (see Section 2 – Safety in Design).</td>
</tr>
</tbody>
</table>
### Staged construction

<table>
<thead>
<tr>
<th>Aspect</th>
<th>Consideration</th>
<th>Comment</th>
</tr>
</thead>
</table>
| Staged construction  | Whole-of-life approach | • Planning and design should take proper account of both expected and potential changes that will or may occur as traffic grows and the surrounding land use develops or changes.  
• This aspect is most important for projects that are planned to be built in stages over a period of time. Making allowance in current designs for these changes ensures that future enhancements can be accommodated in a cost effective, efficient and safe manner.  
• Other future benefits include:  
  - reduced disruption to road users and adjacent property owners  
  - early/coincident resumption of properties  
  - reduced environmental impacts.                                      |

Source: Adapted from DTMR (2010b).

### 4.4 Selection of Recurrence Interval and Flood Immunity

The components of a drainage system are designed to handle a specific design discharge which is linked to the ARI. The ARI is selected after considering the potential effects of the ARI being exceeded. The probability of exceeding the design discharge is determined by a statistical approach rather than being a deterministic statement. Recurrence intervals are a long-term average and intervals between specific discharge events may be considerably shorter or longer than the ARI for that event.

Design ARIs should take into consideration a number of factors including:
- existing capacities of structures, both upstream and downstream and the ARI to which they were designed
- other proposed developments
- road usage and alternative routes
- consequences of the ARI being exceeded.

### 4.5 Defining Immunity

The design criteria for a particular project may be set either by the client or drainage authority and may be based on any of the following conditions:
- flood immunity
- trafficability
- time of submergence/closure.

#### 4.5.1 Flood Immunity

Flood immunity is defined as the ARI of a flood that just reaches the height of the upstream shoulder of the road, or where the road is kerbed, the top of the inlet pit. In other words, the road surface remains dry/is immune to a flood corresponding to the selected ARI. Furthermore, freeboard (see Section 4.7 – Freeboard) may be required to ‘lower’ the water level further to keep the pavement dry in the face of uncertainties in flow estimates, hydraulic calculation, etc.

For a particular road (considering road importance, traffic, alternate routes etc.) this immunity level can be altered to allow water onto the road surface. The amount of acceptable encroachment must be specified by the client or road agency.
4.5.2 Trafficability

In some instances it is desirable to allow traffic to continue to use the road while floodwater crosses the road surface. The design criteria therefore may be specified in terms of the ARI of the flood at the limit of trafficability. This limit is based on a combination of depth and velocity of flow over the road or floodway and is defined as occurring when the total head (static plus velocity) at any point across the carriageway is equal to 300 mm. The road should be closed if the flow is greater than this limit. An example of using trafficability as part of the immunity requirement for a particular location – a crossing is required to be ARI 20 year immune and ARI 50 year trafficable. This means that the road surface will be free of water up to the ARI 20 year event, but still trafficable (having less than 300 mm total head over the road surface) in an ARI 50 year event.

4.5.3 Time of Submergence/Closure

All drainage structures including bridges, culverts and floodways are designed for a particular average recurrence interval flood (e.g. 10, 50, 100 years). Larger catchments may require bridge and floodway sections to cater for a design flood of such magnitude with significant cost implications. For economic reasons the design of the waterway area to carry water beneath the road may have to be based on a lower standard flood which will lead to more overtopping of the road during larger floods.

Strategic transport routes are likely to be provided with a high level of flood immunity with road submergence or closure being a rare event. However, where a low level of flood immunity is considered appropriate, the cost of the drainage structures will be less but the frequent and possibly lengthy road submergence or closure will cause costly delays to traffic, damage to the road surface and require consideration of expensive protective measures for the road embankments.

Consideration of the time of submergence or closure is therefore an important aspect of drainage design to supplement the flood immunity assessments. The time of closure is a measure of the disruption to traffic and in some ways is a better measure of the performance of the road. In some cases, low flood immunity may be acceptable if the times of closure are low and the expected disruption is relatively minor. This measure can be expressed in terms of submergence or closure as discussed below:

**Time of submergence (ToS)**

This is a measure of the expected time that the road is submerged in any flood but especially in a major flood such as the ARI 50 year event. Submergence is defined as the point where the road is just overtopped, even by very shallow water.

**Average annual time of submergence (AAToS)**

This is a measure of the expected average time per year of submergence of the road caused by flooding. It is expressed as time per year.

**Time of closure (ToC)**

This is a measure of the expected time of closure of a road (road not trafficable) in any flood but especially a major flood such as an ARI 50 year event.

**Average annual time of closure (AAToC)**

This is a measure of the expected time of closure of the road due to flooding, expressed as time per year. The procedure included in *AGRD Part 5B – Section 4.4* is also relevant for bridges and culverts.
The average annual times of submergence and closure depend on the frequency of submergence/closure as well as the duration of each occurrence. For example, two streams may have a similar average annual time of submergence, but quite different flood immunity, if one is closed frequently for short durations, while the other is closed more rarely for longer times. The impacts of these different patterns can be analysed to determine the most appropriate design for each particular crossing.

The time of submergence/closure is related to catchment area and response times as well as the flood immunity. These times are calculated either from design flood events or from stream flow data as described later in the Guide.

More details on time of submergence and closure are provided in AGRD Part 5B – Section 4.4 where the methods of calculation are included.

### 4.6 Selection of ARI

It is important to note that the selection of an ARI for each location and/or element within a project is the ARI in isolation from adjacent catchments and/or structures. The events are independent. It is quite a complex and difficult process to specify and determine an ARI for a section of road comprising several catchments and structures. This aspect is highlighted in Section 4.5 – Defining Immunity.

If the project is sufficiently large and complex, a specific risk analysis assessment may be carried out (Alderson 2006) in order to select an appropriate ARI for various elements, but most authorities have a range of default ARI that may be adopted in the absence of other criteria. A guide to typical standards that may be applied to preliminary design for various drainage applications for structures and other public spaces is provided in Table 4.2.

#### Table 4.2: Suggested minimum flood immunity standards for adjacent land use

<table>
<thead>
<tr>
<th>Priority</th>
<th>Situation</th>
<th>ARI(^{(1)}) (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Floor levels of hospitals and civil defence headquarters.</td>
<td>500</td>
</tr>
<tr>
<td></td>
<td>Floor levels of police, ambulance, fire stations, water and waste centres, electric and gas supply stations, convalescent homes and buildings designated as emergency housing used during extreme flooding events.</td>
<td>200</td>
</tr>
<tr>
<td>B</td>
<td>Floor levels of residences, essential food, pharmaceutical, retail stores, department stores, centres employing a large labour force, community administration and education centre, centres of rare artefacts, venues for entertainment, dining, popular indoor sport.</td>
<td>100</td>
</tr>
<tr>
<td></td>
<td>Floor levels of factories and outlets supplying non-essential items, premises of businesses and institutions employing a small labour force, premises of sport or community activities infrequently used. Town centres and intense industrial areas including central business district.</td>
<td>50</td>
</tr>
<tr>
<td>C(^{(1)})</td>
<td>Grounds of all units belonging to priority A, outdoor areas where rare artefacts are displayed or stored.</td>
<td>5–10</td>
</tr>
<tr>
<td></td>
<td>Grounds of all units belonging to priority B.</td>
<td>3–5</td>
</tr>
<tr>
<td></td>
<td>Other open space areas including general parks and outdoor sport and recreation areas(^{(2)}.)</td>
<td>1–3</td>
</tr>
</tbody>
</table>

1. An ARI of 10 years may be applicable for flat or low lying areas.
2. It should be noted that some open space areas may be designed as part of a stormwater management system to attenuate peak discharges and as such may be inundated, or partly inundated on a more frequent basis.


Provision of flood immunity is accomplished by ensuring that drainage systems are designed to maintain flood levels below predetermined levels for facilities in these adjacent areas (note that drainage systems may include adjacent infrastructure that is utilised to convey run-off surcharge such as the road carriageway). The appropriate flood levels vary depending upon the facility in question and the general terrain.
The standards applicable to different drainage components of the road are shown in Table 4.3. The actual standards used should comply with the requirements of the relevant jurisdiction as requirements vary throughout Australia and New Zealand.

Table 4.3: Guide to selection of average recurrence intervals for flood immunity

<table>
<thead>
<tr>
<th>Element</th>
<th>Austroads road classification</th>
<th>Suggested ARI(3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cross drainage (culverts &amp; bridges)</td>
<td>Controlled Access Highways Includes: Motorways &amp; Freeways (National/State/Territory)</td>
<td>100 years</td>
</tr>
<tr>
<td></td>
<td>Arterial Roads Classes 1 &amp; 2(1) Includes: National/State/Territory Highways, Urban Arterial Roads</td>
<td>50–100 years</td>
</tr>
<tr>
<td></td>
<td>Arterial Road Class 3(1) Includes: State/Territory main roads</td>
<td>50 years</td>
</tr>
<tr>
<td></td>
<td>Local Roads Classes 4 &amp; 5(1)</td>
<td>10–20 years</td>
</tr>
<tr>
<td></td>
<td>Urban Collector/Distributor Roads</td>
<td>10–50 years</td>
</tr>
<tr>
<td></td>
<td>Urban Local Roads</td>
<td>10 years</td>
</tr>
<tr>
<td>Diversion channels</td>
<td>All roads</td>
<td>Adopt the ARI for cross drainage</td>
</tr>
<tr>
<td>Cross drainage (floodways)</td>
<td>Arterial Road Class 3(1)</td>
<td>20 years</td>
</tr>
<tr>
<td></td>
<td>Local Roads Classes 4 &amp; 5(1)</td>
<td>5–10 years</td>
</tr>
<tr>
<td></td>
<td>Urban Local Roads</td>
<td>5–10 years</td>
</tr>
<tr>
<td>Road surface (network drainage</td>
<td>All roads other than Local Roads</td>
<td>10 years(4)</td>
</tr>
<tr>
<td>including kerb and channel with inlet</td>
<td></td>
<td></td>
</tr>
<tr>
<td>pit &amp; pipe systems, bridge decks</td>
<td>Local Roads</td>
<td>5 years(3)</td>
</tr>
<tr>
<td>Trapped flows (roads where there is</td>
<td>All roads</td>
<td>50 years</td>
</tr>
<tr>
<td>no escape path for water including at</td>
<td></td>
<td></td>
</tr>
<tr>
<td>a sag in cut)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Longitudinal open drainage (table</td>
<td>All roads</td>
<td>10 years (unless cross</td>
</tr>
<tr>
<td>drains, diversion drains, catch drains</td>
<td></td>
<td>drainage ARI is less,</td>
</tr>
<tr>
<td>and banks etc.)</td>
<td></td>
<td>then adopt the lesser</td>
</tr>
<tr>
<td></td>
<td></td>
<td>value)</td>
</tr>
</tbody>
</table>

2 Road and/or drainage authorities can change suggested ARIs based on link requirements, road importance and inundation risk, for example:
   a. For a local road connecting communities where no alternate route is readily available, an increase in cross drainage ARI from 20 to 50 years could be specified.
   b. For an important motorway where operation during flood events is a key requirement, road surface drainage could be specified as ARI 20 years with clear traffic lanes and/or ARI 50 years with no more than 1 m encroachment into outside traffic lanes.
   c. For a floodway on a local road where alternate routes are readily available, the ARI specified could be reduced to two years.
3 See the requirements of the relevant Municipality for roads under their control.
4 In South Australia urban areas, five year ARI is used.
Source: Adapted from VicRoads (2003).

4.6.1 Example Application of Flood Immunity Considerations

It should be noted that in particular circumstances the ARI of a specific project or structure needs to be considered in the context of flood immunity for a specific route. For example, RTA NSW (1999) notes that a ‘route approach’ should be taken in relation to flood immunity rather than specific elements of the route or of individual structures being considered in isolation:
One factor that should be taken into account in determining the appropriate design level for individual drainage structures is the required level of serviceability for the entire route that the structure is situated on.

For example, the Pacific Highway between Sydney and Brisbane passes through some 15 separate major drainage catchments. Each of these basins can flood more or less independently of the others. Statistics show that, if the floods were fully independent and every structure on the highway was designed for the 100 year event, the route would be closed, on average, once every seven years. Looking at it the other way, if it were determined that the route should be closed only once every 100 years on average, then each structure should be designed to take a 1500 year flood.

This example is simplistic as it does not take into account that small streams will be flooded by a different storm mechanism (thunderstorms) than large rivers (prolonged storm depressions). The issue of duration of closure is also not considered. Nevertheless, the example demonstrates that route serviceability is not the same as the design level of individual structures and therefore special consideration is required.

In the past most agencies or authorities have determined design capacities for the size of drainage systems by choosing, say, 100 years ARI for bridges, 20 to 50 years ARI for culverts and 10 years ARI for pavement run-off. Consequently, the route would be affected more frequently by surface ponding or flooding of the smaller crossings than by the larger streams. In some situations this may be acceptable since the smaller events would typically close the route for a shorter duration.

Route serviceability targets should be considered for use in determining design flood standards for the smaller structures.

4.7 Freeboard

Freeboard is an allowance made between the design water level and some identifiable/specifc point, such as a bridge, top of an embankment, high bank of channel or adjacent buildings. It is an allowance applied to the height of the design flow to cater for:

- the uncertainty of modelling the design flood event and predicting the flood level
- floods of greater magnitude
- floods of the design discharge that may be higher in level due to blockage, debris or other effects that may be difficult to quantify
- wave action on the water flow.

The assessment of the amount of freeboard required needs to be based on the risks of overtopping, characteristics of the catchment and storm flows. Allowances can be adjusted for significant roads, surrounding property development or other criteria. The level of importance of a road may affect the freeboard used to ensure trafficability.

Freeboard is not required in all situations and this may be specified as part of the design. For example, allowance of freeboard for an open channel through parkland may not be required as any overtopping of the channel will simply flood the adjacent parkland, not causing any damage or inconvenience. The following sections discuss the application of freeboard across a variety of drainage scenarios. Figure 4.1 provides some examples of the use of freeboard for various situations.

4.7.1 Kerbed Drainage

Kerbed drainage typically collects water from the pavement surface for typically the design minor event and is discharged via a piped drainage network or open channel.
The constraints imposed by adjacent development on urban drainage networks are significant. There can be considerable financial liability borne on a road agency should residential or commercial property become subjected to flood damage as a result of road drainage infrastructure having insufficient capacity with inadequate, or little to no contingency (i.e. overland escape paths) in place. Freeboard of 0.3 m for the design major event should therefore be taken to the floor level of all potentially susceptible properties as shown in Figure 4.1(A).

The ARI used for this situation should be confirmed with the relevant catchment management authority and is generally the 100 year event.

Kerbed drainage is discussed in detail in AGRD Part 5A – Section 5.

4.7.2 Underground Piped Networks

The underground stormwater networks used to convey road surface run-off to suitable drainage outfalls are generally designed to accommodate the design minor event. Consequently there should be no surcharge of captured run-off from within the network. Freeboard of 0.15 m in this case should be taken to the top of pit, which in the majority of cases (urban road drainage networks) is also the road surface level adjacent to the kerb invert as shown in Figure 4.1(B).

Underground piped networks are discussed in detail in AGRD Part 5A – Section 6.

4.7.3 Drainage Basins

Drainage basins are typically used as intermediate (detention) or end of line (retention) stormwater disposal points. Their consequences of failure are dependent upon the method of their construction as follows:

- Where construction utilises the road formation as part of the containment, freeboard should be taken to the subgrade surface as shown in Figure 4.1(C). ARI as per open drains i.e. 5–10 years.
- Where construction utilises a levee bank, freeboard should be taken to the spillway invert as shown in Figure 4.1(D). Freeboard in this case is the distance between the overflow level for the design event and the major event, e.g. ARI 20–100 years.

Drainage basins are discussed in detail in AGRD Part 5A – Section 7.

4.7.4 Vertical Controls – Flood and Groundwater Levels

On most projects, minimum clearances above flood levels and water tables should be defined by the client either in the design brief or in the design specifications. The following issues should be considered:

- Where a road is designed not to be overtopped during a set ARI flood event, it is desirable to provide freeboard of 0.3 m between the design upstream floodwater surface and the pavement subgrade or upstream road shoulder edge as shown in Figure 4.1(E). The pavement design should make allowance for higher water levels and the likely duration of inundation should the subgrade not be used as a control for freeboard.
- Freeboard of 0.3 m should be provided between the maximum annual groundwater level to the subgrade surface as shown in Figure 4.1(F). In flat terrain, the grade line should generally be located so as to provide a clearance of 0.5 m to 1.0 m between the water table and the pavement boxing at its lowest point. Further geotechnical and pavement design advice should be obtained to determine the required clearance for specific projects. Subsurface drainage is discussed in detail in AGRD Part 5A – Section 8.

See also AGRD Part 3 (Austroads 2010c) for further information on flood levels and water tables.
4.7.5 Open Channel Design

Open channels can be used to either manage the minor or major event ARIs (or both) therefore the size of the open channel can vary greatly depending on its function.

Freeboard allows for inaccuracies in data used in calculation and possible surcharge due to silt/debris build up and/or grass growth in the channel because of delayed maintenance of the channel. In steep terrain, it may be applicable to apply freeboard equal to the design flow depth to compensate for the large variations in flow caused by waves, splashing and surging.

The design water level in a table drain or open median drain should desirably be set below the subgrade level of the road pavement, particularly in flat country or in high water conditions. Where the material between the table drain and the road pavement is impermeable, the table drain may flow up to a level which provides a 0.15 m freeboard against overtopping, (i.e. below the level of the outer shoulder), provided that this does not flood subsurface drain outlets.

The design water level in catch drains should be at least 0.15 m below the top of the bank.

Freeboard as applicable to open channels should be as follows:

- For table drains, freeboard should be as shown in Figure 4.1(C) for drainage basins utilising the road formation for containment. That is, freeboard should be taken to the subgrade surface.
- For median drains between carriageways, freeboard should be taken to the lowest adjacent subgrade surface level as shown in Figure 4.1(G).
- For open channels away from the road formation, freeboard should be taken to the top of the drain as shown in Figure 4.1(H).
- For catch drains, freeboard should be taken to the top of the drain (i.e. top of cut batter) as shown in Figure 4.1(I).

For large open channels, freeboard should be the greater of the following calculations:

- 0.30 m – where flooding of adjacent land and buildings does not represent a risk, the 0.30 m requirement can be reduced to 0.15 m
- 20% of the flow depth
- velocity head of the flow.

For temporary open channels, depending on an assessment of risk, it may be applicable to adopt zero freeboard.

For details regarding the design of open drains see AGRD Part 5B – Section 2.
Figure 4.1: Examples of application of freeboard for a variety of situations

Note: $x$ represents the freeboard to be determined for each situation.
4.7.6 Culvert Design

Culverts and bridges are generally used to manage flows from a major ARI event across a road formation.

Freeboard as applicable to culverts and bridges should be as follows:

- For culverts and bridges with a cross-section greater than 6 m², freeboard of 0.3 m should be provided as shown in Figure 4.1(J) and Figure 4.1(K) is required to prevent blockages of, or damage to, the culvert or bridge by debris.

- Culverts with a cross-section less than 6 m² are usually designed with submerged inlets as shown in Figure 4.1(L) that is, without freeboard, unless it is known that debris will be carried down the catchment. The control point may be the lower edge of shoulder or the subgrade level of the road pavement. The designer should consider the damaging effects of water infiltrating the road pavement when determining or setting the maximum allowable headwater including freeboard for the design of submerged culvert inlets. The use of impermeable shoulder, verge or pavement materials may allow higher water levels and consequently reduced freeboard heights.

For details regarding the design of culverts see AGRD Part 5B – Section 3.

4.8 Other Considerations

4.8.1 Drainage Construction Materials

Designers should carefully consider proposed drainage materials against the site-specific requirements. For example, Appendix B contains a list of materials commonly used in culvert construction and the advantages and disadvantages of each type based on the site-specific conditions. In addition, road agencies may have specific requirements or exclusions around the use of particular materials.

4.8.2 Recycled Materials

Until recently roadwork specifications have generally required naturally occurring materials for elements such as pavement layers, pipe bedding and backfill for pipes and drainage structures. However, specifications have been developed by organisations providing a basis for the increased use of recycled materials in road construction (e.g. Savage 2010).

Recycled materials can be produced from a variety of sources that include crushed concrete, bricks and pavers, glass products, plastics and blast furnace by-products.

Typical applications for the use of recycled material include:

- road base material suitable for a range of traffic conditions
- select fill for improving subgrade performance and also for raising site levels
- bedding material suitable for use as a base layer for pavers
- drainage medium for backfilling drainage structures.

Road agencies and drainage authorities will usually have a policy and specifications regarding the use of recycled material and designers are advised to refer to the relevant documents.

4.8.3 Road Assets

Water may threaten the integrity of the road asset through inundation or by the movement of water causing scour and erosion.
Structural properties of materials used in road and pavement construction will, to varying degrees, depend on their moisture content Austroads (2001) and Austroads Guide to Pavement Technology Part 2 (AGPT Part 2) (Austroads 2012). These properties will be determined by testing prior to design and construction and results incorporated in the final design. Achievement of design performance will require the moisture content to be maintained within the design range. Austroads (2001) and Alderson (2006) deal with the susceptibility of road pavements to moisture. Moisture may enter from beneath through infiltration into the shoulders and pavement edges or through porosity or defects in the road surfacing. These factors need to be considered in the drainage design and are particularly relevant in high rainfall areas.

Depending on the topography, extensive measures may be required to prevent groundwater intruding into the formation and possibly the pavement. This may include provision of cut-off drains to intercept moisture and subsoil drains beneath the formation.

Silty-sand subgrades may incur significant loss of strength if allowed to ‘wet-up’ following construction. Similarly, moisture movement into expansive clay subgrades may lead to substantial loss of pavement shape.

4.8.4 Groundwater

The performance of pavements can be affected by the presence of groundwater which can change the strength or stiffness of the pavement materials. Sometimes lowering the groundwater level can also affect the pavement. Specialist geotechnical advice should be sought when groundwater is encountered.

The presence and quality of groundwater has an influence on the:

- selection and design of subsurface drainage systems to intercept groundwater to lower the water table and to prevent the subgrade and road pavement from wetting-up both during and after construction
- design of special systems to treat groundwater that contains a higher than recommended amount of dissolved minerals and salts, prior to discharge into the surface drainage system and watercourse
- rate and amount of consolidation in locations where stress on the ground is increased, particularly where embankments are to be constructed over soft and/or saturated soils
- design of stable batter slopes.

Site information gathered on groundwater should indicate the location of high water tables, springs and/or aquifers that may influence the stability of cuts and fills or permit the ingress of water into the pavement. In many situations the design of stable slopes and embankments will require complementary subsurface drainage systems.

In high groundwater situations, sealed pipes joints may result in pipe/culvert flotation. An alternative is to utilise a geotextile joint strap and use the pipe/culvert as a subsoil pipe. With this detailing, groundwater lowering can be expected and enhanced pavement protection can be achieved.

The Guide to Road Design Part 7: Geotechnical Investigation and Design (Austroads 2008) provides those engaged in road design activities with a basic understanding and appreciation of the importance of geotechnical investigations and how road design outcomes and other design activities are influenced by site conditions, associated ground response, geological hazards and locally available materials. It also includes information on groundwater including its measurement, interception of groundwater to protect roads and treatment of the water.

4.8.5 Self-cleaning Sections

Self-cleaning sections, e.g. culverts and channels require a reasonably regular flow of a specific velocity/energy that will pick up and transport any silt or debris within the section to a specific location beyond it.
The required minimum velocity/energy for a self-cleaning flow through the section must be determined based on the anticipated sediment and/or debris (type/size/weight) that may accumulate in the section. This flow must be generated by a design storm with a suitable average recurrence interval (ARI) such as ARI one, two or five years depending on how often the channel should be cleaned. Intervals of one or two years are preferred while intervals greater than five years are not recommended.

The location that any silt or debris can be transported to (and deposited) must also be considered as it must:

- be accessible to allow maintenance/clean out
- must not cause any adverse effects to the environment (e.g. water quality and fish passage)
- must not adversely affect any future flows (e.g. cause ponding/increase tail water levels).

Whilst the inclusion/presence of a self-cleaning section does not entirely remove the requirement for regular/routine maintenance it can reduce the frequency that these operations are required.

### 4.9 Extreme Events

An extreme event can be defined as an infrequent event at the high or low end of a range of values of a particular climate or weather variable or the occurrence of earthquakes. An extreme event is either, notable, rare, unique, profound, or otherwise significant in terms of its impacts, effects, or outcomes in the context of the area in which they occur, e.g. a 250 mm rainfall event in Hobart is likely to be considered extreme but not necessarily so in Darwin during the monsoon season.

An extreme event is not simply ‘something big and rare and different’. These events demand some type of temporal and spatial boundaries, and ‘extremeness’ reflects an event's potential to cause change. ‘Extremeness’ comes from the human perception of consequences, which in turn reflects the character of the affected system.

Extreme weather events can cause excessive rainfall which exceeds the design criteria of the infrastructure. This increases the risk of damage to either the road asset or adjacent environment and may also increase the risk of the loss of life to both road users and the general public. An earthquake can cause damage to road drainage infrastructure in a way that adversely affects the performance of that infrastructure.

This Guide does not address the management of an extreme event, however extreme events should be considered in the design of drainage infrastructure to protect the road asset and/or surrounding environment. Where risks and consequences are unacceptable, consider appropriate mitigating measures.

#### 4.9.1 Extreme Event Data Collection and Site Assessment

An important part of the data collection and site assessment work for drainage infrastructure is the collection and analysis of data following naturally occurring events such as seasonal storms and extreme events such as floods, abnormally high tides and storm surges that can accompany cyclones.

These events occur from time to time and are an excellent opportunity to gather data on the performance of the road network. The data collected as part of this program needs to be archived in an appropriate database to be available as a historical record for planning and design. Routine inspections following seasonal storm events provide opportunities to assess drainage performance and document maintenance requirements.

When a surge or flood event has occurred, it is necessary to visit that region as soon as possible after or even during the event if safe access is possible. While there are other requirements for staff to attend to during after extreme events, collected data will provide lasting benefits for many years after the flood has receded.
During site visits, observations and measurements need to be documented to record matters such as:

- flood levels
- inundated areas
- water flow patterns
- scour and erosion behaviour
- debris accumulation and debris levels
- floodway performance
- culvert embankment performance under prolonged headwater
- aerial photography
- fauna assistance measures.

Observed features (e.g. flood and debris marks) should be photographed or videoed and marked for later survey and documentation. Road agency staff, land owners and residents are sources of anecdotal information that could be useful in confirming or calibrating measured information.

4.9.2 Planning and Design

While the planning and design of road drainage systems is based on a determined average recurrence interval or set of average recurrence intervals, it is also a requirement to review designs for possible adverse outcomes that may occur during an extreme rainfall event.

To illustrate this, most arterial roads are designed to an ARI 50–100 year standard. However, should an ARI 100 year event or larger occur, culvert velocities may become unacceptably high causing significant environmental harm. Afflux may increase above the acceptable ARI 50 year limit causing excessive flooding and the road may overtop threatening the integrity of the road embankment, safety of road users, and so on.

The extent of the extreme events to be analysed depends on particular circumstances, so the requirements cannot be defined exactly. Furthermore, while the risk of occurrence of these extreme events is low, the impacts of an extreme event may be severe and should be assessed.

If the adverse outcomes/risks of an extreme rainfall event are deemed to be unacceptable, the design criteria may need to be altered and the design recalculated or appropriate mitigating measures developed and included into the project.

It is important to note that any outcomes (adverse or otherwise) resulting from an extreme rainfall event could occur within both the road and external environments therefore identification of possible outcomes should not be limited to the road reserve and/or chainage limits of the project.

The following sections outline some situations where the design of a project should be assessed for adverse outcomes and risks that may occur during an extreme rainfall event. However, other situations may also exist where assessment should be undertaken, therefore careful engineering consideration and judgement should be exercised. Assistance in identifying or confirming situations requiring assessment and at what level (ARI) assessment should be undertaken can be provided by the drainage authority or drainage expert.

4.9.3 Impacts of Extreme Events on Erodible Soil Environments

If an extreme rainfall event occurs, the maximum allowable velocity for a given structure/device will most likely be exceeded which in turn could result in excessive scour, erosion, environmental harm or even failure or collapse of the structure itself. It is therefore important that these situations are identified and assessed. See AGRD Part 5B – Section 2.7 for information on maximum velocities.
If this situation is considered applicable on a project, specialist advice needs to be sought from the drainage authority or drainage expert as analysis methods are beyond the scope of this Guide.

4.9.4 Excessive Flooding

Larger floods may need to be considered in locations where the impacts of the road on flood levels (based on a normal design ARI) are/will be significant/very severe. These impacts will most likely be worse in a large flood/extreme rainfall event. This issue is particularly important where the road embankment is relatively high and the flood immunity provided by the high embankment is much greater than the usually adopted standard of ARI 50 years. In this case, while larger floods may not overtop the road, a higher peak water level will build up on the upstream side of the road causing excessive flooding and in some cases may cause the overtopping of the catchment boundary, directing or diverting flow to an area not able to handle the increased flow.

The above issues may be further aggravated by blockage of the drainage structure(s) by debris which may lead to a greater risk to road or drainage infrastructure and surrounding area, if the flow cannot overtop the road. However, depending on the blockage and material, high flows are likely to blow the debris build-up out of the culverts and restore normal capacity.

Therefore, where flood impacts would be significant/very severe, it is necessary. This requirement can be specified in design/contract documentation.

4.9.5 Earthquake-prone Areas

At sites that are susceptible to earthquakes, longitudinal connectivity, robustness of the drain foundation and anchorage by the inlet and outlet structures should be considered to mitigate the effects associated with:

- foundation liquefaction
- road formation lateral spreading
- batter instability.

4.10 Waterway Structures

4.10.1 Factors Affecting Selection of Waterway Structure

A key decision in the provision of a road crossing of a waterway is the type of structure to be provided, usually a bridge, culvert or floodway. The factors to be considered in determining the type and size of a waterway structure include:

- the level of serviceability to be provided to traffic and the magnitude of the design flood
- the magnitude of the total waterway design flood where a lower level of serviceability is to be provided to traffic
- road alignment and geometric standards
- the topography at the site and geometric characteristics of the system
- the hydraulic aspects of the stream such as stage/discharge and velocity/discharge relationships, and flow patterns at the site
- limits on backwater imposed by river authorities or resulting from development upstream
- requirements for watercraft navigation
- soil conditions and potential for scour at the site
the incidence and nature of debris carried by the stream during floods
environmental considerations.

Culverts and floodways are discussed in greater detail in AGRD Part 5B – Section 3 and Section 4 respectively.

For the majority of designs it will be obvious as to whether a bridge or culvert is required at a location. The decision will generally be made on the basis of serviceability, the existing bank height of the watercourse, potential for debris to affect the waterway, whether the stream bed is active or not and, in some cases, to allow for the passage of fauna. In the case of an active stream, building a bridge may be easier and will have less impact on the environment than a solution with culverts.

Occasionally however, waterway openings can be provided by either a culvert or bridge. Estimates of costs and risks associated with each will indicate which structure alternative should be selected on the basis of economics. Other considerations which may influence selection of structure type are given in Table 4.4.

Some road agencies have a policy requiring culvert/bridge design to be referred to a structural engineer where the waterway area served by the structure exceeds a specified limit. See AS5100 Set-2007 and local policy for further details.

A floodway is a low-level section of road, specially constructed to allow the passage of floodwater across it without damage to the road. Floodways are generally provided where traffic volumes are low, under the following circumstances:

- where flow across the road will be infrequent or of short duration
- in conjunction with a bridge or culvert, where the bridge or culvert is designed to pass a lesser flood than the total waterway design flood. The bridge or culvert may be designed such that it will not be overtopped or be designed to be submerged as part of the floodway.

The structural design of larger drainage structures is not covered by this Guide. Details on the design of floodways are provided in AGRD Part 5B – Section 4.

4.10.2 Bridges – General

The Guide provides guidance only on the location and layout of a bridge but does not provide a detailed process for bridge design. Additional information on the design of bridges with respect to drainage can be found in:

- Austroads Guide to Bridge Technology series
- AS 5100 Set–2007 Bridge Design Set
- local jurisdictional guidelines.

The Guide does, however, provide designers with an overview of the criteria to be investigated and addressed in establishing:

- deck drainage
- bridge geometry
- bridge skews
- road grade and hydraulic clearance
- span lengths and pier location
- riparian and wildlife corridors under the bridge
• bridge location and waterway alignment
• scour protection
• overtopping of the road
• maintenance requirements.

Table 4.4: Advantages and disadvantages of bridges and culverts

<table>
<thead>
<tr>
<th>Bridges</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Provide greatest flood immunity.</td>
<td>• Higher design and construction costs.</td>
</tr>
<tr>
<td>• Waterway area increases with rising water surface until water surface begins to submerge superstructure.</td>
<td>• Require more structural maintenance than culverts.</td>
</tr>
<tr>
<td>• Less susceptible to clogging with debris.</td>
<td>• Spill slopes susceptible to erosion and scour damage.</td>
</tr>
<tr>
<td>• Scour increases waterway opening.</td>
<td>• Piers and abutments susceptible to failure from scour.</td>
</tr>
<tr>
<td>• Minimal impact on aquatic environment and wetlands.</td>
<td>• Buoyant, drag and impact forces are hazards to bridges.</td>
</tr>
<tr>
<td>• Widening does not usually affect hydraulic capacity.</td>
<td>• Susceptible to stream/channel migration.</td>
</tr>
<tr>
<td>• Less impact on fauna/fish.</td>
<td>• Increased buoyancy, drag and impact risks.</td>
</tr>
<tr>
<td>• Low flow capacity.</td>
<td>• Higher risk for maintenance activities, i.e. traffic management and height.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Culverts</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Changes to vertical geometry and road width can generally be accommodated by extending culvert ends.</td>
<td>• Silting may require periodic cleaning.</td>
</tr>
<tr>
<td>• Require less structural maintenance than bridges.</td>
<td>• High risk maintenance work location being in a stream/river bed.</td>
</tr>
<tr>
<td>• Usually simpler and quicker to design and construct than bridges.</td>
<td>• No increase in waterway as stage rises above soffit.</td>
</tr>
<tr>
<td>• Scour is localised, more predictable and easier to control.</td>
<td>• Generally require higher levels of maintenance due to:</td>
</tr>
<tr>
<td>• Generally the most cost effective option.</td>
<td>- clogging with debris</td>
</tr>
<tr>
<td></td>
<td>- scour at outlets</td>
</tr>
<tr>
<td></td>
<td>- abrasion and corrosion damage.</td>
</tr>
<tr>
<td></td>
<td>• Extension may reduce hydraulic capacity.</td>
</tr>
<tr>
<td></td>
<td>• Inlets of flexible culverts susceptible to failure by buoyancy.</td>
</tr>
<tr>
<td></td>
<td>• Rigid culverts susceptible to separation at joints.</td>
</tr>
<tr>
<td></td>
<td>• Susceptible to failure by piping (leading to failure of embankment).</td>
</tr>
<tr>
<td></td>
<td>• Increased environmental impacts on fauna/fish.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Floodways</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Generally simple to design.</td>
<td>• Allow water flow over road – immunity and safety issues.</td>
</tr>
<tr>
<td>• May offer environmental advantages over culverts and bridges since they will tend to spread flows more widely.</td>
<td>• Increased disruption to traffic due to overtopping.</td>
</tr>
<tr>
<td>• Typically have low embankments.</td>
<td>• Can have higher construction costs than culverts.</td>
</tr>
<tr>
<td>• Risk of scour to waterway and surrounding land is reduced.</td>
<td>• Batter slopes can be affected by erosion/scour (particularly for higher embankments).</td>
</tr>
<tr>
<td></td>
<td>• Generally have costly batter protection requirements.</td>
</tr>
<tr>
<td></td>
<td>• Susceptible to stream/channel migration.</td>
</tr>
<tr>
<td></td>
<td>• Can have environmental impacts (fauna/fish passage).</td>
</tr>
<tr>
<td></td>
<td>• Potential for failure of embankment (depending on provided protection).</td>
</tr>
</tbody>
</table>

Source: Adapted from DTMR (2010b).
4.10.3 Drainage of Bridge Decks

The drainage of bridge decks is an important component of bridge performance. The surface water needs to be removed from the bridge deck effectively to minimise the safety hazard of water on the pavement surface, and prevent corrosion of the structure. The collected run-off needs to be discharged appropriately to meet the environmental requirements, prevent erosion of the surrounding ground and possibly undermining the foundations, and the width of flow on a bridge deck should not exceed that specified for its road approaches. Every effort should be made to ensure that longitudinal sags are not located on bridges.

**Drainage of carriageway**

Transverse and longitudinal drainage of the carriageway should be undertaken by providing a suitable cross-fall and camber or gradient, respectively. Water flowing downgrade on bridge approaches should not be permitted to run onto the bridge unless permitted otherwise by the road agency. To reduce costs, short bridges, should be detailed without formal superstructure drainage wherever possible, with the run-off from the bridge discharged into outfall drains at the end of the structure, as specified by the agency.

Longer bridges require drainage facilities; otherwise flow widths may exceed the allowable limits. Inlet structures, such as flush grates connect to the under-deck pipe work, which discharges away from the structure, waterway, or other thoroughfare beneath the structure. Drainage inlets should be of rigid, ultraviolet and corrosion-resistant material, not less than 100 mm in their least dimension, and should be provided with provision for cleanouts.

Deck drainage should be detailed to prevent the discharge of drainage water against any portion of the structure and to prevent erosion adjacent to the point of impact of the discharge from the outlet of the downpipe. The overhanging portions of a concrete deck should be provided with a drip bead or notch, which should be continuous where possible.

Drainage from bridges should not discharge directly into waterways, onto traffic lanes, railway corridors or any other thoroughfare below. As such the use of scuppers for bridge deck drainage should be precluded.

**Detailing for drainage**

Design details should ensure that water drains from all parts of the structure and should prevent the retention of dirt, leaves or other foreign matter.

Where drainage pipes are provided in the closed cells of bridges, the pipes should be of durable material.

Where pipes carrying liquids are located inside closed cells, drainage should be provided in case of leaking or bursting of the pipes.

**Drainage of ballast railway bridges**

Consideration should be given to the effective drainage of ballast-topped railway bridges, and waterproofing should be provided where necessary.

4.10.4 Bridge Location and Waterway Alignment

Where practicable, the alignment of the bridge should be chosen to avoid unstable sections of a watercourse channel, such as sharp or obviously mobile channel bends.

If piers must be located within the channel, and if a pool is likely to form within the channel at the bridge location, then the foundation design must allow for future bed erosion.
When it is considered necessary to realign a waterway channel as part of a bridge design, the following issues and concerns should be investigated and appropriately addressed:

- potential environmental impacts of sediment run-off from the construction of the new channel
- erosion potential of the downstream channel in response to the proposed realignment
- possible changes to existing bed conditions, including pool-riffle systems, within the channel
- the need for rock protection of the channel bed and banks given the potential to adversely affect the continuity and health of riparian vegetation and consequently the quality of the wildlife corridor
- any reduction in the length of the main channel and a consequential increase in the hydraulic gradient and erosion potential
- the form, condition and location of the low-flow channel; where practical, all of these should be maintained.

The location of the low-flow channel can have a significant effect on channel stability and aquatic habitat values. It can also meander within the bed of the main channel and the form, condition and location of the low-flow channel can vary from flood event to flood event.

### 4.10.5 Bridge Geometry

In establishing the geometry of a bridge, the following aspects need to be considered:

- road grade and height clearances (above accesses, environmental features and services)
- hydraulic clearance (freeboard)
- span lengths and location of piers
- scour protection
- overtopping
- maintenance requirements/access.

**Road grade and hydraulic clearance**

In general clearance is dependent on:

- design road levels
- whether the bridge is to be designed with freeboard or for overtopping
- the size and nature of flood debris
- environmental features
- built features
- navigation requirements (where relevant).

Hydraulic clearance in most bridge designs refers to the amount of freeboard to be provided between the underside of the bridge superstructure and the design flood level.

In some instances, clearance has also to be provided for road access, environmental features and watercraft navigation requirements.
Reference should also be made to jurisdictional legislation and regulations regarding navigable waterways and access rights. It is important to note that when in tidal waters, the relevant authority and/or waterway manager must be consulted. Some jurisdictions may require external approval for bridge spans and vertical clearance for boats (including yachts, where relevant).

**Span lengths and pier location**

In selecting span length, the following factors must be considered:

- bridge design and construction issues
- whether any piers will be allowed within the main watercourse channel
- size of debris that is likely to be present within the watercourse
- required road elevation
- required waterway area to satisfy allowable afflux limits.

The spacing of bridge piers can have an important factor, having a significant influence on the cost of a bridge. It is dependent on:

- the stability of the stream bed and banks
- the environmental sensitivity of the waterway (i.e. should piers be allowed within the low-flow channel)
- the presence of any existing bridge piers – where duplication or upgrade of roadways is planned, new piers should be aligned with existing piers where possible
- geotechnical conditions for pier foundations
- navigational requirements (where relevant)
- required permits.

Where practical, bridge piers should be located away from the low-flow channel. Large-scale turbulence caused by bridge piers located within low-flow channels can adversely affect fish passage and can cause bed and bank erosion.

**Scour protection**

Scour of bridge foundations or abutments can arise when bed or bank surfaces are not designed to resist likely peak velocities. Peak velocities can also be exacerbated when a build-up of debris results in a decrease in waterway area.

The most common form of scour-induced failure relates to scour of the river or creek bed in the vicinity of bridge piers, as shown in Figure 4.2 and/or abutments. During flood events, river or creek beds may be mobilised to significant depths, hence it is necessary to design bridge foundations to remain stable during or following such events.

The selection of appropriate scour protection measures should be based on:

- an understanding of peak velocities (adjacent to both the bed and banks)
- maximum depth of bed mobilisation
- erodibility of bed and bank material
- likelihood of flow deflection occurring (i.e. as a consequence of river geometry)
- an estimation of whether piers are likely to cause deflection of flow (i.e. for a skewed bridge).
For the analysis and design of bridge scour, see AS 5100 Set-2007.

Figure 4.2: Scouring around bridge piers

Source: DTMR (2010b).

Overtopping

The maximum flood design load on a bridge usually occurs when the flood carrying debris is at deck level (not relevant for high level bridges). Hydraulic calculations are required for either a flood at deck level, or the 2000 year ARI flood level (whichever is the higher). Floods at or above superstructure level will also require the consideration of buoyancy factors.

Access for inspection and maintenance

Under workplace health and safety legislation, the finished bridge becomes a workplace for inspection and maintenance personnel. Therefore, provision should be made to facilitate safe work practices.

The design of road, railway, pedestrian and bicycle-path bridge structures must allow for safe access for all inspection and maintenance activities. Anticipated maintenance activities must be listed at the time of design. Maintenance activities are to be in accordance with the road agency inspection and maintenance manuals.

Drainage connections to bridges (including any pollutant control devices) may need to be designed for the ultimate configuration (e.g. need to cope with additional surface run-off from a widened structure).

For further guidance, see the Guide to Bridge Technology Part 4: Design Procurement and Concept Design (Austroads 2009f) and Guide to Bridge Technology Part 7: Maintenance and Management of Existing Bridges (Austroads 2009g).
5. Operations and Maintenance

Appropriate maintenance of drainage infrastructure plays a crucial part in its effective operation. This also minimises environmental harm and provides a level of safety to users of the road corridor.

This section of the Guide is targeted primarily at managers of road maintenance operations and road maintenance contractors. It outlines the maintenance process and uses examples of drainage failures to illustrate the need for effective maintenance operations. It also provides steps for the remediation of problems or deficiencies.

The process outlined in this section relies on the design process undertaken in previous sections and AGRD Part 5A and AGRD Part 5B and reference to the design criteria, assumptions, calculations and assessments within these Guides for the various drainage elements may be required.

Reference should also be made to road agency performance contract manuals and asset maintenance guidelines. The philosophy of this section is to use the maintenance process for identifying failures in the drainage system and to assist learning from these failures to prevent future failures.

Regular inspection and maintenance of road and drainage infrastructure is an essential part of the asset management process aimed at maximising the life of the asset and maintaining the day-to-day operational level of service, minimising environmental harm and ensuring a level of safety to road users. Drainage infrastructure should be designed and constructed acknowledging that such periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users.

An effective design must balance a number of factors against the construction cost of the level of protection proposed including the implications for safe access by maintenance crews and road maintenance cost. Designers should consult with the relevant agency regarding the provision and location of maintenance access facilities and devices.

Safe access needs to be provided to all drainage infrastructure that requires either ongoing (i.e. mowing of drains) or occasional (i.e. removal of debris) inspection and maintenance. This access is required for vehicles and/or maintenance crews depending on the type of maintenance that will be undertaken.

5.1 Maintenance Access and Location

Designers should ensure, as far as it is physically and economically feasible, that safe and efficient maintenance access is provided to all drainage facilities including open drains, inlet structures, pits, channels, piped networks, culverts and outfalls. Drainage infrastructure should be designed to minimise maintenance, provide for safe vehicle and personnel access and have adequate working space for undertaking manual activities. Particular consideration should be given to:

- ensuring drainage infrastructure is located outside the trafficable area for all road users as much as much as possible, including placing pits outside of shared paths
- providing protection for, or suitably delineating, infrastructure that may be susceptible to damage from regular maintenance operations (i.e. the use of coloured guide posts at culvert locations to prevent graders striking headwalls)
- access to pits conforming with workplace health and safety (WHS) requirements with respect to rungs, ladders, landings and working in confined spaces
- outlet screens on pipe/box units up to 1800 mm in width being designed such that the full width of the outfall pipe/box can be accessed for periodic maintenance, including the removal of debris
• culverts or drainage networks under very high fills where there is the possibility of piping and barrel settlement, in which case a minimum 750 mm diameter pipe or 750 mm x 750 mm box allows access to facilitate maintenance inspection

• suitable clearance width between fencing and the top of batter for drainage basins and open drains

• an overall easement/reserve width for open channels having an access/maintenance berm of minimum width 4.5 m on at least one side of the main channel

• the location of maintenance berms within the channel (not desirable in natural channel design) if it is necessary to provide access for mowing or debris removal, or if it is important to obtain maximum hydraulic efficiency within a specified easement width. These berms may be benched into the channel bank at an elevation above the one year ARI flow. In addition, a 1.5 metre wide safety/access strip should be provided along at least one side of the channel above the design flood level in addition to the access/maintenance berm

• maximum channel depth may be limited by maintenance requirements

• where a floodway (very wide berms) is incorporated into a channel, a minimum floodway cross slope of 1 in 80 should be adopted to prevent water logging problems and allow regular maintenance mowing

• vehicle access, including:
  – width and type of construction vehicle for all-year access
  – the use of cross-sections suitable for conventional equipment such as graders, back hoes, front-end loaders, mowers and trucks (i.e. batter widths and flat bottom drains at least as wide as mower/grader blades)
  – clearly identified and delineated entry and exit access to alert maintenance personnel of their location in addition to warning the general public that large, slow moving vehicles may be entering or exiting the roadway

• the use of low maintenance treatments, such as concrete lined or stone pitched channels in locations where it is difficult to provide access for conventional maintenance equipment.

5.2 Operation

The operation of the road and in turn the drainage system commences immediately after the road is opened to traffic. This section deals with the period immediately after construction while Section 5.3 – Maintenance discusses the ongoing maintenance considerations and activities which span the life of the road.

An important function or activity that should be conducted in the period after construction is the inspection/check of the drainage system’s actual operation or performance against the design intent. This can only happen after a reasonable rainfall/storm event and will either validate the design or identify deficiencies. This performance check is particularly important for drainage devices protecting/maintaining water quality. Depending on the deficiency, remedial works (see Section 5.5 – Remediation) may be covered under the defects liability component of the construction project.

It is important to note that the inspection period for the site should be extended to check the performance of any remedial work.

5.2.1 Period of Inspection

It is difficult to define an actual time period for inspection(s)/check(s) to be undertaken, as rainfall events are unpredictable and therefore it is impractical to define a set time period. As stated in the Section 5.2 – Operation, the purpose of the inspection is to check the operation or performance of the drainage system and this requires at least one significant storm or rainfall event to test the system. It is recommended that several rainfall events should be monitored/checked to ensure the on-going, successful operation of the road drainage system.
It is also recommended that inspections should occur as soon as possible after a rainfall event has commenced so that actual stormwater flows can be observed/tested.

5.2.2 Performance

Drainage infrastructure is constructed to primarily carry/transfer and possibly treat stormwater. These devices are designed for a certain discharge and/or capability. With respect to drainage devices constructed to protect and/or maintain water quality, they have been designed to meet specific water quality requirements. It is important that the performance of these devices be checked to ensure that they are achieving the design requirements. If it is found that the requirements are not being met, the site:

- must be fully investigated to determine the reasons why the device is not achieving required targets
- requires evaluation for the appropriate remedial action to be planned and designed to correct the deficiency
- should be listed for remedial work to be undertaken.

This investigation and remedial work must be undertaken as soon as possible after the deficiency has been identified as the risk of causing harm or damage will remain elevated until the work is completed.

5.3 Maintenance

5.3.1 Maintenance Process

Part of the management of a road is to maintain the road network to a standard which ensures the safety and efficiency of the travelling public and protection of the environment.

The road drainage infrastructure (or system) is designed for a certain discharge and/or capability and needs to be properly maintained to ensure continued performance. Poor maintenance reduces the performance (capacity) of the drainage device or system and this in turn can increase the risk of:

- upstream flooding
- failure of the device/system and potentially the road
- accelerated deterioration of the road asset
- accidents (such as unexpected water on the road surface)
- damage to the environment.

Maintenance works on the state controlled road networks should be undertaken in accordance with agencies’ maintenance contract requirements.

5.3.2 Types of Maintenance

Routine maintenance is the most common type of work undertaken on drainage infrastructure.

Emergency maintenance work relates primarily to work performed immediately following an emergency (e.g. vehicle accident, natural event) to ensure the safety of motorists and/or pedestrians using the corridor. Other routine maintenance work may be necessary after making the situation safe. Drainage structures should be designed and constructed acknowledging that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users.
5.4 Drainage Failures

A failure in road drainage may be caused by any number of problems or combination of problems and can occur during the construction or operation of the road. Erosion at culvert outlets, undermining of pavement and drainage structures, soil loss on steep batters and sedimentation of drains are all common failures. Figure 5.1 illustrates a drainage failure that has occurred during construction.

While flooding of the road corridor is the most common problem resulting from insufficient drainage capacity or a blockage of cross drainage, there are other issues that can occur as a result of failure (reduced performance/capability).

Figure 5.1: Drainage failure

Source: DTMR (2010b).

5.4.1 Causes of Failure

Failures in the drainage system or device/component may result from a number of situations. Some of the more prominent causes of failure are:

- inadequate/inappropriate design
- poor construction practice/post-construction inspection
- changes to the site’s physical conditions (e.g. alteration to landform, vegetation, and surrounding land use)
- poor maintenance inspection/practice
- an extreme rainfall event which delivers a storm much greater than the design storm the drainage infrastructure was designed to handle
- out-of-specification materials.

Failure may also be caused by site maintenance operations. For example, the re-grading or re-cutting of table drains (to clean out silt and so on) may accidentally knock out table drain blocks. These devices are typically used on downgrades to ‘dam’ stormwater flow and direct it into culverts to carry the stormwater across the road. If these blocks are removed, the stormwater will bypass the culvert and continue downgrade until it reaches the last or lowest point culvert. This culvert will now be expected to carry much more stormwater than it was designed for which could result in higher outlet velocities or even water overtopping the road. Where water overtops the road, the risk of an accident occurring is greatly increased.
Another example could be where a slasher, mowing a narrow grassed verge at the toe of a steep batter, damages the batter toe. During a storm event, run-off over the exposed batter face could lead to erosion and undermining of the batter toe which could lead to slippage of the batter slope.

An actual or potential failure in the road drainage infrastructure is often not evident until it has been subject to storm events or similar conditions. For example, the scouring of a creek bed downstream of a culvert outlet may not occur or be evident until the first storm event.

It is recognised that maintenance of the drainage infrastructure plays a crucial part in its effective operation.

5.4.2 Types of Failure

A number of types of failures are evident in the road drainage system. The more common failures include:

- surface/slope/bank erosion
- undermining or piping
- sedimentation
- debris accumulation (e.g. litter and vegetation)
- structural failure of the drainage device/component.

These are discussed below.

**Erosion**

Erosion is the most common failure. Roads tend to concentrate stormwater flows which in turn increases flow velocities and energy. This combination increases the risk of erosion and scour. Figure 5.2 shows an example of erosion on a fill embankment.

**Figure 5.2:** Erosion of a highly dispersive soil

Source: DTMR (2010b).

Erosion can occur at or in:

- culverts (inlets and outlets)
- piped network (outlets)
• bridges
• floodways
• diversion channels
• catch banks and drains
• table drains
• unsealed shoulders, verges and batters.

**Undermining or piping**

Undermining refers to the loss of soil from underneath some part of the road infrastructure (e.g. pavement surface, concrete-lined drain, culvert apron). This can result in direct damage to the road infrastructure such as cracking or slumping. Figure 5.3 shows an example of undermining.

Piping is the term used to describe the mode of embankment failure that involves the washing out of the smaller soil particles from a section of the road embankment by water leaking through a weak point in its structure. The weak point can occur either from the side on the embankment, through the top surface of the embankment via cracks in surfacing or unsealed shoulders or through gaps in disjointed culverts. The progressive removal of fine soil particles will further increase the rate of flow of water and the rate of removal of the soil particles. As the rate of leakage accelerates, larger soil particles can be transported. Eventually localised collapse of the embankment will occur.

*Figure 5.3: Undermining-dispersive soil*

Source: DTMR (2010b).

**Sedimentation**

Sedimentation is another common cause of failure and is the deposition of soil that has been transported by flowing water. Soil particles settle once the flowing water has slowed or stopped. This often occurs in culvert inlets and outlets as well as creeks and other watercourses. Figure 5.4 shows an example of sediment deposition in a culvert. This failure is also termed blockage and reduces the capacity of the culvert, which in turn can increase flooding (afflux) upstream.
Debris accumulation

Debris accumulation includes the accumulation of vegetation, litter and other gross pollutants in the drainage system. This may be caused by insufficient hydraulic capacity of the drainage structure, the size of debris entering the drainage system or lack of maintenance. Figure 5.5 shows debris accumulation via the growth of vegetation in the outlet of a culvert.

Structural

Structural failure is the failure of a drainage structure either by separation of units making up a single structure (e.g. the disjointing of culvert pipes or box units or the headwall separating from the barrel) or the actual structural failure of a unit (e.g. the collapse of a concrete pipe unit due to excessive loading or the collapse of a steel culvert due to weakening by corrosion).
5.4.3 Environmental Impacts of Failures

Drainage failures have the potential to cause environmental harm. Some common impacts include:

- disruption to vegetation (direct terrestrial and aquatic habitat loss)
- soil erosion and sedimentation
- altered stream hydrology
- altered overland flow paths causing soil moisture changes
- weed invasion.

5.4.4 Identifying Failures

As discussed, routine maintenance of the road corridor is generally carried out by either the road agency maintenance staff or maintenance contractor. Part of the maintainer’s role is to undertake regular surveys and inspections of the corridor to identify and prioritise maintenance works.

Significant failures, such as that shown in Figure 5.6 need to be identified quickly so that appropriate remedial works can be determined, approved and undertaken.

Figure 5.6: Culvert headwall scouring

Source: DTMR (2010b).

The inspection and reporting process of failures should form part of the maintenance activities. Any failure reports could also be submitted as part of a regular report on the operation of the drainage network, if required.

Reports should include/detail:

- the identified drainage failure(s)
- relevant field notes/photographs
- identification of potential remediation types.

This information can then be used to assist in the development of an appropriate remediation solution.
Following the completion of the report, the maintenance staff or contractor, in consultation with the department, should prioritise remediation works and/or report maintenance requirements.

Descriptions of remediation options are provided in the following section. Table 5.1 summarises the key drainage failures, causes and the potential resultant impacts that may be experienced with roads.

5.4.5 Reporting of Failures

It is important that failures of drainage devices or system be identified, investigated and reported. Reports must identify the reason or cause for the failure.

While reports are processed and subsequent remedial works undertaken, it is advantageous that copies of reports and determined remedial action are obtained to enable a review of drainage failure/deficiencies, which may lead to changes in design/construction methodology, and/or documents and policy.
Table 5.1: Common drainage failures, causes and impacts

<table>
<thead>
<tr>
<th>Failure type</th>
<th>Failure</th>
<th>Cause</th>
<th>Examples of potential impact</th>
</tr>
</thead>
<tbody>
<tr>
<td>Erosion</td>
<td>Scouring at culvert outlet</td>
<td>• Lack of downstream energy dissipation/scour protection.</td>
<td>• Sedimentation of downstream waterways.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• High velocity through culvert.</td>
<td>• Decrease in water quality.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Underestimation of catchment size.</td>
<td></td>
</tr>
<tr>
<td>Scour of bridge piers</td>
<td></td>
<td>• Obstruction to flow.</td>
<td>• Damage to bridge.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Shape of piers (poor design).</td>
<td>• Scour of stream.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Poor construction method.</td>
<td>• Compromise of public safety.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Concentration of floodplain flow.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Velocities exceed threshold values.</td>
<td></td>
</tr>
<tr>
<td>Slips on cuttings</td>
<td></td>
<td>• Cutting too steep.</td>
<td>• Blockage of road.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Poor drainage.</td>
<td>• Loss of soil.</td>
</tr>
<tr>
<td>Erosion of creek banks</td>
<td></td>
<td>• Skewed culverts discharging high flows into creek bank.</td>
<td>• Loss of riparian habitat for semi-aquatic wildlife such as platypus, water rat.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Bridge piers constructed on creek banks.</td>
<td></td>
</tr>
<tr>
<td>Erosion of road embankments/batters</td>
<td></td>
<td>• Flood overtopping road and inadequate batter protection.</td>
<td>• Weed invasion.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• No catch drains provided.</td>
<td>• Undermining of road surface.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Batters too steep for revegetation.</td>
<td>• Blockage of table drains.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Lack of permanent erosion protection.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Pavement overlay often raised and shoulders left at lower level causing run-off to drop onto exposed embankment.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Excessive mowing/slashing of road shoulders.</td>
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</tr>
<tr>
<td>Scouring of sodic soils</td>
<td></td>
<td>• Concentration of run-off.</td>
<td>• Sedimentation of downstream waterways.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Lack of controlled drainage.</td>
<td>• Decrease in water quality.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Lack of surface protection.</td>
<td></td>
</tr>
<tr>
<td>Scouring around culvert headwalls</td>
<td></td>
<td>• Run-off drains behind headwalls and causes scour.</td>
<td>• Sedimentation of downstream waterways.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Lack of vegetation cover owing to weed spraying.</td>
<td>• Decrease in water quality.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Insufficient culvert lengths resulting in steep embankments above headwall.</td>
<td></td>
</tr>
<tr>
<td>Scouring of channels and table drains</td>
<td></td>
<td>• Inadequate scour protection.</td>
<td>• Loss of vegetation and fauna habitat.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Maintenance activities stripping table drains of vegetation.</td>
<td>• Loss of vegetation for water quality filtering purposes.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Channels and drains excavated to carry much larger capacities.</td>
<td></td>
</tr>
<tr>
<td>Scouring of bridge abutments</td>
<td></td>
<td>• Blockage of bridge waterway.</td>
<td>• Sedimentation of downstream waterways.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Design underestimates degree of construction disturbance around abutment thus resulting in exposed areas subject to erosion.</td>
<td>• Decrease in water quality.</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Abutment surfaces difficult to revegetate.</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Lack of controlled drainage.</td>
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</tbody>
</table>
## 5.5 Remediation

### 5.5.1 Introduction

The purpose of remediation is to restore the drainage device or system to the level of performance/capability it was designed for.

After inspection/checks have been carried out and deficiencies and/or failures have been identified and investigated, appropriate remedial work options need to be developed and the best solution determined.

### 5.5.2 Remediation Options

The remediation option may include one or a number of the following:
- one-off repair work
• augmentation of existing works
• re-design (and re-construction)
• alteration to maintenance practices.

Field inspection to review site conditions, failures and remediation options, should be undertaken to confirm the most appropriate option/solution.

**Repair**

A failure may be attributed to a one-off event such as a severe or extreme storm event, vehicle accident, or vandalism. Such instances may require repair work. General ‘wear and tear’ on the drainage system will also require repair.

Repair work should provide for the drainage system to operate to its intended design function and capacity. Such repair work would require that the failure does not continue.

Changes to routine maintenance activities may be required following repair works. These changes in maintenance activities would need to be specific to the repair works undertaken. The maintenance contractor’s routine maintenance practices should be amended as necessary to accommodate these changes.

**Augmentation**

Augmentation or retrofitting of existing drainage works may be required to avoid the continuation of an existing failure.

A review of the site conditions that influence the operation of the drainage structure is required. This may include a review of:

• flow velocity
• soil type/erosion risk rating
• catchment area
• receiving environment.

The type of augmentation works required will depend on the failure identified.

**Re-design**

Where simple repair or augmentation works are not likely to or will not correct the problem, a full review of the failure and original design intent followed by the re-design process will be required to develop a suitable solution. It is important to review the original design to identify possible deficiencies in the design (which lead to failure) and to ensure any re-design work is compatible with the rest of the drainage system.

The focus of re-design is to ensure that the new drainage structure avoids a recurrence of the drainage failure caused by the same factors.

**Maintenance practices**

Where an existing maintenance practice or activity is causing a drainage failure, it should be reviewed and modified. New practices may also need to be devised and implemented.
Some common failures of the drainage system resulting from maintenance activities include:

- over-clearing or mowing of grassed table drains (Figure 5.7), batter toes and other areas resulting in soil exposure and subsequent erosion
- weed spraying resulting in vegetation loss and subsequent soil erosion
- clearing of vegetation from headwalls and at culvert outlets resulting in soil exposure and subsequent erosion
- machinery hitting and damaging drainage infrastructure which is hidden from view by vegetation (such as headwalls) and so on. Damage could be structural or involve exposure of backfill material supporting the structure (which is then subject to erosion).

Figure 5.7: Grassed table drain damage

Source: DTMR (2010b).

If a failure is attributed to maintenance activities, a review of existing practices and implementation of new practices should be undertaken. Table 5.2 and road agency guidelines should be reviewed in relation to appropriate maintenance practices.

5.5.3 Evaluation

In order to evaluate the effectiveness of maintenance practices, and reduce the potential for future drainage failures, information recorded during the maintenance process should be reviewed. This information can then be used for the planning, design, construction and maintenance of future drainage systems.

Those undertaking road maintenance works are typically required to measure and record all maintenance work undertaken. This is undertaken to provide a basis for progress payments if appropriate. This information provides an insight into the type and frequency of failures, the frequency of maintenance works and types of remediation.

To supplement this information, the following would also assist in evaluating maintenance activities and remediation techniques:

- observations and records of recurring problems and failures and any resultant environmental harm
- practicable and effective remediation options to suit site-specific failures.

Review of the above information should be undertaken by road drainage planners and designers to minimise the potential for recurring failures of the drainage system in the future.
Table 5.2: Summary of maintenance practice

<table>
<thead>
<tr>
<th>Drainage failure</th>
<th>Required maintenance</th>
<th>Best practice guidelines</th>
</tr>
</thead>
</table>
| Erosion          | Backfilling, Mulching, Revegetation, Reprofiling | • Determine cause of erosion (e.g. concentration of high velocity flow).  
• Identify actual and potential sediment sources during routine maintenance.  
• Plan to protect exposed areas and sediment sources as soon as practicable.  
• Minimise the amount of disturbance to vegetated and other sensitive (e.g. waterway) areas.  
• Minimise the amount of disturbance to unstable areas (e.g. drains, banks, batters).  
• Plant only indigenous species native to the area. |
| Undermining      | Backfilling, Installation of protection measures (e.g. rock, concrete) | • Minimise the amount of disturbance to unstable areas (e.g. drains, banks, batters).  
• Simulate natural conditions where practicable. |
| Sedimentation    | Hand removal of sediment, Mechanical removal of sediment, Disposal of sediment | • Remove and dispose of sediment in an area where it is unlikely to wash into a drainage line or waterway (i.e. flat area as far away as practicable from the nearest drainage line).  
• De-water sediment prior to disposal to reduce the potential for sediment-laden run-off draining from the disposal site.  
• Identify any potential for sediment to be contaminated from industrial or other sources. Where sediment is likely to be contaminated, contact should be made with the local authority to determine the most suitable disposal location and method of transport.  
• Identify opportunities for re-use of uncontaminated sediment (e.g. top dress landscaping, use in backfilling).  
• Clean drains in a manner that does not result in soil disturbance or exposure to erosion. |
| Debris accumulation | Hand removal of debris, Mechanical removal of debris, Disposal of debris material | • Ensure that any potential changes or inadequacies in hydraulic capacity of the drain, culvert inlet or similar are identified and managed to minimise potential environmental harm.  
• Ensure access is sufficient to allow the full removal of all debris.  
• Continual debris build-up may indicate culverts or bridge spans are of insufficient size. |
| Weed removal     | Removal of vegetation, Herbicide application | • Ensure control agents are appropriate for the purpose and that only the necessary rates of application are used.  
• Ensure that chemical spill containment and clean-up equipment is available when chemicals are being used and staff is trained in the use of such equipment. |
| Structural       | Joint repair, Pipe collapse | • Regular inspection of pipes.  
• Identify causes of pipe joint failure or pipe collapse.  
• Review applicability of the type of pipe joints suitable for the location.  
• Identify changes in traffic loadings and consider placing load limits. |

Source: DTMR (2010b).
6. Hydrology

6.1 General

Hydrology for road drainage design is the estimation of flood run-off from a catchment. This is normally expressed as a peak flood discharge, given as a volume per unit time, normally cubic metres per second (m³/s). The calculation can be based on an assessment of rainfall or on recorded stream flow data. Methods for calculating peak flood discharges described in AR&R, Vol. 1 and Vol. 2 (Pilgrim 2001, 2007) in order of preference include:

- flood frequency analysis (FFA), where a sequence of historical flood flows have been recorded in the catchment
- flood routing, with calibration for catchments that have observed flood data
- flood routing, without calibration for catchments that do not have observed flood data
- probabilistic rational method (PRM).

The choice of method usually depends on the amount of information available and/or assessment of risk. In some cases, the drainage authority or local government may have undertaken a study that provides peak flood discharge rates. Information from available and appropriate studies should be used in preference to the PRM.

For catchments where there is no available detailed study, but the calculation of peak discharge is a key factor in sizing substantial infrastructure, the use of flood routing to determine peak flood discharge rates is likely to be the most cost effective.

There are numerous cases where a relatively quick analysis method is appropriate for calculating peak flood discharges – the PRM was developed for these cases. For routine analysis of small catchments, it is almost always based on analysis of rainfall, and this is the procedure described in this Guide.

It is well understood that rainfall events or storms are variable and dynamic in nature. That is, rainfall:

- is not of uniform intensity over the whole of the catchment
- may not cover the whole catchment.

The principal requirement for routine drainage design is that it is based on peak discharge from the catchment for a specific average recurrence interval (ARI) or range of intervals. These can be calculated by relatively simple procedures. To complete these calculations, some simple assumptions are required to define the design storm and these assumptions tend to produce a conservative peak discharge. While peak discharge allows the sizing of hydraulic components, the time variation of in-flow to a drainage system is required when flow mitigation, storage and stormwater treatment devices need to be designed. In order to develop the appropriate design in-flow hydrograph, many factors need to be considered and it is recommended that specialist hydrologist advice be sought.

Understanding the hydrological conditions of the project site is vitally important. These conditions include:

- rainfall (duration and intensity)
- type of precipitation (rain, hail, sleet or snow)
- topography (slope and soil type)
- catchment area and shape
- land use
• vegetation coverage
• water flow paths
• areas of water inundation and storage
• water harvesting (farm dams)
• volume of run-off generated.

Understanding these aspects ensures that any drainage system design:
• meets the drainage design criteria
• is economical
• is safe for all users
• protects investment in the road asset
• protects the environment from harm.

6.2 Rainfall – Run-off Relationship

A key hydrologic concept designers need to understand is the relationship that exists between rainfall and run-off. As rainfall hits the ground, run-off is generated from the catchment. Therefore, a relationship between the rainfall that hits the ground and the run-off generated exists for a given catchment and discharge point. This relationship can be plotted as flood discharge against time and the resulting graph is called a hydrograph. Should the discharge point (e.g. culvert site) or ARI change, the relationship will change.

An actual or ‘real’ discharge hydrograph plots the flows from rainfall events and could have several peaks. The rainfall pattern for particular storm events may be complex and may vary from one event to another so this hydrograph may also be complex. However, for design purposes, design flood hydrographs are usually calculated using an idealised, theoretical procedure and a ‘synthetic’ hydrograph is produced. This plot will usually have only one peak for each ARI event. Figure 6.1 shows examples of actual and synthetic hydrographs.

Hydrographs are useful tools particularly when reviewing total flow volume and time of flow for a catchment and selected ARI. Also, hydrographs are used for the determination of time of submergence and time of closure of roads or road structures.

The development of a hydrograph can be difficult and generally requires specialist hydraulic engineering input. However, for the design of most road drainage infrastructure, the two key points on a hydrograph axis that are of most interest to designers are the peak discharge generated (y-axis) and the time it occurs (x-axis). The hydrologic method presented in this guide simply calculates these two points.

The time that peak discharge occurs is referred to as the time of concentration \( t_c \) for the catchment. This is discussed further in Section 6.6.2 – Time of Concentration. If required, the duration of the design storm event can be estimated as; storm duration approximately equals 2.7 times \( t_c \). This duration, together with the peak discharge and \( t_c \), can be used to plot a simple triangular hydrograph for the catchment. For further information on this relationship refer to AGRD Part 5A – Section 7.2.8.

In order to raise confidence with the run-off assessment of any catchment, the designer should make best use of available historic flood data from published reports, technical papers, other related hydrological studies or using anecdotal evidence/records as appropriate.
6.3 Rainfall

The rainfall characteristics for each specific geographical location form the basis for the design of road infrastructure. Rainfall intensity in particular, is the key variable required for procedures used to calculate design floods.
In New Zealand the key reference for design rainfall data is the National Institute of Water and Atmosphere (NIWA) software package High Intensity Rainfall Design System (HIRDS). It is recognised that there are some low quality rainfall record areas in New Zealand and alternative rainfall data, such as that available from the *The Frequency of High Intensity Rainfalls in New Zealand* (Tomlinson 1980) and actual rain gauge records should be used to verify the design data.

In Australia, design rainfall intensities are published in *AR&R* Vol. 2 (Pilgrim 2007). This is the primary reference used for most design rainfall calculations throughout Australia though alternative methods have been used in unique circumstances.

The design rainfall intensity data is provided for all ARIs up to 100 years and for standard durations ranging from 5 minutes to 72 hours. The different ARIs are needed to assess floods of different risk levels and the range of durations allows for different catchment response times to be considered. The design rainfall intensities provided in *AR&R* Vol. 2 (Pilgrim 2007) have been determined by the Bureau of Meteorology and have been based on extensive rainfall data collected throughout Australia.

*AR&R* Vol. 2 (Pilgrim 2007) provides a detailed procedure for calculating design rainfall intensities and this procedure should be followed in all cases. Intensity-Frequency-Duration (IFD) tables for selected ARIs can be developed either manually, by using specialised software or obtained through the Bureau of Meteorology (see Figure 6.2). Determination and application of design rainfall intensities is discussed further in Section 6.3.2 – IFD Tables.

### 6.3.1 Rainfall Intensity, Frequency and Duration

Intensity, frequency and duration are the three parameters used to define rainfall events as follows:

- **Intensity** – the rate at which rainfall occurs and is measured in millimetres per hour (mm/h). The intensity of a tropical downpour (say 100 mm/h) is much greater than a light shower (say < 1 mm/h).

- **Frequency** – how often a particular storm event is likely to occur. Frequency is usually expressed as the average recurrence interval (ARI). See *AR&R* Vol. 1 (Pilgrim 2001) and the Australian Bureau of Meteorology (BoM) website for further explanation and details of these terms.

- **Duration** – how long the storm event lasts.

These three parameters are inter-linked. Higher intensity rainfall events occur less frequently than lower intensity rainfall events and for a given rainfall intensity, longer duration events are less frequent than short duration events.

Storm frequency (ARI) is usually fixed by the design documentation and is generally a function of the type and importance of the facility being designed and the consequences of it being overtopped and/or closed to traffic. Evaluation of risk needs to be considered. Typically, minor drainage elements are designed for frequent (smaller) storm events, and major drainage elements are designed to cater for less frequent (but larger) storm events. The issue is whether a short, intense storm will create a greater discharge impact than a long, less intense storm.

Storm duration is largely dictated by catchment characteristic and how quickly run-off drains from the catchment.

The design rainfall intensity is dependent on storm frequency and duration. The design rainfall intensity varies with location and topography and this should be considered when using the rainfall intensity-frequency-duration (IFD) calculations.
6.3.2 IFD Tables

For calculation and production of design IFD rainfall tables, designers can use any of the following options:

1. The tool provided on the Australian Bureau of Meteorology website. This tool requires the Latitude and Longitude coordinates of the city, town or location of interest\(^3\).

2. By manual calculations as described in \textit{AR&R} Vol. 1 (Pilgrim 2001) which requires the input of nine parameters as determined from maps contained in \textit{AR&R} Vol. 2 (Pilgrim 2007). These parameters are:
   - \(2I_1\) (2 year, 1 hour log-normal rainfall intensity)
   - \(2I_{12}\) (2 year, 12 hour log-normal rainfall intensity)
   - \(2I_{72}\) (2 year, 72 hour log-normal rainfall intensity)
   - \(50I_1\) (50 year, 1 hour log-normal rainfall intensity)
   - \(50I_{12}\) (50 year, 12 hour log-normal rainfall intensity)
   - \(50I_{72}\) (50 year, 72 hour log-normal rainfall intensity)
   - \(G\) – skewness factor
   - \(F_2\) – geographical factor
   - \(F_{50}\) – geographical factor.

3. By using an IFD software application which also requires input of the nine parameters as described above. A tabulation of IFD values for durations from 5 minutes to 72 hours and average recurrence intervals from 1 to 100 years is a standard output from most IFD software but values for non-standard times are also readily obtained.

An important observation of an IFD table is that the longer the duration of rainfall, the lower the intensity of the storm.

\textit{AR&R} Vol. 2 (Pilgrim 2007) also details the use of temporal patterns and areal reduction factors. Temporal patterns are applied to large catchments to allow for rainfall intensity variations across the catchment over the storm duration. Areal reduction factors are applied to point rainfall intensities to address the issue that the application of point rainfall values over a large catchment is unrealistic as such intensities are unlikely to be maintained across the entire area. An area size factor is used in Victoria (see Appendix F).

These two aspects are not considered for use with the simple hydrologic method presented in this Guide because:

- temporal patterns are not needed since the adopted method only uses a uniform rainfall intensity based on the time of concentration
- the adopted method is limited to small catchments up to an area of 25 km\(^2\) where the areal reduction factor is approximately equal to 1.0.

Figure 6.2 shows an example of a typical IFD table. The table provides rainfall intensities in mm/h for various durations and return periods.

\( ^3 \) Further information on obtaining rainfall information is provided in Appendix D.
6.4 Method for Run-off Calculation

There are several methods or techniques available for flood estimation in various sized catchments and these procedures are described in detail in AR&R Vol. 1 (Pilgrim 2001). Road agencies or project owners may specify a method they require or prefer and this will be specified within local guides or design documentation.

6.4.1 Rational Method

The adopted standard method of run-off calculation for small rural and urban catchments is the Rational Method. The Rational Method is a simple, statistical method used to calculate peak discharge from a catchment for a given ARI and is widely accepted and used in Australia and internationally. The Rational Method has its limits and these are discussed in Section 6.4.2 – Applicability of Rational Method. Use of this method outside of these limits can give poor or inaccurate results.

The Rational Method assumes a relationship between the duration of a constant intensity rainfall event required to produce peak outflow from a catchment and the longest travel time or ‘time of concentration’, tc, of the catchment.

The application of the Rational Method is based on the following assumptions:

- the rainfall has a uniform area distribution across the catchment
- the rainfall has a uniform time distribution during the time of concentration
• the peak discharge occurs at the end of the critical storm duration or time of concentration
• the run-off coefficient remains constant throughout the duration of the storm
• the return period of the peak flow is the same as that of the rainfall intensity.

6.4.2 Applicability of Rational Method

As a guide the Rational Method can be used to estimate the peak discharge for small and simple rural catchments up to 25 km² in area and due to complexity, urban catchments up to 1 km². For commentary on the accuracy of the different peak discharge assessment methods refer to McKerchar and Macky (2001).

The method is not applicable for complex catchments, irrespective of size. Complex catchments include:
• multiple streams
• branched catchments
• mixed land use catchments
• situations where a catchment may be inundated by another catchment
• situations where the catchment may overflow into an adjacent catchment
• catchments with significant storage capacity (dam, swamp and major retention/detention basin)
• irrigated land.

6.5 Catchments

Defining the extent and features of the catchment is fundamental to determining the catchment discharge, as they will largely determine the design rainfall intensity and duration and the extent to which rainfall is converted to run-off.

Some features that need to be considered during the assessment include:
• catchment area
• topography
• geology
• extent and nature of vegetation cover
• land use
• existing drainage infrastructure (including potential outfalls)
• impact on flora, fauna, sites of historical or cultural importance, etc.
• catchment retention or detention characteristics.

Several of these features can be resolved from examination of existing information such as topographic maps, construction records, aerial photographs, etc. Some features however will need to be confirmed by a survey of, or visit to, the proposed site.

6.5.1 Catchment Area

The catchment boundary needs to be clearly defined in relation to topographic information. The elevation of outfalls and any receiving waters within the catchment need to be identified as well as locating any water bodies such as ponding basins or wetlands that will affect run-off characteristics. However, in very flat areas these may require careful interpretation and site inspections and/or detailed topographic survey.
In most cases, catchments will have to be subdivided into sub-catchments because of the requirements of a hydrological model.

The catchment area (in hectares or km²) is typically determined from topographical mapping, i.e. contour maps, aerial laser survey, field survey, aerial photographs used as stereo pairs for the basis of photogrammetric contour plots, etc.

In some cases, especially in flat terrain, catchment boundaries may also be defined by infrastructure, particularly roads. These tend to change the catchment both in a topographic sense (cut and fills may change overland flow patterns) and in a drainage sense (intercept and divert overland and subsurface flows into engineered routes).

Contour maps are available for Australia and New Zealand and for most urban areas these are quite detailed. Some areas are also covered by digital maps or web-based satellite imagery. Whichever is used, drainage designers should seek the most current and the most detailed.

Catchment boundaries interpreted from aerial photographs in heavily timbered or flat country can be misinterpreted, therefore some form of site verification should be undertaken.

In urban areas, the determination of catchment boundaries can be difficult but, because all the features are man-made, it should be possible to quantify these by liaison with the various drainage authorities. Natural boundaries are often affected by road works, railway embankments or other earthworks, building works, underground piped stormwater networks and property fences. In complex situations, catchment boundaries should be verified by site inspection and survey.

6.5.2 Catchment Development

Consideration should be given to existing drainage infrastructure and the influence of potential developments on the adequacy of existing systems.

A natural catchment is a complex environment in which vegetation and soils have established a natural balance with the size and form of the stream channel. Generally, less than 20% of storm rainfall on a natural catchment (but can be up to 70% in tropical areas) is discharged as surface run-off. Some rainfall evaporates directly from the surface in hot weather. Some rainfall infiltrates into the soil and is added to groundwater storage, or is taken up by trees and shrubs and eventually transpires to the atmosphere.

Urban development reduces the amount of vegetative cover and introduces impervious areas such as roofs, roads, car parks and concrete paving. This reduces the level of infiltration and evapotranspiration, resulting in increases in run-off and also a reduction in the time of concentration of the catchment (affecting the design rainfall). This is illustrated by Figure 6.3 which shows the relationship between the peak discharge and the imperviousness of the catchment.

In rural areas the agricultural activity should be considered. Intense agricultural uses, such as market gardens can often increase erosion due to regular tillage and irrigation. They can create water pollution due to the use of fertilisers and insecticides. This may influence what water treatments are possible and what pollutant loads are likely to be contained in any run-off.

If there is existing drainage infrastructure within the proposed scheme, the details should be identified and capacities determined or estimated. Any utilities that occur within the scheme should be identified and marked on plans. All rights of way should also be identified.
6.5.3 Future Developments

Where proposals for future development of the catchment exist, there may be a requirement to provide additional drainage capacity during first stage works to cater for the ultimate design drainage requirement. The legal implications may vary between jurisdictions and should be checked.

In the absence of any legal requirements, the decision to provide additional drainage capacity during first stage works should consider:

- disruption to traffic during reconstruction at a later date
- whether an increase in upstream water levels due to backwater will be possible at a later date
- whether an increase in downstream flows will be permitted at a later date.

Before the task of designing a stormwater drainage scheme for a developing catchment can be commenced, the drainage designer should obtain or compile a plan for its ultimate development. The catchment drainage plan should be integrated into the master drainage plan for the drainage basin in which it is located (where such a master drainage plan is available).

Urban zones can remain remarkably static for long periods, e.g. some rural village residential blocks as well as areas that have reached a high level of commercial and/or industrial development. Others, in particular the low and medium-density residential suburbs, frequently experience rapid growth and change. This may follow land value increases or the enactment of by-laws aimed at reducing urban sprawl.

Drainage designers should derive ultimate development predictions by extrapolation and engineering judgement. Examination of regional long-term planning predications and development trends taking place in the areas of concern and in similar areas elsewhere will assist in this endeavour. The time horizon for such development prediction should be 30 to 50 years. Information that can assist in predicting the fully developed state of the catchment can be obtained from:
- aerial photographs (may help define development corridors between centres of activity, stereoscopic review may help interpretation of landforms)
- contour maps and models (also assist in developing catchment boundaries)
- site inspection (to distinguish features that are less than the contour interval of maps)
- government published projections and infrastructure development plans (see items such as proposed road networks, the Natural Resources Atlas, Australian Local Government Association, individual city and regional planning strategies, etc.).

In its ‘ultimate’ state, a catchment, particularly in urban areas, will have evolved almost certainly as an internal drainage network extending that originally provided. The peripheral components of the long-term development, e.g. rear-of-allotment drainage channels etc. may be omitted in the first-stage building program. However, the principal in-ground components and areas for above-ground amelioration of flood waters required by the ultimate drainage support system (i.e. underground pipes, kerb inlets, junction pits, retardation basins, etc.) are provided and designed to cater for the stormwater load which the system will in time be called upon to carry. The completed drainage network must of course be assumed in the design and selection of these components.

### 6.5.4 Catchment Retention or Detention Characteristics

The discharge from a catchment may be substantially modified by natural or man-made retention or detention structures or features. These may be subject to specific requirements which should be determined during the design process.

Some simple run-off models ignore the effects of on-site retention or detention characteristics. This can be appropriate for small catchments particularly in urban areas and where the run-off characteristics of the catchment have not been determined from an analysis of historical records.

Retention and detention can be provided by installation of purpose-built basins, or seek to use existing facilities such as lakes, wetlands, etc., provided the water quality of the run-off is appropriate.

### 6.6 Rural Hydrology

#### 6.6.1 Rational Method

Peak catchment discharge is estimated using the Rational Method and is calculated using Equation 3 below:

\[ Q_Y = k \times C_Y \times I_{tc,Y} \times A \]

where

- \( Q_Y \) = Flow rate, \( Q \) (m\(^3\)/s) for an ARI of \( Y \) years
- \( k \) = A conversion factor, \( k = 0.278 \) when \( A \) is in km\(^2\) and \( 0.00278 \) when \( A \) is \( A \) hectares (ha) and for smaller catchments (i.e. urban) when \( A \) is m\(^2\), \( k = 0.278 \times 10^{-6} \)
- \( C_Y \) = Run-off coefficient, \( C \) (dimensionless) for an ARI of \( Y \) years
- \( I_{tc,Y} \) = Average rainfall intensity, \( I \) (mm/h) for design duration of \( t_c \) (time of concentration, see Section 6.6.2 – Time of Concentration) and ARI of \( Y \) years
- \( A \) = Area of catchment either hectares, km\(^2\) or m\(^2\)
It is important to note that the Rational Method has difficulty in modelling temporary storage, infiltration, retention, evaporation or variation in rainfall intensities within a storm of particular ARI, as these are all subsumed in the selection of a run-off coefficient.

Table 6.1 provides a summary for the application of flood estimation methods for rural areas in Australia and New Zealand.

Table 6.1: Summary of application of Rational Method for rural areas

<table>
<thead>
<tr>
<th>Region</th>
<th>Time of concentration</th>
<th>Run-off coefficient</th>
<th>Frequency factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>NSW – Eastern</td>
<td>$t_c = 0.76 A^{0.38}$ where: $t_c =$ time of concentration (hours) $A =$ catchment area (km$^2$)</td>
<td>$C_{10}$ from AR&amp;R Vol. 2 (Pilgrim 2007)</td>
<td>Read off map$^{(1)}$ with adjustment for elevation</td>
</tr>
<tr>
<td>NSW – Western (flat catchments)</td>
<td>$t_c = 0.76 A^{0.38}$ where: $t_c =$ time of concentration (hours) $A =$ catchment area (km$^2$)</td>
<td>$C_{10}$ from AR&amp;R Vol. 2 (Pilgrim 2007) with adjustment for area</td>
<td>Read off map$^{(1)}$ with adjustment for elevation</td>
</tr>
<tr>
<td>NSW – Western (hilly catchments)</td>
<td>Regional flood frequency method applicable</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Victoria$^{(2)}$</td>
<td>$t_c = 0.76 A^{0.38}$ where: $t_c =$ time of concentration (hours) $A =$ catchment area (km$^2$)</td>
<td>$C_{10}$ ($or$ $P_{10}$) from AR&amp;R Vol. 2 (Pilgrim 2007)</td>
<td>Tabulated values</td>
</tr>
<tr>
<td>Queensland$^{(3)}$</td>
<td>$t_c = \frac{F \times L}{A^{0.1} \times S^{0.2}}$</td>
<td>$C_{50}$ determined from summation of rainfall intensity, topography, storage and ground cover factors</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Western Australia</td>
<td>$t_c = \Delta \delta$ where the factors $\Delta$ and $\delta$ are related to one of 7 zones</td>
<td>$C_{10} = \phi 10^{0.86} (LS_{0})^\gamma$ where the factors $\phi \in \Phi$ and $\gamma$ are related to one of 7 zones</td>
<td>$C_\gamma/C_{10}$ factors are based on the percentage of cleared land</td>
</tr>
<tr>
<td>South Australia – Eastern</td>
<td>$t_c = 0.5 A^{0.65}$ where: $t_c =$ time of concentration (hours) $A =$ catchment area (km$^2$)</td>
<td>Predetermined values</td>
<td>Not applicable</td>
</tr>
<tr>
<td>South Australia – Northern and Western</td>
<td>$t_c = \frac{F \times L}{A^{0.1} \times S^{0.2}}$</td>
<td>Values dependent upon slope</td>
<td>Not applicable</td>
</tr>
<tr>
<td>Tasmania</td>
<td>$t_c = \frac{F \times L}{A^{0.1} \times S^{0.2}}$ for rural catchments (less than 20 ha)</td>
<td>$C = 0.9$ for paved areas $C = 0.35$ for rural catchments (common to apply a sensitivity check to test robustness of results)</td>
<td>Not used</td>
</tr>
<tr>
<td>ACT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Northern Territory$^{(4)}$</td>
<td>$t_c = \frac{F \times L}{A^{0.1} \times S^{0.2}}$</td>
<td>Approximate values given</td>
<td>Not applicable</td>
</tr>
</tbody>
</table>
6.6.2 Time of Concentration

In the Rational Method, the time of concentration \( t_c \) for a catchment is defined as either:

- the time taken for water to flow from the most time-remote point on the catchment to the outlet or point of interest, or
- the time taken from the start of rainfall until all of the catchment is simultaneously contributing to flow at the outlet or point of interest.

The significance of the time of concentration is that peak outflow will always result when the entire catchment is contributing flow from rainfall on the catchment (excluding partial area effects, see Section 6.6.4 – Partial Area Effects). The most intense rainfall that contributes to the outflow will be that with duration equal to the time of concentration.

Therefore, \( t_c \) is the duration used to select the design rainfall intensity from the IFD table generated in Section 6.3 – Rainfall.

The time of concentration is generally made up of three components:

- overland flow time across natural surfaces until flow concentration occurs forming a stream
- time of flow in channels (natural and artificial)
- time of flow in pipes (if applicable).

The type of flow will vary throughout the catchment, although once channelised, overland flow conditions do not normally recur. Overland flow to channel flow and pipe flow back to channel flow can be expected to occur. There may also be overland or channel flow parallel with pipe flow at full capacity. Several flow paths may need to be examined to determine which is the longest or most critical in terms of design flows.
The absolute minimum time of concentration to be used in design for catchments is 5 minutes as prescribed in AR&R Vol. 1 (Pilgrim 2001).

In designing culverts for road crossings, the time of concentration used should allow for future development of the upstream catchment (see Section 6.5 – Catchments).

In several of the Rational Method procedures outlined in Table 6.1, formulae are specified for estimating the time of concentration. The specified formula must be used with the particular procedure. Where a complete procedure based on observed data is not available, the Bransby-Williams formula or the Ramser-Kirpich formula can be used. The Bransby-Williams formula is more commonly used in Australia, whereas the Ramser-Kirpich formula is used in New Zealand, possibly due to the catchments characteristics being typically steeper and the stream flows more concentrated. Both methods can be used and then a judgement applied to adopt the most appropriate method.

For all catchment sizes within the limits of the Rational Method, the time of concentration is commonly determined using one of the relevant formulae outlined in Table 6.1. These formulae which include overland flow and channel flow conform to the accepted practices in AR&R Vol. 1 (Pilgrim 2001).

The Bransby-Williams formula is (Equation 4):

\[ t_c = \frac{F \times L}{A^{0.1} \times S_{eq}^{0.2}} \]

where

- \( t_c \) = Time of concentration (min)
- \( F \) = A conversion factor. \( F = 58.5 \) when \( A \) is in km\(^2\) and 92.7 when \( A \) is hectares (ha)
- \( L \) = Length of mainstream(km) from the outlet to the catchment divide
- \( A \) = Area of catchment (either km\(^2\) or hectares)
- \( S_{eq} \) = Equal area slope (m/km) as defined in Figure 6.4

If the catchment has several possible flows paths upstream of the site, each path will have to be assessed to determine the path with the longest time of concentration.
6.6.3 Run-off Coefficient

The run-off coefficient relates the volume of water that is discharged from a catchment to the rain falling over the catchment.

The run-off coefficient includes effects of catchment characteristics, infiltration and other losses as well as rainfall intensity. The run-off coefficient $C_Y$, as used in the Rational Method, is a function of the design ARI ($Y$ in years) and depends on many features of the catchment area including:

- rainfall intensity
- relief or slope of catchment
- storage or other detention characteristics
- ground characteristics such as vegetation cover, soil type, and impervious areas.

The run-off coefficient can be determined by rearranging the Rational Method formula, shown in Equation 5 and using the catchment data for the rainfall and stream flows, from stream gauges:

$$ C_Y = \frac{360 \times Q_Y}{A \times I_{tc}^Y} \tag{5} $$

where

- $C_Y$ = Coefficient of runoff from a storm with an ARI of $Y$ years
- $Q_Y$ = Discharge from a storm of $Y$ years (m$^3$/s)
- $A$ = Catchment area (ha)
- $I_{tc}^Y$ = Storm intensity (mm/h) for a storm duration $t_c$ and an ARI of $Y$ years

Source: Derived from Pilgrim (2001).
The data is often analysed by different methods and a summary of the methods used across Australia and New Zealand is shown in Table 6.1.

Coefficients of run-off can be determined for storms of other ARI by use of frequency factors ($F_Y$) as shown in Table 6.2 and Equation 6:

Table 6.2: Frequency factors ($F_Y$) for the coefficient of run-off for use in the Rational Method

<table>
<thead>
<tr>
<th>$Y$ (years)</th>
<th>1</th>
<th>2</th>
<th>5</th>
<th>10</th>
<th>20</th>
<th>50</th>
<th>100</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_Y$</td>
<td>0.80</td>
<td>0.85</td>
<td>0.95</td>
<td>1.00</td>
<td>1.05</td>
<td>1.15</td>
<td>1.20</td>
</tr>
</tbody>
</table>

$$C_Y = F_Y \times C_{10}$$

where

- $Y = \text{ARI in years}$
- $F_Y = \text{Frequency factor from Table 6.2}$
- $C_{10} = \text{10 year ARI run-off coefficient}$

See Appendix F for further information regarding the estimation of the coefficient of run-off in Queensland.

6.6.4 Partial Area Effects

In general, the appropriate time of concentration ($t_c$) for calculation of the peak flow at any point is the longest time of travel to that point. However, in some situations, the maximum flow may occur when only part of the upstream catchment is contributing. Thus the product of a lesser $C \times A$ and a higher $t_{c_Y}$ (resulting from a lower $t_c$) may produce a greater peak discharge than that if the whole upstream catchment is considered. This is known as the ‘partial area effect’.

Partial area effects usually result from the most upstream portion of the overall catchment (typically relatively small) but having a $t_c$ considerably longer than the rest of the catchment. This can result from differences within a catchment of surface slope, shape, or ground covering type.

The designer needs to be aware of the possibility of the partial area effect and to check as necessary to ensure that an appropriate peak discharge is obtained.

The occurrence of the Partial Area Effect in the rural environment is not common, but designers should look for catchments that display the characteristics that may allow the partial area effect to occur. These catchments need to be checked to ensure that the peak discharge for the catchment is correctly determined.

Figure 6.5 shows two examples of rural catchments that may experience the partial area effect.

**Figure 6.5: Examples of catchments that may be subject to partial area effects**

Source: Adapted from DTMR (2010b).
In Catchment (I), Figure 6.5, the whole catchment is assessed with all parameters for the time of concentration and run-off calculations determined normally. The catchment is then divided, where the catchment changes from wide to narrow, into two portions. The lower portion is then assessed as if it was the whole catchment, that is, $A$ will be smaller, and $S_e$ will be based on the channel bed from the catchment outlet to the dividing line between the portions and so on. In this case $t_c$ will be shorter and therefore $I_Y$ will be higher. This is undertaken as the narrow portion of the catchment significantly increases the time of concentration, with only a relatively small area contributing to the run-off. The higher discharge of the two assessments is deemed to be the peak discharge.

In Catchment (II), Figure 6.5, the whole catchment is again assessed with all parameters for the time of concentration and run-off calculations determined normally. The catchment is then divided, where the catchment slope changes from steep to flat. The lower portion is then assessed as if it was the whole catchment. In this case, $S_e$ will be much higher and will have a bigger impact on reducing $t_c$ and so intensity will therefore be higher. This is due to the steeper section of the catchment having a relatively shorter time of concentration, hence higher $I_Y$ with the flatter section having a relatively longer $t_c$ which reduces the $I_Y$. Again, the higher discharge of the two assessments is deemed to be the peak discharge.

### 6.6.5 Progressive Catchments

A situation that often occurs in rural environments is where a stream crosses a road several times and receives flows from different catchments along its course, which add to the stream flow and this is known as ‘Progressive Catchments’. In concept, this is similar to the partial area effects a catchment may experience.

The Rational Method can only estimate the run-off at a point, usually the outlet of the catchment, or the site for a culvert. Therefore, disregarding any upstream crossing, the variables $A$ and $C$ used in the Rational Method (Equation 3) must describe the whole upstream catchment and the variables $S_e/t_c$ must be based on the flow path from the site to the top of the catchment. If there are several flow paths or streams, time of concentration calculations will need to be undertaken on each path to determine the critical duration.

To demonstrate the approach, using the catchments shown in Figure 6.6, the peak discharge at Point 5 would be estimated from assessing catchment C normally. To estimate the peak discharge at Point 3, variables $A$ and $C$ must cover both Catchments B and C while variables $S_e/t_c$ would be based on the critical duration determined from paths (3, 4, 10), (3, 4, 5, 6, 7) or (3, 4, 5, 6, 8) and the resulting area contributing for that critical duration. To estimate the peak discharge at Point 1, variables $A$ and $C$ need to cover Catchments A, B and C and variables $S_e/t_c$ would be based on the critical duration determined from paths (1, 2, 9), (1, 2, 3, 4, 10), (1, 2, 3, 4, 5, 6, 7) or (1, 2, 3, 4, 5, 6, 8) and the resulting are contributing for that critical duration.
Technically, any upstream crossing could act as a detention device. That is, a device that reduces the peak flow but lengthens the time flow occurs at that point, though this effect would usually be small. This would have an impact on the flow at some downstream point, as determined by using the Rational Method as explained in this section. The impact would typically be a reduction in the peak discharge. Therefore the approach adopted here is considered conservative.

If the land use or components of C vary between the catchments or if a more accurate estimate of run-off is required, then use of an appropriate numerical run-off-routing model is needed and assistance from a suitably qualified specialist is required.

### 6.7 Urban Hydrology

#### 6.7.1 Rational Method

The Rational Method provides a simple methodology for assessing the design peak flow rate to enable the determination of the sizes of drainage systems within an urban catchment area less than 100 hectares (1 km²). However, the Rational Method in urban environments has significant limitations, and it is the task of the designer to be familiar with these limitations and to know when an alternative methodology is required.

The choice of hydrologic method must be appropriate to the type of catchment and the required degree of accuracy. The Rational Method should not be used whenever a full design hydrograph is required for flood mapping or to assess flood storage issues. Instead the more reliable run-off-routing techniques presented in publications such as *AR&R Vol. 1 and Vol. 2* (Pilgrim 2001, 2007) should be adopted.
The Rational Method formula for flow estimation (see Equation 3 in Section 6.6.1 – Rational Method) can be reconfigured for a flow rate in litres per second (L/s) to reflect the peak discharge in small urban catchments, as shown in Equation 7:

\[
Q_y = \frac{C_y \times \gamma I_{tc} \times A}{360}
\]

where

- \(Q_y\) = Peak discharge for a storm with an ARI of \(Y\) years (m³/s)
- \(C_y\) = Run-off coefficient for a storm with an ARI of \(Y\) years (dimensionless)
- \(\gamma I_{tc}\) = Average rainfall intensity for a storm of \(t_c\) minutes duration (time of concentration) and an ARI of \(Y\) years (mm/h)
- \(A\) = Catchment area (ha)

### 6.7.2 Time of Concentration

Most urban catchments are largely man-made and hence the flow paths can generally be readily determined. However, urban hydrology is much more complicated than rural hydrology and therefore designers need to undertake this work carefully with regular reviews and checks. Designers should also note that care is required when the larger events cause overland flow to bypass underground pipe systems – as the \(t_c\) may be smaller.

For further description of the time of concentration in the Rational Method, see Section 6.6.2 – Time of Concentration.

In designing culverts for road crossings, the time of concentration used should allow for future development of the upstream catchment. This development could be the changing of land use due to farming or urbanisation near or within a town or city. Consider the example illustrated in Figure 6.7.

**Figure 6.7:** Catchment development

Source: DTMR (2010b).
If the time of concentration to point A is calculated in the Existing Catchment, it will be made up of:

- a considerable length of overland and channel flow
- a short length of flow in pipes.

In the case with Possible Catchment Development where the drainage system in the catchment upstream of the road is improved, the overland flow time will be reduced and the time of concentration to A also reduced. With this shorter time of concentration the design rainfall intensity will increase and therefore increase the amount of run-off generated. Designers need to check the full range of possible cases.

In determining the time of concentration, the designer should adopt the appropriate catchment conditions in accordance with the required analysis. Ultimate flow conditions should be based on a fully developed catchment in accordance with the allowable land use shown in the relevant local authority plans, or as directed by the local authority.

It should be noted that the time of concentration as used in the Rational Method is not the same as the critical storm duration or time to peak as determined from run-off-routing models. It is therefore inappropriate to adopt the critical storm duration determined from a run-off-routing model and apply it as the time of concentration for a Rational Method analysis.

To apply the Rational Method in an appropriate and consistent manner, four different methodologies for determination of the time of concentration are presented below for different types of drainage catchments:

1. Predominantly piped or channelised urban catchments less than 100 ha with the top of the catchment being urbanised
2. Predominantly piped or channelised urban catchments less than 100 ha with the top of the catchment being bushland or a grassed park
3. Bushland catchments too small to allow the formation of a creek with defined bed and banks
4. Urban creeks with a catchment area less than 100 ha.
1. Predominantly piped or channelised urban catchments less than 100 ha with the top of the catchment being urbanised

Components of time of concentration:
- Standard inlet time (see Table 6.3). If the actual length of kerb/channel travel is unusually long, then an additional travel time must be added to the standard inlet time (next dot point below). If a gully/field inlet does not exist near the top of the catchment, determine the initial travel time to the start of the kerb/channel, and then add the travel time along the kerb/channel. The standard inlet time includes the travel time along a typical length of kerb/channel from near the top of the catchment to the first pipe or channel inlet.
- Pipe flow time using actual flow velocities determined from a pipe network analysis or Manning’s Equation (see Equation 8). Alternatively, if the pipe flow time is not critical, an average pipe flow velocity of 2 m/s and 3 m/s may be adopted for low gradient and medium to steep gradient pipelines respectively.
- Creek and/or channel flow time using actual flow velocity determined from numerical modelling or Manning’s Equation. Alternatively, if the expected travel time in the creek is not critical, an average flow velocity of 1.5 m/s may be adopted (not applicable to constructed channels).

2. Predominantly piped or channelised urban catchments less than 100 ha with the top of the catchment being bushland or a grassed park.

Components of time of concentration:
- Estimate the length of ‘sheet’ run-off at the top of the catchment using Table 6.5 or field observations, then estimate the sheet flow travel time.
- Determine the remaining distance of assumed concentrated overland flow from the end of the ‘sheet’ run-off to the nearest kerb, pipe inlet, open channel or creek. Then determine the travel time for this concentrated overland flow based on the calculated flow velocity.
- Pipe and/or channel flow as per Case (1) above.

3. Bushland catchments too small to allow the formation of a creek with defined bed and banks.

Time of concentration is determined as for (2) above.

4. Urban creeks with a catchment area less than 100 ha.

Time of concentration for an urban catchment containing a watercourse with defined bed and banks may be determined as for rural catchments but only if the following conditions apply:
- channel storage along the watercourse, for the catchment condition being analysed, is not significantly reduced from the natural (i.e. pre-urbanisation) conditions
- less than 20% of the catchment drains to a pipe network.

If the above two conditions do not apply, then the time of concentration should be based on the procedures outlined in (1) or (2) above as appropriate for the catchment conditions.

**Overland flow**

Overland flow at the top of a catchment will initially travel as sheet flow, after which it will move down the catchment as minor concentrated flow. Travel times for the sheet flow and concentrated flow components need to be determined separately.
The sheet flow travel time is defined as the travel time from the top of a catchment to the point where stormwater run-off begins to concentrate against fences, walls, gardens, or is intercepted by a minor channel, gully or piped drainage. This concentration of flow may also occur in the middle of vegetated areas as the stormwater concentrates in minor drainage depressions.

The time required for water to flow over a homogeneous surface such as lawns and gardens is a function of the surface roughness and slope. There are a number of methods available for the determination of sheet flow travel times and a local government may direct which of these methods should be applied. Two such methods are presented in this section.

Irrespective of which method of calculation is adopted, it is the designer’s responsibility to determine the effective length of this sheet flow.

In urban areas, the length of overland sheet flow will typically be 20 m to 50 m, with 50 m being the recommended maximum. In rural residential areas the length of overland sheet flow should be limited to 200 m; however the actual length is typically between 50 m and 200 m where the flow will be concentrated in small rills, channels, or tracks.

**Standard inlet time**

Use of standard inlet times for developed catchments is recommended because of the uncertainty related to the calculation of time of overland flow. The standard inlet time is defined as the travel time from the top of the catchment to a location where the first gully or field inlet would normally be expected as depicted in Figure 6.8.

Suggested standard inlet times are presented in Table 6.3. These inlet times are considered appropriate for traditional (i.e. non water sensitive urban design (WSUD)) low density residential areas where the top of the catchment is low density residential, but not a park or bushland.

**Figure 6.8: Application of standard inlet time**
Table 6.3: Suggested standard inlet times

<table>
<thead>
<tr>
<th>Location</th>
<th>Inlet time (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road surfaces and paved areas</td>
<td>5</td>
</tr>
<tr>
<td>Urban residential areas where average slope of land at top of catchment is greater than 15%</td>
<td>5</td>
</tr>
<tr>
<td>Urban residential areas where average slope of land at top of catchment is greater than 10% and up to 15%</td>
<td>8</td>
</tr>
<tr>
<td>Urban residential areas where average slope of land at top of catchment is greater than 6% and up to 10%</td>
<td>10</td>
</tr>
<tr>
<td>Urban residential areas where average slope of land at top of catchment is greater than 3% and up to 6%</td>
<td>13</td>
</tr>
<tr>
<td>Urban residential areas where average slope of land at top of catchment is up to 3%</td>
<td>15</td>
</tr>
</tbody>
</table>

*Note: The average slopes referred to are the slopes along the predominant flow path for the catchment in its developed state. Source: DTMR (2010b).*

If the top of the catchment consists of high density residential development, then the local government should be consulted for inlet times appropriate for the catchment. In such cases it is recommended that the standard inlet time should not exceed 10 minutes unless demonstrated otherwise by the designer.

If the hydrologic analysis is being performed on a development located at the top of the catchment, the actual inlet time is likely to be less than the minimum time suggested in Table 6.3. In these cases the minimum inlet time should be adopted.

If the first gully or field inlet is located further down the catchment slope than would normally be expected, then the standard inlet time should only account for the travel time down to the location where the first gully or field inlet would normally have been located.

If the urban drainage system does not incorporate pipe drainage (i.e. no gully or field inlet exists) then the standard inlet time should extend down the catchment to a location where an inlet would normally be located in a traditional pipe drainage system.

A standard inlet time should not be adopted in sub-catchments where detailed overland flow and kerb/channel flow calculations are justified. However, a local government may require that the use of standard inlet times shall not apply within their area and may recommend alternative methods.

In certain circumstances, the use of standard inlet times may result in times of concentration being unacceptably short for the catchment under consideration, such as airports, or large flat car parks. In these cases the designer should utilise Friend’s Equation (Equation 10) to determine the time of initial overland flow (see *Overland Flow*, below). Inlet times calculated by these methods should only be adopted for design if the sheet flow length criteria discussed below are met and if due consideration is given to the type and continuity of the surface where overland flow is occurring.

Notwithstanding the above, it is recommended that a maximum inlet time of 20 minutes be adopted for urban and residential catchments, including playing fields and park areas.

*Roof to main system connection*

In cases where use of a standard inlet time is not considered appropriate, the roof to main system flow travel times as shown in Table 6.4 are suggested.
Table 6.4: Suggested roof drainage system travel times

<table>
<thead>
<tr>
<th>Development category</th>
<th>Time to point ‘A’ (minutes)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural residential, residential low density. For the roof, downpipes and pipe connection system from the building to the kerb and/or channel or a rear-of-allotment drainage system, Figure 6.9(a).</td>
<td>5</td>
</tr>
<tr>
<td>Residential medium and high density, commercial, industrial and central business. For the roof and downpipe collection pipe to the connection point to the internal allotment drainage system abutting the building Figure 6.9(b).</td>
<td>5</td>
</tr>
</tbody>
</table>

*Note: The flow time from Point A (Figure 6.9 (a) & (b)) through the internal allotment pipe system to the kerb and/or channel, street underground system or rear of allotment system for the more intense developments noted should be calculated separately.*

*Source: DTMR (2010b).*

Figure 6.9: Examples of roof drainage systems for residential and industrial allotments

*Note: Point A is referred to in Table 6.5.*

*Source: DTMR (2010b).*

**Design steps**

To determine the overland flow travel time the following steps should be applied:

1. Where practical, inspect the catchment to determine the length of initial overland sheet flow, or for new developments measure the length of overland flow from the design plans.

2. Where it is not practical to inspect the catchment, determine the likely length of overland sheet flow based on Table 6.5.

3. Determine the sheet flow travel time one of the overland sheet flow calculation methods described below.

4. Determine or measure the remaining distance of assumed concentrated overland flow from the end of the adopted sheet flow to the nearest kerb, channel, or pipe inlet.
5. Determine the concentrated flow travel time using either Manning’s Equation or for preliminary design purposes, Figure 6.10.

Manning’s Equation is (Equation 8):

\[ V = \frac{R^{2/3}S^{1/2}}{n} \]

where

- \( V \) = Velocity in the channel (m/s)
- \( R \) = Hydraulic radius (m)
- \( S \) = Slope of energy line or hydraulic gradeline (m/m)
- \( n \) = Manning’s roughness coefficient

and the hydraulic radius is determined by (Equation 9):

\[ R = \frac{A}{P} \]

where

- \( A \) = Flow area (m²)
- \( P \) = Wetted perimeter (m)

Figure 6.10: Example of kerb and channel flow time using Manning’s Equation

Source: DTMR (2010b).
For simplicity, several standard times can be adopted for overland flow:

- A two minute minimum flow time is often adopted for the flow to travel from the crown of a road to the gutter.
- On wider carriages, of say 10 m and above, a three minute travel time or more, may be more appropriate.
- Typically a minimum travel time of five minutes is adopted for any drainage element to run-off into an underground or surface system. As a guide, carriageway elements up to 200 m in length, regardless of slope, fall within this minimum.

### Table 6.5: Recommended maximum length of overland sheet flow

<table>
<thead>
<tr>
<th>Surface condition</th>
<th>Assumed maximum flow length (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steep (say &gt; 10%) grassland (Horton’s $n = 0.045$)</td>
<td>20</td>
</tr>
<tr>
<td>Steep (say &gt; 10%) bushland (Horton’s $n = 0.035$)</td>
<td>50</td>
</tr>
<tr>
<td>Medium gradient (approx. 5%) bushland or grassland</td>
<td>100</td>
</tr>
<tr>
<td>Flat (0–1%) bushland or grassland</td>
<td>200</td>
</tr>
</tbody>
</table>

### Overland sheet flow calculations

Three different formulae are detailed here for overland flow calculations. Designers can choose the method appropriate or approved for the work being undertaken.

Formula attributed to Friend, may be used for the determination of overland sheet flow times. This was derived from previous work in the form of a nomograph, as shown in Figure 6.11, for shallow sheet flow over a plane surface.

Friend’s Equation is (Equation 10):

$$ t = \frac{107 n L^{2/3}}{S^{1/2}} $$

where

- $t$ = Overland sheet flow travel time (mins)
- $L$ = Overland sheet path length (m)
- $n$ = Horton’s roughness value for the surface
- $S$ = Slope of surface (%)

Surface roughness values for Horton’s $n$ are similar but not identical to Manning’s $n$ values. See Table 6.6 for values for Horton’s $n$. 
Figure 6.11: Overland sheet flow times – shallow sheet flow only

Table 6.6: Horton’s roughness values

<table>
<thead>
<tr>
<th>Surface condition</th>
<th>$n^{(1)}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete or asphalt</td>
<td>0.010–0.013</td>
</tr>
<tr>
<td>Bare sand</td>
<td>0.010–0.016</td>
</tr>
<tr>
<td>Gravelled surface</td>
<td>0.012–0.030</td>
</tr>
<tr>
<td>Bare clay – loam soil (eroded)</td>
<td>0.012–0.033</td>
</tr>
<tr>
<td>Sparse vegetation</td>
<td>0.053–0.130</td>
</tr>
<tr>
<td>Short grass paddock</td>
<td>0.100–0.200</td>
</tr>
<tr>
<td>Lawns</td>
<td>0.170–0.480</td>
</tr>
</tbody>
</table>

1 Horton’s roughness coefficient, ‘$n$’ is similar but not the same as the Manning’s ‘$n$’


The second formula or method attributed to Oakden (1977) is (Equation 11):

$$T_c = 500nL^{2/3}S_{f}^{-1/3}$$  \(11\)

where

- $T_c$ = Time of concentration (seconds)
- $L$ = Length (m)
- $S_{f}$ = Slope (m/m)
- $n$ = Manning’s roughness value
The third method is attributed to Ragan and Duru (1972) and often referred to as the kinematic wave equation:

\[ T_o = 6.94 \frac{(Ln)^{0.6}}{I^{0.45}n^{0.3}} \]  

where

\[ T_o = \text{Time of overland flow (minutes)} \]
\[ L = \text{Overland flow path length (m)} \]
\[ n = \text{Manning’s roughness value} \]
\[ I = \text{Rainfall intensity from the design ARI event (mm/h)} \]
\[ S = \text{Slope of overland flow path (m/m)} \]

Using Equation 12 requires an iterative approach because the rainfall intensity alters as the time of concentration alters. The kinematic wave equation can be applied quite simply, without iteration, if a suitable design aid is prepared by plotting values of \( T_o I^{0.4} \) against duration.

The formula can be applied to multiple flow segments. Where the drained area is composed of different surfaces and slopes, it should be divided into segments or planes, and the calculated travel times for these combined. This causes complications in selecting the intensity to be used in the formula – it should be the average intensity over the total time involved.

It is incorrect to add values of \( T_o I^{0.4} \) for each of the segments, as Equation 12 is based on the assumption that no flow is entering the flow segments from upstream. The following method of combining segments is outlined in AR&R Vol. 1 (Pilgrim 2001, S Mills in Book 8):

For two segments A and B, the total overland flow time is (Equation 13):

\[ T_{\text{total}} = T_{(L_A)} + T_{B(L_A + L_B)} - T_{B(L_A)} \]  

where

\[ L_A = \text{Length of flow for segment A} \]
\[ L_B = \text{Length of flow for segment B} \]
\[ T_{(L_A)} = \text{Time for flow across segment A} \]
\[ T_{B(L_A + L_B)} = \text{Time for flow across the total length of segments A and B using the slope and roughness of segment B} \]
\[ T_{B(L_A)} = \text{Time for flow across a virtual segment along a length equal to segment A using the slope and roughness of segment B} \]
For each additional segment, the following value should be added (Equation 14):

\[ T \times (L_{\text{total}}) - T \times (L_{\text{total}} - L_X) \]

where

| X | The segment name and \( L_{\text{total}} \) is the total length of flow, including the current segment \( X \). Use the slope and roughness of segment \( X \). |

Calculations for single flow and multiple flow segments utilising the kinematic wave equation are presented in the Section 6.9 – Worked Example (Urban).

**Kerb/gutter flow**

The flow time in gutters is a component of the time of concentration for a drainage area to an inlet. To find the gutter flow component of the time of concentration, a method for estimating the average velocity in a gutter is needed. The velocity in a gutter varies with the flow rate and the flow rate varies with the distance along the gutter, i.e. both the velocity and flow rate in a gutter are spatially varied. The time of flow can be estimated by use of an average velocity obtained by integration of the Manning’s equation for the gutter section with respect to time (Brown et al. 2009).

Table 6.7 and Figure 6.12 can be used to determine the average velocity in triangular gutter sections. In Table 6.7, \( T_1 \) and \( T_2 \) are the flow spreads at the upstream and downstream ends of the gutter section respectively. \( T_a \) is the spread at the average velocity. Figure 6.12 is a nomograph to solve Equation 15 (repeated from AGRD Part 5 – Section 5.5.1) for the velocity in a triangular channel with known cross-slope, gutter slope and spread (Brown et al. 2009):

**Table 6.7: Spread at average velocity in a reach of triangular gutter**

<table>
<thead>
<tr>
<th>( T_1/T_2 )</th>
<th>0</th>
<th>0.1</th>
<th>0.2</th>
<th>0.3</th>
<th>0.4</th>
<th>0.5</th>
<th>0.6</th>
<th>0.7</th>
<th>0.8</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_a/T_2 )</td>
<td>0.65</td>
<td>0.66</td>
<td>0.68</td>
<td>0.70</td>
<td>0.74</td>
<td>0.77</td>
<td>0.82</td>
<td>0.86</td>
<td>0.90</td>
</tr>
</tbody>
</table>

**Source:** Brown et al. (2009).

\[ V = \frac{K_c S_x^{2/3} S_L^{1/2} T^{2/3}}{n} \]

where

| \( K_c \) | Conversion factor = 0.376 |
| \( S_x \) | Crossfall (m/m) |
| \( S_L \) | Longitudinal slope (m/m) |
| \( T \) | Width of spread (m) |
| \( n \) | Manning’s roughness coefficient |

or, for preliminary design purposes, use Figure 6.10.
Pipe flow

Wherever practical, pipe travel times should be based on calculated pipe velocities either using a pipe flow chart (refer to AGRD Part 5A – Appendix B), uniform flow calculations using Manning’s Equation, or results from a calibrated numerical drainage model.

For preliminary design purposes, pipe flow travel time can be estimated using Figure 6.10. Alternatively, if the travel time within the pipe is small compared to the overall time of concentration, then an average pipe velocity of 2 m/s and 3 m/s may be adopted for low gradient and medium to steep gradient pipelines respectively.
**Channel flow**

The time stormwater takes to flow along an open channel may be determined by dividing the length of the channel by the average velocity of the flow.

The average velocity of the flow is calculated using the hydraulic characteristics of the open channel. Manning's Equation (Equation 8) repeated here, is suitable for this purpose:

\[
V = \frac{R^{2/3}S^{1/2}}{n}
\]

where

- \( V \) = Velocity in the channel (m/s)
- \( R \) = Hydraulic radius flowing full (m) (see Equation 9)
- \( S \) = Slope of energy line or hydraulic gradeline (m/m)
- \( n \) = Manning's roughness coefficient for the channel

Where an open channel has varying roughness or depth across its width it may be necessary to split the channel into sections and determine the average flow velocity in each section, to determine the overall flow time.

**Grass swales**

Flow travel times along grassed swales can vary significantly depending on flow depth and vegetation. Swale roughness, \( n \) should be determined from the vegetation retardance charts presented in *AGRD Part 5B* – Appendix A.

**Estimate of kerb, pipe and channel flow time**

For checking or preliminary design purposes, an overall flow time can be determined from Figure 6.13. The chart may be used directly to determine approximate travel times along a range of rigid channel types and with the application of multiplier \( \Delta \), for a range of loose-boundary channel forms.

Flows can reach drains/channels via roof to gutter conduits, overland flow paths or along gutters. In many cases, flows travel along two or three consecutive paths, and it is necessary to calculate a total travel time.
Figure 6.13: Flow travel time in pipes and channels

Notes:
Flow travel time (approximate) may be obtained directly from this chart for:
- kerb-and-gutter channels
- stormwater pipes
- allotment channels of all types (surface and underground)
- drainage easement channels (surface and underground).
Multiplier $\Delta$ should be applied to values obtained from the chart as per:
- grassed swales, well maintained and without driveway crossings – $\Delta = 4$
- blade-cut earth table drains, well maintained and no driveway crossings – $\Delta = 2$
- natural channels – $\Delta = 3$.
Source: DTMR (2010b).
6.7.3 Run-off Coefficient

The run-off coefficient relates the volume of water that is discharged from a catchment to the rain falling over the catchment. The value is not constant, but varies with rainfall intensity and the proportion of impervious areas. Different approaches are applied to urban and rural situations. As catchment size increases, it can be beneficial to determine the sensitivity of the analysis to variations in the run-off coefficient.

It is common to prescribe run-off coefficients (e.g. 0.9 for urban areas and 0.35 for rural areas). However, it is worthwhile checking the sensitivity of the run-off volumes to variations in run-off coefficients and how this will affect the drainage design.

The following is an extract from *AR&R* Vol. 1 (Pilgrim 2001, Book 8):

*Run-off coefficient C can be interpreted in different ways:*

- as a ratio between run-off and rainfall volumes
- as the ratio of their peak rates
- as the ratio of the run-off to rainfall frequency curves.

In this last, ‘probabilistic’ interpretation, the value of C does not relate to a particular storm. This concept is discussed in detail in Book 4 of *AR&R* Vol. 1 (Pilgrim 2001). It covers the whole range of possible events, involving different combinations of rainfalls and antecedent conditions. Values have been derived for some medium-sized, gauged urban catchments in Australian Capital Cities by Aitken (1975), Pilgrim (1982) (both cited in *AR&R* Vol. 1 Pilgrim 2001) and others.

Many relationships have been proposed relating run-off coefficients to factors such as land use, surface type, slope and rainfall intensity. The one given in Figure 6.14 is a composite relationship reflecting experience of drainage authorities and evidence from the few gauged urban catchments with suitable lengths of record. It should be used in preference to the run-off coefficient relationships given in previous editions of this publication.

Figure 6.14 relates the coefficient for a 10 year ARI, C_{10}, to the pervious and impervious fractions of the catchment, and to its rainfall climate, expressed through the 10 year ARI, 1 hour duration rainfall intensity, 10I_{1}.

---

4 Figure and equation references have aligned with this Guide.
The upper line represents conditions for areas where $10I_1$ is 70 mm/h or greater, and the lower one is for areas where $10I_1$ is 25 mm/h or lower. For areas where $10I_1$ is between 25 and 70 mm/h, a line can be interpolated using the equations:

\[
C_{10} = 0.9 x f + C_{10}^1 (1 - f)
\]

and

\[
C_{10}^1 = 0.1 + \frac{(0.7 - 0.1)(10I_1 - 25)}{70 - 25}
\]

where

- $C_{10}$ = The 10 year ARI run-off coefficient
- $C_{10}^1$ = The pervious area run-off coefficient
- $f$ = The fraction impervious (0.0 to 1.0)

Pervious area run-off coefficients range from 0.1 to 0.7, corresponding to the respective $10I_1$ limits of 25 and 70 mm/h. These are likely to differ from coefficients derived in regional procedures for rural design flow estimation (such as those in Book 4, Section 1 (Pilgrim 2001)), due to different interpretations of the Rational Method, the different scales of catchment size and the different times of concentration.

For average recurrence intervals other than 10 years, the $C_{10}$ value is multiplied by a frequency factor from Table 6.2:

\[
C_Y = F_Y C_{10}
\]

where

- $C_Y$ = Run-off coefficient for a storm with an ARI of $Y$ years (dimensionless)
\[ F_Y = \text{Frequency factor (from Table 6.2)} \]

\[ C_{10} = \text{The 10 year ARI run-off coefficient} \]

*Where run-off coefficients calculated from the above equations exceed 1.0, they should be arbitrarily set equal to 1.0.*

Note that no allowance is made for slope and soil type. While it seems logical that they would affect run-off coefficients, there is little firm evidence to confirm this. To some extent, the effect of slope is incorporated in the time of concentration estimate. As for soil type, designers may make adjustments based on local evidence, if it is available.

The above relationships can be applied both to areas which are essentially homogeneous, and to those where pervious and impervious portions are intermixed. Where a catchment consists of portions which are significantly different, they should be separated and different C values applied.

For an example of the use of the design pivot line refer to *AGRD Part 5A* – Section 5.7.2.

For simplicity, Equation 19 can be used to estimate the percentage of impervious area for residential areas. The equation is valid up to a residential density of 20 dwellings per hectare and caution should be used in applying Equation 19 to developments with greater densities:

\[
f = 3 \times RD - 5
\]

where

\[ RD = \text{The number of residences per hectare (ha)} \]

See Appendix E for further information regarding the estimation of the coefficient of run-off in Queensland.

**Future development of urban catchments**

The run-off coefficient must account for the future development of the catchment as depicted in the planning scheme or zoning maps for the relevant local government, but should not be less than the value determined for the catchment under existing conditions.

Table 6.8 provides guidance in the selection of fraction impervious values for various development categories.

In making the decision on whether or not to allow for future development, the disruption to traffic when the additional waterways are constructed in the future must be considered. Other considerations include the requirements by some local governments to not allow any increase in water discharging into drainage structures in the road corridor from development of an upstream catchment.

Detention basins are therefore often specified in the design of the development, particularly in small, urbanised catchments. In this case there is no need to consider the effect of development.

In the case where the detention basin only moderates the run-off from the development, then the parameters of the detention basin design need to be considered in the discharge calculations at the departmental drainage structure.
Table 6.8: Fraction impervious vs development category

<table>
<thead>
<tr>
<th>Development category</th>
<th>Fraction impervious (fi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Central business</td>
<td>0.90–1.00</td>
</tr>
<tr>
<td>Commercial, local business, neighbouring facilities, service industry, general industry, home industry</td>
<td>0.90</td>
</tr>
<tr>
<td>Significant paved areas e.g. roads and car parks</td>
<td>0.90</td>
</tr>
<tr>
<td>Urban residential – high density</td>
<td>0.80–0.90</td>
</tr>
<tr>
<td>Urban residential – low density (including roads)</td>
<td>0.45–0.75</td>
</tr>
<tr>
<td>Urban residential – low density (excluding roads)</td>
<td>0.30–0.50</td>
</tr>
<tr>
<td>Rural residential</td>
<td>0.10–0.30</td>
</tr>
<tr>
<td>Open space and parks, etc.</td>
<td>0.0–0.20</td>
</tr>
</tbody>
</table>

Notes:
The designer should determine the actual fraction impervious for each development. Local governments may specify default values.

Typically for urban residential high density developments:
- townhouse type development $fi = 0.70$
- multi-unit dwellings > 20 dwellings per ha
- high-rise residential development $fi = 0.90$.

In urban residential low density areas $fi$ may vary depending upon road width, allotment size, house size and extent of paths, driveways etc.

Source: Modified from DNRW (2007).

6.7.4 Partial Area Effects

The partial area effect phenomenon outlined in Section 6.6.4 – Partial Area Effects for rural catchments is also applicable for urban catchments. The occurrence of partial area effects in the urban environment is much more common than in the rural environment.

Figure 6.15 shows various examples of urban catchments that may experience partial area effect.

A simplified procedure is based on a comparison between the full area discharge and the partial area peak discharge for the time of concentration of the impervious areas of the critical sub-catchment. Care must be exercised as this procedure can underestimate the peak discharge.

The method involves the use of a time of concentration $t_c$ corresponding to the flow travel time from the most remote, directly connected, impervious area of the catchment to the point under consideration. Thus, the calculated peak discharge is that from the impervious portion of the catchment plus that from the previous part of the catchment which has begun to contribute up to time $t_c$ since the storm began.

Therefore (Equation 20):

\[ CA = C_i A_i + \left( \frac{t_i}{t_c} C_p A_p \right) \]

where

- $C = $ Overall coefficient of run-off with $C_i$ and $C_p$ being the coefficients for the impervious and pervious areas respectively
- $A = $ Overall areas with $A_i$ and $A_p$ being the impervious and pervious areas respectively
\[ t_i = \text{Time of concentration from impervious area (minutes)} \]
\[ t_c = \text{Time of concentration for the catchment (minutes)} \]

Figure 6.15: Examples of urban catchments that may be subject to partial area effects

Source: Adapted from DNRW (2007).
6.8 Worked Example (Rural): Rural Run-off

This example describes the process to determine the stormwater run-off from a simple rural catchment.

The example commences after the catchment area has been determined and catchment data has been gathered.

The task for this example is, given the catchment data below, estimate the peak stormwater discharge at Point A, the site for a proposed culvert, for the ARI’s 50, 20 and 10 year flood events.

**Catchment data**
- area = 6 km²
- catchment, Figure 6.16, is predominately flat to rolling country used for grazing cattle
- channel is well defined and with little storage
- chainage and heights for stream profile, as shown, have been extracted from a topographic map.

**Figure 6.16: Catchment**

![Catchment Diagram]

*Source: DTMR (2010b).*

**Solution**

The Rational Method formula (Equation 3) needs to be used to solve this – $Q_{50} = k \times C_{50} \times I_{50} \times A$.

The area has been calculated as $A = 6$ km², therefore $k = 0.278$. Then determine $C_{50}$ and $I_{50}$.

Because $I_{50}$ is required to determine $C_{50}$, calculate $I_{50}$ first.

**Step 1**

To determine $I_{50}$, firstly calculate $t_c$ for the catchment. To do this, use Bransby-Williams formula (Equation 4):

$$t_c = \frac{F \times L}{A^{0.1} \times S_e^{0.2}}$$
Previously it has been determined that $A = 6 \text{ km}^2$ (therefore $F = 58.5$) and the length of the catchment as $L = 1.9 \text{ km}$, therefore calculate $S_e$.

To calculate $S_e$, use equal area slope method.

Plot the stream profile. Set Point A as datum. Mark distances and heights relative to Point A, see Figure 6.17.

**Figure 6.17: Stream profile**

![Equal Area Slope](source: DTMR (2010b)).

Now calculate the area under the stream profile.

Using the above area, calculate the right ordinate of a triangle, which has the equivalent area.

Plot this ordinate (known as the equal area ordinate) and draw a line back to Point A, and calculate the slope of this line (Figure 6.18).
Now use Bransby-Williams formula, to calculate $t_c$.

$$
t_c = \frac{58.5 \times 1.9}{6^{0.1} \times 2.47^{0.2}}
$$

$t_c = 78$ minutes

**Step 2**

Now determine the rainfall intensity for the ARI 50, 20 and 10 year storm events, each with a duration of 78 minutes.

A variety of methods may be employed to generate an IFD table for the project site (see Section 6.3.2 – IFD Tables). For this worked example a software application, RAIN, developed by DTMR, has been utilised.
It can be seen from Figure 6.19, that 78 minutes falls between the standard durations of 1 and 1.5 hours. Interpolate the required intensities:

<table>
<thead>
<tr>
<th>Duration</th>
<th>Intensity ARI 50</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 h (60 min)</td>
<td>88.73</td>
</tr>
<tr>
<td>1.5 h (90 min)</td>
<td>69.33</td>
</tr>
</tbody>
</table>

\[
\left( \frac{(88.73 - 69.33)}{90 - 60} \right) \times (90 - 78) + 69.33 = 77.09
\]

Therefore \( I_{50} = 77.1 \text{ mm/h}. \)

Interpolating for the ARI 20 year and 10 year events, \( I_{20} = 64.8 \text{ mm/h} \) and \( I_{10} = 55.8 \text{ mm/h}. \)

**Step 3**

The last variable to determine is \( C_Y \), the run-off coefficient. See Section 6.6.3 – Run-off Coefficient and Appendix E (for Queensland application).

For the given catchment characteristics of the project site, the run-off coefficient \( C_{50} \) can be determined from Table E 1 in Appendix F.

Note that Table E 1 only provides the run-off coefficient for the ARI 50 year event (\( C_{50} \)) and these are used to calculate \( C_{50} \) as shown in Table 6.9.
Table 6.9: Calculation of $C_{50}$

<table>
<thead>
<tr>
<th>Intensity</th>
<th>27.1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Catchment relief</td>
<td>0</td>
</tr>
<tr>
<td>Catchment storage</td>
<td>10</td>
</tr>
<tr>
<td>Ground characteristics</td>
<td>40</td>
</tr>
<tr>
<td>$C_{50} = \frac{77.1}{100} = 0.77$</td>
<td></td>
</tr>
</tbody>
</table>

To determine $C_Y$ for the other ARIs, use the factors given in Table 6.10.

Table 6.10: Extract from Table E 2

<table>
<thead>
<tr>
<th>ARI</th>
<th>Coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.8 $C_{50}$</td>
</tr>
<tr>
<td>20</td>
<td>0.9 $C_{50}$</td>
</tr>
</tbody>
</table>

Therefore:

Table 6.11: Summary of $C_Y$ calculation

<table>
<thead>
<tr>
<th>$C_{20}$</th>
<th>0.77 x 0.9 = 0.69</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{10}$</td>
<td>0.77 x 0.8 = 0.62</td>
</tr>
</tbody>
</table>

**Step 4**

Now calculate the peak catchment discharge for each ARI using the Rational Method (using values from Table 6.11):

$q_{10} = 0.278 \times 0.62 \times 55.8 \times 6 = 57.7 \text{ m}^3/\text{s}$

$q_{20} = 0.278 \times 0.69 \times 64.8 \times 6 = 74.6 \text{ m}^3/\text{s}$

$q_{50} = 0.278 \times 0.77 \times 77.1 \times 6 = 98.9 \text{ m}^3/\text{s}$

### 6.9 Worked Examples (Urban): Urban Run-off

#### 6.9.1 Overland Flow Time

**Example 1: Overland flow time calculation**

Consider overland flow travel time for a sub-catchment in Melton, Victoria, with a length of 45.0 m, a slope of 0.01 m/m, and a roughness of 0.200. The design ARI event is 10 years.
The form of the equation (Equation 12) is:

\[ T_o = 6.94 \times \frac{(Ln)^{0.6}}{I^{0.4}S^{0.3}} \]

where

- \( T_o \) = Overland flow time (minutes)
- \( L \) = Overland flow path length (m)
- \( n \) = Manning's surface roughness
- \( I \) = Rainfall intensity for the design ARI event (mm/h)
- \( S \) = Slope of overland flow path (m/m)

The equation is solved for \( T_o^{0.4} \) and the corresponding number is interpolated from a tabulation of \( T_o^{0.4} \) values to arrive at a time of overland flow.

Table 6.12 shows the rainfall intensity frequency and duration table (IFD) for Melton, Victoria.

### Table 6.12: IFD table for Melton

<p>| Location: 37.675S 144.575E NEAR.. MELTON, Victoria Issued: 15/6/2012 |
|---------------------------------|------------------|------------------|------------------|------------------|------------------|------------------|------------------|
| Rainfall intensity in mm/h for various durations and Average Recurrence Interval |
| Average Recurrence Interval     |</p>
<table>
<thead>
<tr>
<th>Duration</th>
<th>1 YEAR</th>
<th>2 YEARS</th>
<th>5 YEARS</th>
<th>10 YEARS</th>
<th>20 YEARS</th>
<th>50 YEARS</th>
<th>100 YEARS</th>
</tr>
</thead>
<tbody>
<tr>
<td>6Mins</td>
<td>46.5</td>
<td>61.2</td>
<td>86.3</td>
<td>104</td>
<td>127</td>
<td>181</td>
<td>195</td>
</tr>
<tr>
<td>6Mins</td>
<td>42.4</td>
<td>57.0</td>
<td>80.4</td>
<td>96.6</td>
<td>116</td>
<td>148</td>
<td>176</td>
</tr>
<tr>
<td>10Mins</td>
<td>34.4</td>
<td>46.2</td>
<td>64.8</td>
<td>77.8</td>
<td>84.6</td>
<td>120</td>
<td>141</td>
</tr>
<tr>
<td>20Mins</td>
<td>24.8</td>
<td>33.3</td>
<td>46.3</td>
<td>55.3</td>
<td>67.2</td>
<td>54.6</td>
<td>91.4</td>
</tr>
<tr>
<td>30Mins</td>
<td>20.0</td>
<td>26.7</td>
<td>37.0</td>
<td>44.2</td>
<td>53.6</td>
<td>57.3</td>
<td>78.7</td>
</tr>
<tr>
<td>1Hr</td>
<td>13.3</td>
<td>17.7</td>
<td>24.4</td>
<td>28.9</td>
<td>35.0</td>
<td>43.7</td>
<td>51.0</td>
</tr>
<tr>
<td>2Hrs</td>
<td>8.00</td>
<td>11.4</td>
<td>15.5</td>
<td>18.3</td>
<td>22.0</td>
<td>27.3</td>
<td>31.7</td>
</tr>
<tr>
<td>3Hrs</td>
<td>5.02</td>
<td>8.76</td>
<td>11.8</td>
<td>13.9</td>
<td>16.6</td>
<td>20.6</td>
<td>23.8</td>
</tr>
<tr>
<td>6Hrs</td>
<td>4.21</td>
<td>5.55</td>
<td>7.39</td>
<td>8.61</td>
<td>10.3</td>
<td>12.6</td>
<td>14.5</td>
</tr>
<tr>
<td>12Hrs</td>
<td>2.26</td>
<td>3.48</td>
<td>4.61</td>
<td>5.35</td>
<td>6.35</td>
<td>7.76</td>
<td>8.91</td>
</tr>
<tr>
<td>24Hrs</td>
<td>1.24</td>
<td>2.16</td>
<td>2.88</td>
<td>3.31</td>
<td>3.93</td>
<td>4.80</td>
<td>5.52</td>
</tr>
<tr>
<td>48Hrs</td>
<td>0.92</td>
<td>1.28</td>
<td>1.71</td>
<td>1.99</td>
<td>2.36</td>
<td>2.90</td>
<td>3.35</td>
</tr>
<tr>
<td>72Hrs</td>
<td>0.70</td>
<td>0.90</td>
<td>1.22</td>
<td>1.43</td>
<td>1.70</td>
<td>2.09</td>
<td>2.41</td>
</tr>
</tbody>
</table>

(Rain data: 10.29, 3.56, 0.84, 40.40, 7.21, 1.94, demp=0.35, f2=4.3, f50=14.92) © Australian Government, Bureau of Meteorology

Source: BOM website.

The kinematic wave equation has been chosen for this example involves rainfall intensity, it must be solved together with the relationship between duration, \( t \) and intensity, \( I \). To simplify calculations, a tabulation of \( T_o^{0.4} \) can be prepared as shown in Table 6.13.
Table 6.13: Tabulation of $T_o I^{0.4}$

<table>
<thead>
<tr>
<th>Duration, $T_o$ (mins)</th>
<th>Average recurrence interval</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1 year</td>
</tr>
<tr>
<td>5</td>
<td>23.0</td>
</tr>
<tr>
<td>6</td>
<td>26.9</td>
</tr>
<tr>
<td>10</td>
<td>41.2</td>
</tr>
<tr>
<td>20</td>
<td>72.2</td>
</tr>
<tr>
<td>30</td>
<td>99.4</td>
</tr>
<tr>
<td>60</td>
<td>168.9</td>
</tr>
</tbody>
</table>

**Solve for $T_o$.**

Using Equation 12:

$$T_o = 6.94 \times \frac{(45.0 \times 0.200)^{0.6}}{I^{0.4} \times 0.01^{0.3}}$$

Therefore:

$$T_o I^{0.4} = 103.3$$

The value of $T_o I^{0.4}$ corresponds to an overland flow time between 20 and 30 minutes for the 10 year ARI. By interpolation from Table 6.13:

$$T_o = 21.0 \text{ minutes}$$

From Table 6.13, if the design ARI is two years, a value of 103.3 corresponds to duration of 27.3 minutes. For a one year ARI the duration is 31.7 minutes, while for an ARI of 100 years it is 15.8 minutes.

**Example 2: Overland flow time for multiple flow segments**

Consider overland flow travel time for two segments a and b, where segment a is an asphalt paved surface and segment b is a grassed median batter as shown in the Figure 6.20.
For this example the project location is Melton, Victoria, design ARI event is 10 years and the following Manning’s surface roughness values have been adopted:

\[ n = 0.013 \text{ for asphalt paved surface} \]

\[ n = 0.200 \text{ for short grass median batter} \]


For the two segments a and b, the total overland flow time is (Equation 13):

\[ T_{\text{total}} = T_{\left( L_A \right)} + T_{B\left( L_A + L_B \right)} - T_{B\left( L_A \right)} \]

where

\[ L_A = \text{The length of flow for segment a} \]

\[ T_{\left( L_A \right)} = \text{The time for flow across segment a, from point a to b} \]

\[ T_{B\left( L_A + L_B \right)} = \text{The time for flow across the total length of segments a and b using the slope and roughness of segment b, from point a to c} \]

\[ T_{B\left( L_A \right)} = \text{The time for flow across a virtual segment along a length equal to segment a using the slope and roughness of segment b, from point a to b} \]

Note it is incorrect to add values of \( T_{\text{ToI0.4}} \) for each of the segments, as the kinematic wave equation is based on the assumption that no flow is entering the flow segment from upstream.
Solve for \( T_{(L_A)} \)

The time for flow across segment a (paved asphalt surface) point a to b:

\[
T_{(L_A)} = 6.94 \times \frac{(13.0 \times 0.013)^{0.6}}{I^{0.4} \times 0.03^{0.3}}
\]

\( T_{(L_A)} I^{0.4} = 6.8 \)

By interpolation from the tabulation of \( T_c I^{0.4} \) values for Melton, Victoria (Table 6.13) for the 10 year ARI:

\[
T_{(L_A)} = 5 \times \frac{6.8}{32} = 1.1 \text{ minutes}
\]

Solve for \( T_{B(L_A+L_B)} \)

The time for flow across the total length of segments a and b, using the slope and roughness of segment b (short grass median batter) (point a to c) is as follows:

\[
T_{B(L_A+L_B)} = 6.94 \times \frac{(19.2 \times 0.200)^{0.6}}{I^{0.4} \times 0.167^{0.3}}
\]

\( T_{B(L_A+L_B)} I^{0.4} = 26.6 \)

Again by interpolation:

\[
T_{B(L_A+L_B)} = 5 \times \frac{26.6}{32.0} = 4.2 \text{ minutes}
\]

Solve for \( T_{B(L_A)} \)

The time for flow across a virtual segment along a length equal to segment a using the slope and roughness of segment b (short grass median batter) (point a to b):

\[
T_{B(L_A)} = 6.94 \times \frac{(13.0 \times 0.200)^{0.6}}{I^{0.40} \times 0.167^{0.3}}
\]

\( T_{B(L_A)} I^{0.4} = 21.1 \)

By interpolation:

\[
T_{B(L_A)} = 5 \times \frac{21.1}{32.0} = 3.3 \text{ minutes}
\]
Solve for $T_{\text{total}}$

The total overland flow time for two segments $a$ and $b$:

$$T_{\text{total}} = T_a + T_b + T_a - T_b$$

$$T_{\text{total}} = 1.1 + 4.2 - 3.3 \text{ minutes}$$

$$T_{\text{total}} = 2.0 \text{ minutes}$$

Additional segments

For each additional segment, the following value should be added (see Equation 14):

$$T_X(L_{\text{total}}) - T_X(L_{\text{total}} - L_X)$$

where

$X$ = The segment name and $L_{\text{total}}$ is the total length of flow, including the current segment $X$. Use the slope and roughness of segment $X$

$T_X(L_{\text{total}})$ = The time for flow across the total length of segments $a$, $b$ and $X$, using the slope and roughness of segment $X$ (point $a$ to $d$)

$T_X(L_{\text{total}} - L_X)$ = The time for flow across a virtual segment along a length equal to segment $a$ and $b$, using the slope and roughness of segment $X$ (point $a$ to $c$)

Figure 6.21: Overland flow travel time for additional segments

Source: VicRoads.
6.9.2 Use of Rational Method

The following provide examples of using the Rational Method, for different catchment characteristics.

**Example 1: Single land-use catchments**

Consider run-off from an impervious catchment (run-off coefficient = 1, i.e. all rain falling on the catchment flows out of the catchment) of plan area $A$ ha, so shaped that all surface run-off is conveyed directly to, and discharged from, O (Figure 6.22). If this catchment were subject to a storm event of constant intensity, $I$ mm/h for a long duration, then the resulting relationship between discharge, $Q$, and time would take the form of the run-off hydrograph shown in Figure 6.23. Note that the ‘time of rise’ of the hydrograph is $t_c$ minutes, the catchment's time of concentration.

**Figure 6.22:** Impervious catchment of area $A$ ha draining to O

The peak (steady state) outflow that occurs at O, $Q_O$, must equal the rate at which precipitation is being supplied to the catchment during the event since there are no losses. The discharge at O is found by application of the rational formula method with the run-off coefficient equal to one.

Consider now run-off from Catchment B (Figure 6.24), a rectangular, impervious catchment of uniform slope and plan area 0.10 ha, draining to O, whose time of concentration (i.e. travel time from the most remote point) is 20 minutes. Catchment B is located in the climatic region whose rainfall IFD relationship for frequency ARI = 10 years is presented in Figure 6.25.
If the storm events (intensity constant) from the curve in Figure 6.25 are applied to Catchment B, the resulting run-off hydrographs at O take the forms shown in Figure 6.26. It has been assumed that the speed at which the run-off travels to the discharge point at O, is the same for the whole catchment, hence the straight lines in the hydrographs in Figure 6.26.

- intensity $10I_{15} = 55$ mm/h, for a duration of 15 minutes (dash-dot line)
- intensity $10I_{20} = 48$ mm/h, for a duration of 20 minutes (solid line)
- intensity $10I_{25} = 42$ mm/h, for a duration of 25 minutes (dotted line)
- intensity $10I_{1} = 25$ mm/h, for a duration of 60 minutes (dashed line).

For the hydrographs listed above, the peak flow rate at O, $Q_O$, is given by application of Equation 22 and is:

$$Q_{peak} = \frac{(C \times A) \times 10I_D}{0.36}$$  \hspace{1cm} \text{Equation 22}$$

where

- $Q_{peak}$ = Peak flow rate (L/s)
- $C$ = Run-off coefficient (≈ 1.00 in each case)
- $A$ = Catchment area contributing the discharge at time of concentration (ha)
- $10I_D$ = Rainfall intensity corresponding to an ARI of 10 years with a duration of $D$ (mm/h)
For the first case, where the storm duration (15 minutes) is less than the time of concentration (20 minutes), only 15/20 parts of the catchment contribute to the peak discharge. For the three other cases, the storm duration is equal to, or longer than, the time of concentration and so the whole catchment contributes to the peak discharge. Therefore the peak discharges are:

\[ Q_0 = (1.0 \times 0.10 \times (15/20)) \times 55/0.36 = 11.5 \text{ L/s} \]

\[ Q_0 = (1.0 \times 0.10) \times 48/0.36 = 13.3 \text{ L/s} \]

\[ Q_0 = (1.0 \times 0.10) \times 42/0.36 = 11.7 \text{ L/s} \]

\[ Q_0 = (1.0 \times 0.10) \times 25/0.36 = 6.9 \text{ L/s} \]

Figure 6.26: Hydrograph for rainfall bursts on Catchment B


The peak discharges can then be plotted against storm duration to determine the design discharge for the catchment as shown in Figure 6.27. The maximum discharge occurs when the rainfall duration is 20 minutes, i.e. when the rainfall duration is equal to the time of concentration for Catchment B (see Section 6.4.1).
The trends displayed in Figure 6.26 and Figure 6.27 are typical when Rational Method assumptions and rainfall intensity-duration relationships are combined. Storms which produce peak discharge from a catchment have durations equal to the catchment time of concentration, $t_c$, or longest travel time.

The application of a run-off coefficient to take account of losses due to infiltration, depression storage, etc., in pervious catchments has little influence on the assumed relationship between critical storm duration and $t_c$ stated above.

**Example 2: Multi-land-use catchments**

When discharge estimates are required in a multi-land-use catchment of the type illustrated in Figure 6.28, conventional use of the Rational Method requires the critical storm duration to be still set to the total catchment longest travel time. This is accepted practice even though overland flow speed in its various components may be demonstrably different. Travel time from point F will therefore over-ride travel from point P because run-off movement across the pervious surface is much slower than across the impervious surface.

The only concession made in the Rational Method for the composite nature of the catchment draining to O is in the adoption of a weighted run-off coefficient in proportion to the areas of the land use components.

Given a catchment containing two different land use areas, $A_1$ and $A_2$, with corresponding run-off coefficients $C_1$ and $C_2$, the weighted run-off coefficient, $C_w$, is (Equation 23): 

$$C_w = \frac{C_1A_1 + C_2A_2}{A_1 + A_2}$$
The following example illustrates a conventional application of the Rational Method to a multi-land-use catchment, in this case Catchment E (Figure 6.28):

- impervious area $A_i = 0.10$ ha with $C_i = 1.00$
- pervious area $A_p = 0.20$ ha with $C_p = 0.40$
- travel time from point P to O $t_i = 20$ minutes
- travel time from point F to O $t_c = 60$ minutes

Hence the weighted run-off coefficient $C_w$ is:

$$C_w = \frac{(1.00 \times 0.10) + (0.40 \times 0.20)}{(0.10 + 0.20)} = 0.60$$

Time of concentration must be the greater of $t_i$ and $t_c$ and therefore is equal to 60 minutes. Using the same rainfall intensity data as used in the previous single-land-use example, the design discharge occurs when the storm duration is equal to the time of concentration. Therefore the design rainfall intensity is 25 mm/h. Therefore the design discharge from the Catchment E is given by application of the Rational Method:

$$Q_o = \frac{(C_w A)_{10} t_i}{0.36} = \frac{(0.60 \times 0.30) \times 25}{0.36} = 12.5 \text{ L/s}$$

But this is less than the peak discharge estimated at O for that part of Catchment E previously determined for Catchment B.
Example 3: Time/area representation

The Rational Method assumption that the speed with which run-off elements travel to discharge point, O, is steady, as stated in the previous section, is capable of much wider interpretation than might be assumed for it implies the existence of a fixed relationship of proportionality between catchment area and time. Figure 6.29 illustrates this for the case of a rectangular impervious catchment of uniform slope and plan area A, draining to point O. Time of concentration for the catchment is $t_c$ minutes.

In this catchment, the speed of travel is the same at all points at all times (Rational Method assumptions). It follows that run-off from the lowest quarter of the area has either passed through O at time $t_c/4$ minutes after the commencement of a constant intensity storm on the catchment or just arrived there. The area, A/4, is described in this situation as the area contributing to run-off at O at time $t_c/4$. Similarly, this applies for the area/time pairs A/2 and $t_c/2$, 3A/4 and 3$t_c/4$, A and $t_c$. These pairs, plotted in Figure 6.29, yield the time-area relationship for the catchment.

Figure 6.29: Time-area graph for simple catchment of area, A and time of concentration, $t_c$


Similar graphs can be constructed to describe the time-area responses of real-world catchments which are irregular in shape, non-uniform in slope and which include a mixture of pervious and impervious components. Representation of such catchments on simple time-area plots requires the ‘contributing area’ of each component to be expressed in terms of equivalent impervious area i.e. the product of run-off coefficient and component area (CA). The time-area graph of Catchment E, presented in Figure 6.30, illustrates this process.
Figure 6.30: Time-area graph for Catchment E


Catchment E:

- At 20 min: Rain fall stops. All of the impervious area contributing, one-third of the pervious area contributing (i.e. time of concentration is 60 minutes for the pervious area and flow is assumed to be constant, therefore 20 mins/60 mins contributing).

- At 40 min: All of impervious area ceases to contribute, the lowest third of the pervious area ceases to contribute but mid third area of pervious area contributes to discharge. Therefore at this time one-third of the pervious area contributes.

- At 60 min: The furthest point in the previous area begins to contribute. The mid third area ceases to contribute to discharge. The furthermost third of the pervious area contributes.

- At 80 min: All flow ceases.

Example 4: Partial area method

This method combines a number of Rational Method and time-area properties leading to a design flow estimation procedure that is non-graphical and can be carried out ‘by hand’ (i.e. tabular), using spread-sheet technology or by computer programming. It produces two possible critical design storm outflows (constant intensity storms) at the discharge point of each drainage sub-catchment. The two design outflows arise from ‘full-area’ and ‘part-area’ considerations.

Full-area flow estimate

Peak run-off flows are determined for single or multiple land-use drainage sub-catchments in the manner described when using the Rational Method procedures for multiple land use:

1. critical design storm duration = $t_o$, travel time from the outer extremity of the most remote pervious area
2. full equivalent impervious area $(CA)_{full} = (C_W A)$, where $C_W$ is the weighted run-off coefficient
3. rainfall intensity, $YI_{tc} = \text{average intensity, duration to be obtained from catchment rainfall intensity-duration chart for selected ARI of } Y \text{ years}$

4. discharge is calculated using the Rational Method formula.

The flows calculated by this approach are referred to as full-area flow estimates.

Directly connected impervious (or paved) areas are those that contribute run-off directly to the drainage collection network. Such run-off may be conveyed by pipe, channel or informally across the impervious surface before reaching the formal collection system (e.g. roadside channels). Run-off, from impervious (or paved) areas not directly connected to the formal collection system, is included with the pervious areas, $A_p$.

**Part-area flow estimate**

Peak run-off flows are also determined for the same sub-catchments using:

1. critical design storm duration = $t_c$, travel time from the outer extremity of the most remote, directly connected impervious area

2. 'part' equivalent impervious area is calculated in Equation 20:

$$CA_{part} = C_i A_i + \left( \frac{t_i}{t_c} \times C_p A_p \right)$$

3. rainfall intensity $YI_{tc} = \text{average intensity, duration to be obtained from catchment rainfall intensity-duration chart for selected ARI of } Y \text{ years}$.

The flows calculated by this approach are referred to as part-area flow estimates.

The theoretical basis for the part equivalent impervious area calculation $(CA)_{part}$ follows from the time-area representation (see Example of time/area representation earlier in this section) of the lumped paved and separately lumped pervious components present in a multi-land-use catchment.

Strict time/area representation of the paved and pervious components of real-world catchments differ from the simple model. Justification for its use is therefore claimed on the grounds of simplicity and adequacy. It is simple because it translates into an easily understood tabular flow estimation procedure, and adequate because it yields estimates which involve much the same level of uncertainty as is associated with more complex and time-consuming methods.

**Example of partial area rational method**

Consider the peak flow estimates (design ARI = 10 years) which may be derived from the full-area and part-area conditions which arise in Catchment E, represented in Figure 6.28.

Full-area estimate by the Rational Method formula:

Equivalent impervious area = $(CA_w) = 1.0 \times 0.1 + 0.4 \times 0.2 = 0.18 \text{ ha}$

Storm duration (equal to $t_c$) = 60 minutes, hence $10I_{60} \text{ m} = 25 \text{ mm/h}$

Hence, full-area peak flow estimate:

$$Q_f = \frac{0.18 \times 25}{0.36} = 12.5 \text{ L/s}$$
Part-area estimate \((CA)_{\text{part}}\) is:

\[
\text{Equivalent impervious area} = C_i A_i + \left( \frac{t_i}{t_c} \times C_p A_p \right)
\]

\[
= 1.0 \times 0.10 + \left( \frac{20}{60} \times 0.4 \times 0.2 \right)
\]

\[
= 0.127 \text{ ha}
\]

Storm duration (equal to \(t_i\)) = 20 minutes, hence \(I_{20} = 48 \text{ mm/h}\)

Part-area peak flow estimate:

\[
Q_p = \frac{0.127 \times 48}{0.36} = 16.9 \text{ L/s}
\]

Recommendation: outlet works for point O in Catchment E should carry a design flow (ARI = 10 years) of 16.9 L/s.
References


Austroads 2006, *Guide to road design: part 2: design considerations*, AGRD02/06, Austroads, Sydney, NSW.

Austroads 2008, *Guide to road design: part 7: geotechnical investigation and design*, AGRD07/08, Austroads, Sydney, NSW.


Austroads 2009c, *Guide to road design: part 6B: roadside environment*, AGRD06B/09, Austroads, Sydney, NSW.


Austroads 2009e, *Guide to pavement technology: part 7: pavement maintenance*, AGPT07/09, Austroads, Sydney, NSW.

Austroads 2009f, *Guide to bridge technology: part 4: design procurement and concept design*, AGBT04/09, Austroads, Sydney, NSW.

Austroads 2009g, *Guide to bridge technology: part 7: maintenance and management of existing bridges*, AGBT07/09, Austroads, Sydney, NSW.

Austroads 2010a, *Guide to road design: part 1: introduction to road design*, AGRD01/10, Austroads, Sydney, NSW.

Austroads 2010b, *Austroads glossary of terms*, 4th edn, AP-C87/10, Austroads, Sydney, NSW.

Austroads 2010c, *Guide to road design: part 3: geometric design*, AGRD03/10, Austroads, Sydney, NSW.

Austroads 2010d, *Guide to road design: part 6: roadside design, safety and barriers*, AGRD06/10 Austroads, Sydney, NSW.


Austroads 2013b, *Guide to road design: part 5B: drainage: open channels, culverts and floodways*, AGRD05B/13, Austroads, Sydney, NSW.


Department of Main Roads 2000, *Fauna sensitive road design: vol 1: past and existing practices*, DMR, Brisbane, Qld.

Department of Natural Resources & Water 2007, *Queensland urban drainage manual*, vol. 1, 2nd edn, DNRW, Brisbane, Qld.

Department of Transport and Main Roads 2010a, *Fauna sensitive road design manual: vol 2: preferred practices*, DTMR, Brisbane, Qld.

Department of Transport and Main Roads 2010b, *Road drainage manual*, 2nd edn, DTMR, Brisbane, Qld.


Fairfull, S & Witheridge, G 2003, *Why do fish need to cross the road? Fish passage requirements for waterway crossings*, NSW Fisheries, Cronulla, NSW.


Main Roads Western Australia 2006, *Floodway design guide*, MRWA, Perth, WA.


Roads and Traffic Authority 1999, *RTA code of practice for water management, road development and management*, RTA, Sydney, NSW.

Rosewell, CJ & Keats, J 1993, *SOILLOSS: a program to assist in the selection of management practices to reduce erosion*, 2nd edn, technical handbook no. 11, Department of Conservation and Land Management, Sydney, NSW.


Savage, M 2010, *Specification for supply of recycled material for pavements, earthworks and drainage*, Department of Environment, Climate Change and Water, Sydney, NSW.


Thompson, PL & Kilgour, RT 2006, *Hydraulic design of energy dissipators for culverts and channels*, hydraulic engineering circular no. 14, Federal Highway Administration, Arlington, Virginia, USA.


Western Australia Department of Water 2004, *Stormwater management manual for Western Australia 2004–2007*, Western Australia Department of Water, Perth, WA.


Wong, THF (ed) 2006, *Australian runoff quality: a guide to water sensitive urban design*, Engineers Media, Crows Nest, NSW.

**Standards Australia**

AS/NZS 1254:2010, *PVC-U pipes and fittings for stormwater and surface water applications*.

AS 1597.1:2010, *Precast reinforced concrete box culverts: small culverts not exceeding 1200 mm span and 1200 mm height*.

AS 1597.2:1996, *Precast reinforced box culverts: large culverts (from 1500 mm span and up to and including 4200 mm span and 4200 mm height)*.


**Further Reading**


Auckland Regional Council (ARC) 2003, *Stormwater management devices: design guidelines manual* (TP10), Auckland Regional Council, NZ.


**Federal Highway Administration (FHWA) Hydraulic Design Series**

HDS 04 2008, *Introduction to highway hydraulics* (NHI-08-090)

HDS 05 2012, *Hydraulic design of highway culverts*, 3rd edn (HIF-12-026)

HDS 06 2001, *River engineering for highway encroachments* (NHI-01-004)

**Federal Highway Administration (FHWA) Hydraulic Engineering Circular**

HEC 09 2005, *Debris control structures evaluation and counter measures* (IF-04-016)


HEC 14 2006, *Hydraulic design of energy dissipaters for culverts and channels* (NHI-06-086)

HEC 15 2005, *Design of roadside channels with flexible linings*, 3rd edn (IF-05-114)

HEC 18 2012, *Evaluating scour at bridges*, 5th edn (HIF-12-003)

HEC 20 2012, *Stream stability at highway structures*, 4th edn (HIF-12-004)

HEC 21 1993, *Bridge deck drainage systems* (SA-92-010)

HEC 22 2009, *Urban drainage design manual*, 3rd edn (NHI-10-009)

HEC 23 2009, *Bridge stability and stream instability countermeasures experience, selection and design guidance*, 3rd edn, vol. 1 and 2 (NHI-09-111 and NHI-09-112)

HEC 26 2010, *Culvert design for aquatic organism passage* (HIF-11-008)
Appendix A  Design of an Infiltration Basin

A.1 Design of an Infiltration Basin

The basin is to be sited in an area of sandy loam (permeability of $5 \times 10^{-5}$ m/s) which has a catchment area of five ha containing 50% impervious areas. The time of concentration is 20 minutes and it is desired to design the basin to cater for a storm event with a 10 year ARI. The groundwater is located approximately 3 m below the floor of the basin. The batter slopes are to be 1:8.

Referring to Table A 1 the following process is followed:

- using the procedures in Pilgrim (2001) the rainfall intensities (col. 2 of Table A 1) were calculated for storms of varying durations (col. 1) with a 10 year ARI
- the Rational Method was used to estimate the run-off volumes (col. 3) assuming constant intensity storm events
- the volume of water entering the basin for each time period was then calculated (col. 4)
- a trial basin size of 60 m long with a width of 25 m was adopted
- the depth in the basin of the run-off is then calculated assuming that there are no losses (col. 5)
- the depth in the basin is halved (col. 6)
- half of the hydraulic radius is calculated (col. 7)
- radius of influence is calculated (col. 8) from Equation A1:

$$ R = r + 50 \left( H^1 + d \right) K^{0.5} $$  \hspace{1cm} A1

where

- $H^1 = \text{Half the depth of water in basin (m)}$
- $K = \text{Soil permeability (m/hour)}$

- Outflow is calculated (col. 9) from Equation A2:

$$ Outflow = \frac{K\pi(2T)(H' + d)^2}{\ln \left( \frac{R}{r} \right)} $$  \hspace{1cm} A2

where

- $Outflow = \text{At time } 2T \text{ (m}^3\text{)}$
- $K = \text{Soil permeability (m/s)}$
- $T = \text{Storm duration (sec)}$
- $H' = \text{Half depth of water in basin (m)}$
- $d = \text{Distance from floor of basin to the water table (m)}$
- $r = \text{Half wetted perimeter}$
\[ R = \text{Radius of influence from the centre of the basin} \]

- Storage is calculated by subtracting the outflow from the inflow.
- For the new storage volume a new depth can be calculated to check that it does not exceed the recommended maximum of 0.6 m.

\[
\text{Depth at maximum storage volume} = \left[ \frac{W^2 + 4(BS)V^{0.5}}{2BS} \right] - W
\]

where

\[ W = \text{Width of basin floor (m)} \]
\[ BS = \text{Batter slope (1:x)} \]
\[ V = \text{Storage volume (m}^3\text{)} \]
\[ L = \text{Length of basin floor (m)} \]

and detention time \( s \) is given by Equation A4:

\[ \frac{d}{K(H' + \frac{d}{d})} \]

The depth at maximum storage volume and the detention time for the example can then be calculated from these equations.

\[
\text{Depth} = \frac{((25^2 + 4[8] \times 1070/60)^{0.5} - 25)/(2\times8)}{2} = 0.59 \text{ m} < 0.6 \text{ m}, \text{ therefore acceptable}
\]

Check the detention time using equation \[ = 3/(5 \times 10^{-5}((0.54/2+3)/3)) = 55 \text{ 050 s} \] = 15.3 hours < 72 hours, therefore acceptable

Check the detention time at minimum inundation level \[ = 3/(5 \times 10^{-5}(0/2+3)/3) = 16.7 \text{ hours} > 12 \text{ hours}, \text{ therefore acceptable} \]
Table A 1: Tabulation of inflow and outflow hydrographs

<table>
<thead>
<tr>
<th>Storm duration (h)</th>
<th>10(I) (mm/h)</th>
<th>(Q_{10}) (m(^3)/s)</th>
<th>Inflow (m(^3))</th>
<th>Depth (m)</th>
<th>Depth/2 (m)</th>
<th>r (m)</th>
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Appendix B  Drainage Construction Material Considerations

The types of pipes and culverts generally used in drainage systems are listed in Table B 1, outlining the general properties and applications of the different types of pipes. Designers should confirm the use of the pipe types with the relevant road agency. Designers should confirm the types of pipes for use as not all road agencies endorse the use of UPVC or HDPE pipes or have conditions on their use.
### Table B 1: Pipe and culvert materials

<table>
<thead>
<tr>
<th>Culvert type/material</th>
<th>Steel reinforced concrete pipes (SRCPs)</th>
<th>Fibre reinforced concrete pipes (FRCPs)</th>
<th>Reinforced concrete box culverts (RCBCs)</th>
<th>Corrugated aluminium pipes (CAPs)</th>
<th>Corrugated steel pipes (CSPs)</th>
<th>Unplasticised polyvinyl chloride (UPVC)</th>
<th>High-density polyethylene pipes (HDPE)</th>
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<tbody>
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<td>Relevant (main) standards</td>
<td>AS/NZS 4058 for manufacture. AS/NZS 3725 for design and installation.</td>
<td>AS 4139 for manufacture. AS/NZS 3725 for some loading requirements.</td>
<td>AS 1597.1 for design, manufacture and installation of spans up to 1200 mm. AS 1597.2 for design, manufacture and installation of spans greater than 1500 mm and up to 4200 mm.</td>
<td>AS/NZS 2566.1 for structural design. AS/NZS 2566.2 for installation.</td>
<td>AS/NZS 2041.1 for design methods. AS/NZS 2041.2 for installation. AS/NZS 2041.4 for helically formed sinusoidal pipes. AS/NZS 2041.6 for bolted plate structures.</td>
<td>AS/NZS 2566.1 for design details. AS/NZS 2566.2 for installation requirements. AS/NZS 1254 for stormwater and surface water applications.</td>
<td>AS/NZS 5065 for manufacture. See AS/NZS 2566.1 for design details. AS/NZS 2566.2 for installation requirements.</td>
</tr>
<tr>
<td>Size&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>225 mm – 3600 mm diameter available.</td>
<td>225 mm – 750 mm.</td>
<td>300 mm x 225 mm to 1200 mm x 1200 mm for small units. Large units from 1500 mm x 900 mm – 4200 mm x 4200 mm.</td>
<td>300 mm – 3600 mm diameter.</td>
<td>300 mm – 1800 mm (68 mm x 13 mm corrugations). 900 mm – 3600 mm (75 mm x 25 mm). 1200 mm – 3600 mm (125 mm x 25 mm)&lt;sup&gt;(2)&lt;/sup&gt;.</td>
<td>90 mm – 225 mm diameter.</td>
<td>100 mm – 1500 mm diameter.</td>
</tr>
<tr>
<td>Culvert type/material</td>
<td>Steel reinforced concrete pipes (SRCPs)</td>
<td>Fibre reinforced concrete pipes (FRCPs)</td>
<td>Reinforced concrete box culverts (RCBCs)</td>
<td>Corrugated aluminium pipes (CAPs)</td>
<td>Corrugated steel pipes (CSPs)</td>
<td>Unplasticised polyvinyl chloride (UPVC)</td>
<td>High-density polyethylene pipes (HDPE)</td>
</tr>
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</tr>
<tr>
<td><strong>Strength</strong></td>
<td>Typically classes 2 to 10 as specified in AS 4058 but may be manufactured up to Class 12.</td>
<td>Classes 1, 2, 3 &amp; 4 as specified in AS 4139 but may be manufactured up to Class 6.</td>
<td>Design to 112 kN test load to AS 1597.1 for small span units. Determined in accordance with design requirements in AS 1597.2 for large span units.</td>
<td>As specified in AS/NZS 2041.</td>
<td>As specified in AS 2041.4.</td>
<td>As specified in AS 1254 or AS 2439. Class 12 minimum for plain pipes in road works. For corrugated pipes, Class 400 behind kerbs, Class 1000 minimum under roads. See AS/NZS 2566 for design details.</td>
<td>Specified by Manufacturer’s Profile Number in the range 003 to 290. Stiffness SN 1800 N/m/m is the minimum allowable. See AS/NZS 2566.1.</td>
</tr>
<tr>
<td><strong>Stock length</strong></td>
<td>2.44 m. Flush joint pipes – 1.22 m if ordered.</td>
<td>4.0 m.</td>
<td>1.22 m</td>
<td>6.0 m to 12.0 m as ordered for ease of transporting, but can vary considerably.</td>
<td>6.0 m to 12.0 m as ordered for ease of transporting, but can vary considerably.</td>
<td>6.0 m</td>
<td>6.0 m</td>
</tr>
<tr>
<td><strong>Joint types</strong></td>
<td>Flush, socket &amp; rubber ring. Dia &lt; 600 mm recommend rubber ring or socket. Dia &gt; 675 mm recommend flush or butt.</td>
<td>Double v-ring, rubber ring superlite. Rebated joints do not comply with current AS 4139.</td>
<td>Butt</td>
<td>Patented bolted bands or plate bolting.</td>
<td>Patented bolted bands or plate bolting.</td>
<td>Solvent weld, rubber ring.</td>
<td>Rubber ring, socket fusion, fusion weld, bolted flange.</td>
</tr>
<tr>
<td>Culvert type/material</td>
<td>Steel reinforced concrete pipes (SRCPs)</td>
<td>Fibre reinforced concrete pipes (FRCPs)</td>
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</tr>
<tr>
<td>Minimum cover</td>
<td>Dependent on size and class of pipe. Larger pipes can have less than 0.4 m of fill for class 2 whereas smaller pipes generally require more than 600 mm for the same class. See Concrete Pipe Association of Australasia software ‘Pipeclass’ (CPAA 2012).</td>
<td>See manufacturer’s installation details.</td>
<td>In certain situations can be directly driven on but depends on base slab design. It is generally good practice to locate the crown of the box unit outside of the pavement layers. See AS 1597.1 or AS 1597.2 and road agencies for further details.</td>
<td>600 mm for typical highway loads. See manufacturer’s requirements for construction traffic loadings.</td>
<td>600 mm for highway loads. See manufacturer’s requirements for construction traffic loadings.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Other installation conditions</td>
<td>Invert in fresh water for extended periods, typical conditions.</td>
<td>Invert in fresh water for extended periods, typical conditions.</td>
<td>Invert in fresh water for extended periods, typical conditions. Saltwater and aggressive soils require appropriate cover, cement and treatment. Maximum fill height limited to 2 m for small spans and 10 m for large spans.</td>
<td>Aggressive soil (e.g. pH 4 to 9 high chloride, high sulphate) invert in fresh water for extended periods, typical conditions.</td>
<td>Not suitable for saltwater of aggressive soil. Maximum height of fill may vary from 9 m to 30 m depending on the site loading conditions and the structural strength of the pipe (VicRoads 2003).</td>
<td>Saltwater, aggressive soil, invert in fresh water for extended periods, typical conditions.</td>
<td>Saltwater, aggressive soil, invert in fresh water for extended periods, typical conditions. Fill heights in excess of 30 m have been achieved.</td>
</tr>
<tr>
<td>Culvert type/material</td>
<td>Steel reinforced concrete pipes (SRCPs)</td>
<td>Fibre reinforced concrete pipes (FRCPs)</td>
<td>Reinforced concrete box culverts (RCBCs)</td>
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<td>----------------------------------------</td>
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</tr>
<tr>
<td><strong>Advantages</strong></td>
<td>Spun concrete more dense than wet cast, making it more durable. Does not require support from surrounding soil.</td>
<td>Does not require support from surrounding soil. Internal diameters remain the same for all pipe classes.</td>
<td>Have distinct structural and geometric advantages over pipes. See AGRD Part 5B Section 3. Continuous base slab minimises differential settlement. Does not require support from surrounding soil.</td>
<td>Lightweight – lower freight costs and can be installed without heavy machinery. Can be installed in high embankments.</td>
<td>Lightweight – lower freight costs and can be installed without heavy machinery. Can be installed in high embankments.</td>
<td>Lightweight – lower freight costs and can be installed without heavy machinery. Chemically inert. Hydraulically smooth. Can be installed in high embankments.</td>
<td>Lightweight – lower freight costs and can be installed without heavy machinery. Chemically inert. Hydraulically smooth. Can be installed in high embankments.</td>
</tr>
<tr>
<td><strong>Disadvantages</strong></td>
<td>Heavy – increased freight costs and requires heavy machinery for installation. Internal diameters decrease with increased class. Lower heights of fill possible.</td>
<td>Heavy – increased freight costs and requires heavy machinery for installation. Lower heights of fill possible. Refer also to any road agency requirements.</td>
<td>Heavy – increased freight costs and requires heavy machinery for installation. Wet cast concrete more prone to chemical attack. Lower fill heights.</td>
<td>Light weight makes them susceptible to floating. Lower strength than steel pipes. More costly than steel. Corrugations have high Manning’s n. Compaction of surrounding soil is vital for strength.</td>
<td>Light weight makes them susceptible to floating. Highly susceptible to corrosion and other chemical attack. Corrugations have high Manning’s n. Compaction of surrounding soils is vital for strength.</td>
<td>Light weight makes them susceptible to floating. Standard pipes not suited to pressure applications. Compaction of surrounding soil is vital for strength. Only available in limited sizes. Refer also to any road agency requirements.</td>
<td>Light weight makes them susceptible to floating. Standard pipes not suited to pressure applications. Compaction of surrounding soil is vital for strength. Only available in limited sizes. Refer also to any road agency requirements.</td>
</tr>
<tr>
<td>Culvert type/material</td>
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</tr>
<tr>
<td>Design life</td>
<td>Pipes manufactured and installed in accordance with the relevant standards can expect to achieve a design life in excess of 100 years.</td>
<td>Pipes manufactured to AS 4139 can expect to achieve a design life in excess of 50 years provided the semi-rigid property of the pipe material remains.</td>
<td>Culverts manufactured and installed in accordance with the relevant standards can expect to achieve a design life of 100 years.</td>
<td>Design life in excess of 75 years may be achieved in low abrasion environments having pH between 5 and 9 and resistivity above 1500 ohm-cm. CaCO₃ levels do not affect service life.</td>
<td>A 50 year design life can be expected for CCP’s having standard galvanising where pH is between 6 and 10, resistivity f between 2000 and 10 000 ohm-cm and CaCO₃ levels are above 50 ppm.</td>
<td>Design life in excess of 50 years (and as high as 100 years) is achievable when designed and installed in accordance with the relevant standards and manufacturer’s requirements.</td>
<td>Design life in excess of 50 years (and as high as 100 years) is achievable when designed and installed in accordance with the relevant standards and manufacturer’s requirements.</td>
</tr>
</tbody>
</table>

1. The minimum recommended size of pipes for stormwater drainage system is 300 mm diameter. The minimum recommended size across a road formation (i.e. culvert) is 375 mm diameter, larger may be required in areas of high debris (refer to local requirements). Minimum recommended height for a box culvert is 375 mm although 300 mm may be used in tight conditions. Refer to AGRD Part 5B – Section 3 for further details.
2. Pipe diameters as low as 150 mm (with corrugation size 38 mm x 6.5 mm) are available but not commonly used.
3. Flush or butt joints are recommended where movement will be minimal.
4. All cross drainage and longitudinal drainage pipes located within any fill material should be rubber ring jointed.
Source: Adapted from VicRoads (2003).
## Appendix C  Summary of Erosion and Sedimentation Control Techniques

### Table C 1: Summary of erosion and sediment control techniques

<table>
<thead>
<tr>
<th>Technique</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Roadway surface</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
| Crowning to ditch or sloping to single berm | Directs surface water to protected channel  
Minimises erosion.                      | None – should be part of good construction procedures.                                           |
| Compaction                             | The final lift of each day’s work should be compacted and bladed to drain to a ditch or berm. | None – should be part of good construction procedures.                                           |
| Aggregate cover                        | Minimises surface erosion.  
Permits construction traffic during adverse weather.  
May be used as part of permanent base construction. | Requires reworking and compaction if exposed for long periods.  
Loss of surface aggregates can be anticipated.                                                   |
| Seed/mulch                             | Minimises surface erosion.                                                 | Must be removed when pavement construction is commenced.                                           |
| **2. Roadway channels**                |                                                                             |                                                                                                  |
| Sediment traps/straw bale filters      | Can be located as required to collect sediment during construction.  
Clean-out can usually be done by the equipment onsite. | Little guidance on spacing and size.  
Sediment removal may be difficult.  
Specifications must include provisions for periodic clean-out.  
May require seeding, sodding or paving during final clean-up.                                     |
| Check dams                            | Maintains low velocities.  
Catches sediment.  
Can be constructed of logs, rock, timber, masonry or concrete. | Close spacing on steep grades.  
Requires clean-out.  
Unless keyed at sides and bottom, erosion may occur.                                              |
| Sodding                                | Easily placed, minimum preparation.  
Can be repaired during construction.  
Immediate protection.  
May be used on sides of lined channels to increase capacity. | Requires watering during first few weeks.  
Sod not always available.  
Will not withstand high velocity or severe abrasion from sediment load.                          |
| Seeding with mulch and matting         | Usually least expensive.  
Effective for channels with low velocities.  
Easily placed in small quantities by inexperienced personnel. | Will not withstand medium to high velocities.                                                      |
| Paving, riprap, rubble                 | Effective for high velocities.  
May be part of the permanent erosion control features. | Cannot always be placed when needed due to construction traffic and final grading and dressing.  
Initial cost is high.                                                                             |
<table>
<thead>
<tr>
<th>Technique</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Cutting slopes</td>
<td>Diverts water from cutting. Collects for slope drains or lined channels. May be constructed prior to excavation.</td>
<td>Access to top of cutting. Difficult to build on steep slopes or rock surfaces. Concentrates water and may require channel protection or energy dissipation devices.</td>
</tr>
<tr>
<td>Berm at top of cutting</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diversion dyke</td>
<td>Collects and diverts water at selected location reduces erosion potential. May be incorporated in permanent drainage system.</td>
<td>Access for construction. May be continual maintenance problem if not lined. Disturbed material or berm easily eroded.</td>
</tr>
<tr>
<td>Benching</td>
<td>Slows velocity of run-off. Collects sediment. Provides access to slope for seeding, mulching and maintenance. Collects water for slope drains or may divert to natural ground. Assists in establishing vegetation.</td>
<td>May cause sloughing of slopes due to water infiltration. Requires additional right of way. May not be possible in unsuitable material. Requires maintenance to be effective. Increases excavation quantities.</td>
</tr>
<tr>
<td>Slope drains</td>
<td>Prevents erosion of slope. Can be part of temporary or permanent system. Can be constructed or extended as excavation progresses.</td>
<td>Requires other structure to collect water. Permanent construction not always compatible with other project work. Usually requires some type of energy dissipation.</td>
</tr>
<tr>
<td>Seeding/mulching</td>
<td>Contributes to a grassed slope. Mulch provides temporary erosion protection until grass is rooted. Temporary or permanent seeding may be used. Mulch should be anchored. Larger slopes can be seeded in stages if smaller equipment is used.</td>
<td>Difficult to schedule high production units for small increments. Success depends largely on season. May require supplementary watering.</td>
</tr>
<tr>
<td>Sodding</td>
<td>Provides immediate protection. Can be used to protect adjacent property from sediment and turbidity.</td>
<td>Difficult to place until embankment is complete. Sod not always available. May be expensive.</td>
</tr>
<tr>
<td>Riprap, rock mattresses or hard sealing</td>
<td>Provides immediate protection for high risk areas and under structures. Sealing may be pre-cast or cast-in-situ.</td>
<td>Expensive. Difficult to place on high slopes. May be difficult to maintain.</td>
</tr>
<tr>
<td>Temporary cover (plastic sheeting, geotextiles etc.)</td>
<td>Easily placed and removed. Useful for providing some degree of protection for high risk areas.</td>
<td>Provides only temporary protection. Original surface usually requires additional treatment when cover is removed. Must be anchored to prevent wind damage.</td>
</tr>
</tbody>
</table>

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<table>
<thead>
<tr>
<th>Technique</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4. Embankment slopes</strong>&lt;br&gt;Berms at top of embankment</td>
<td>Prevents run-off running down face. Collects run-off for slope drains or channels. Can be placed as part of the normal construction operation.</td>
<td>Requires monitoring to ensure effective placement. Failure to compact properly results in failure of berm. Sediment build up.</td>
</tr>
<tr>
<td><strong>Slope drains</strong></td>
<td>Prevents run-off running down face. Can be full or half pipe, pre-cast sections, rock mattresses, or other materials. Can be extended as construction progresses. Can be either temporary or permanent.</td>
<td>Energy dissipator required at outlet. Removal of temporary drain may disturb growing vegetation.</td>
</tr>
<tr>
<td><strong>Embankment berms or benches</strong></td>
<td>Reduces velocity of slope run-off. Collects sediment. Provides access for maintenance. Collects water for slope drains. Can be used to spoil excess material.</td>
<td>Requires additional material if excess spoil is not available. May cause sloughing. Additional right of way may be needed.</td>
</tr>
<tr>
<td><strong>Seeding/mulching</strong></td>
<td>Can decrease slope exposure if applied at appropriate time. Mulch that is cut in or otherwise anchored will collect sediment.</td>
<td>Difficult to place until cutting is complete. Sod not always available. May be expensive.</td>
</tr>
<tr>
<td><strong>5. Protection of adjacent property – brush barriers</strong></td>
<td>Use slashing and logs from clearing operation. May be covered and seeded later. Eliminates need for burning or disposal of cleared material.</td>
<td>May be considered unsightly in urban areas.</td>
</tr>
<tr>
<td><strong>Straw bale barriers</strong></td>
<td>Bales readily available in most areas. When properly installed and maintained, they filter sediment and some turbidity from run-off.</td>
<td>Require removal. Subject to damage by vandals. Flow is slow through straw, requiring considerable area. May introduce unwanted species of vegetation.</td>
</tr>
<tr>
<td><strong>Sediment traps</strong></td>
<td>Collects much of the sediment from embankment slopes and channels. Inexpensive. Can be cleaned and expanded to meet need.</td>
<td>Do not remove all sediment and turbidity. Space not always available. Require constant maintenance. Usually need to be removed.</td>
</tr>
<tr>
<td><strong>Energy dissipators</strong></td>
<td>Minimises erosion away from project. Slows velocity, permits sediment deposition and collection downstream.</td>
<td>May collect debris. Require special design. Can be expensive. May be quite large structures.</td>
</tr>
<tr>
<td><strong>Level spreaders</strong></td>
<td>Converts concentrated channel or pipe flow back to sheet flow. Avoids channel easements and construction off project. Simple to construct.</td>
<td>Adequate space may not be available. Sodding of overflow is required. Must be part of permanent erosion control effort. Requires constant maintenance.</td>
</tr>
<tr>
<td>Technique</td>
<td>Advantages</td>
<td>Disadvantages</td>
</tr>
<tr>
<td>-----------------------------------------------</td>
<td>-----------------------------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>6. Protection of stream construction dyke</td>
<td>Permits work to continue during normal stream stages.</td>
<td>Usually requires pumping of seepage water out of the work site. Subject to erosion from stream and from direct rainfall on dyke.</td>
</tr>
<tr>
<td>Coffer dam</td>
<td>Work can be continued during most anticipated stream conditions. Clear water can be pumped directly back into stream. No material deposited in stream.</td>
<td>Expensive.</td>
</tr>
<tr>
<td>Temporary stream channel change</td>
<td>Prepared channel keeps flow away from construction.</td>
<td>Channel will usually require protection. Stream must be returned to old channel and temporary channel refilled when finished.</td>
</tr>
<tr>
<td>Riprap</td>
<td>Easy to stockpile and place. Can be installed in increments as needed.</td>
<td>Can be expensive.</td>
</tr>
<tr>
<td>Temporary culverts for haul roads</td>
<td>Minimises turbulence and turbidity. Provides uninterrupted route for fish. Normal flow can be provided by pipes, higher flows can pass over roadway.</td>
<td>Space not always available without conflicting with permanent structure work. Larger pipe sizes may be expensive. May be subject to washouts.</td>
</tr>
<tr>
<td>Rock lined low-level crossing</td>
<td>Minimises stream turbidity. Inexpensive. May also serve as a channel flow check or sediment trap.</td>
<td>May not be fordable during high flows. During periods of low flows, passage of aquatic life may be blocked.</td>
</tr>
</tbody>
</table>

*Source: VicRoads.*
Appendix D  Obtaining Rainfall Information (Australia)

Rainfall information is collected to:

- select a value for a design event ARI
- calibrate recoded rainfall intensities.

A very accessible way to obtain rainfall information is from the Bureau of Meteorology.

A set of accurate, consistent intensity-frequency-duration (IFD) design rainfall data has been derived for the whole of Australia. This work was done by the Bureau of Meteorology as part of the revision of AR&R Vol. 2 (Pilgrim 2007). Book 2, Section 1 of Vol. 1 (Pilgrim 2001) and details procedures for the construction of a set of IFD curves for a specific location. Vol. 2 (Pilgrim 2007) contains a series of maps of IFD design rainfall.

However, an on-line IFD data system is available on the Bureau of Meteorology website www.bom.gov.au.

First, obtain the latitude and longitude to identify the project area.

Below are some methods of sourcing latitude and longitude values:

1. From the survey source, request that the latitude and longitude be provided for the project.
2. Google earth is another option which can be found at www.google.com/earth/index.html.
3. From the World Wide Web http://itouch.com/latlong.html. To find the latitude and longitude of a point, click on the map, drag the marker, or enter the address.
4. IFD information is held at a resolution of about 2.5 km. It is therefore important to provide accurate location data and not just the name, or central location, of a large city such as Melbourne or Sydney (which would contain many grid points). The Geoscience Australia website: http://www.ga.gov.au/map/names/ is where a general place name and state may be entered and then the latitude and longitude read from the appropriate entry within the comprehensive list displayed.

Figure D 1:  Example from Geoscience Australia

The on line IFD data system

Below is the link to the IFD generator and an explanation of how the data is entered:

Tables can be exported to spreadsheet format, when using Adobe Flash version, or copy the table, and paste to text pad.

To import into Excel:
1. Save the file as a .csv (comma separated values) file. (e.g. myifdtable.csv).
2. Open excel and import the saved myifdtable.csv file.

Below is a link to Running the Program – Inputs/FAQs:

Figure D 2: IFD data system from BoM website

---

**Welcome to the Rainbow IFD Data System**

This system produces an Intensity-Frequency-Duration design rainfall chart and table between 5 minutes and 72 hours in duration and Average Recurrence intervals from 1 year to 100 years. A coefficient table is also produced which you can use to derive the results or interpolate for values between those given.

NEW: Calculate the Average Recurrence Interval for the chosen location, using a rain duration and total.

---

**Example Table:**

<table>
<thead>
<tr>
<th>YEARS</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
</tr>
</thead>
<tbody>
<tr>
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<td>2.8419933653</td>
<td>-5.13452834E-1</td>
<td>-7.00874117E-2</td>
<td>1.2903611E-2</td>
<td>2.9833287E-3</td>
<td>-8.182830E-4</td>
<td>2.247912E-5</td>
</tr>
<tr>
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<td>-4.4730081E-2</td>
<td>1.0891339E-2</td>
<td>0.7183956E-2</td>
<td>-7.055683E-3</td>
<td>5.896117E-5</td>
</tr>
</tbody>
</table>

*Note:*
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Appendix E  Estimation of coefficient of run-off – Queensland Application

E.1 Rural Hydrology

The run-off coefficient for the ARI 50 year event (C_{50}) is determined using Table E 1. To determine the value of C for other ARIs, the C_{50} value is modified using the factors from Table E 2.

It should be noted that this method can give a C_{50} value greater than 1.0. This can occur when rainfall intensity exceeds 120 mm/h and the remaining characteristics are at maximum values (possible in small, steep catchments). In this instance, C_{50} should be rounded down to 1.0.

The run-off coefficient C is a statistical composite of several aspects including the effects of rainfall intensity, catchment characteristics, infiltration (and other losses) and channel storage. It should not be confused with the volumetric run-off coefficient which is the ratio of total run-off to total rainfall.

Table E 1: Estimation of the run-off coefficient for rural catchments

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Run-off producing values (in brackets) as % in calculation of C for a 50 year average recurrence interval event</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rainfall intensity</td>
<td>(C) = 0.3 I_{50} + 4</td>
</tr>
<tr>
<td>Catchment relief</td>
<td></td>
</tr>
<tr>
<td>Very steep slopes &gt; 15%</td>
<td>(10)</td>
</tr>
<tr>
<td>Hilly to steep slopes 4–15%</td>
<td>(5)</td>
</tr>
<tr>
<td>Flat to rolling slopes &lt; 4%</td>
<td>(0)</td>
</tr>
<tr>
<td>Catchment storage</td>
<td></td>
</tr>
<tr>
<td>Well defined water courses,</td>
<td>(10)</td>
</tr>
<tr>
<td>negligible storage</td>
<td>(5)</td>
</tr>
<tr>
<td>Overland Flow is significant,</td>
<td>(0)</td>
</tr>
<tr>
<td>some floodplain storage</td>
<td></td>
</tr>
<tr>
<td>Poorly defined water courses,</td>
<td></td>
</tr>
<tr>
<td>large flood plain storage</td>
<td></td>
</tr>
<tr>
<td>capacity</td>
<td></td>
</tr>
<tr>
<td>Ground characteristics</td>
<td></td>
</tr>
<tr>
<td>Grazing land and open forest</td>
<td>(40)</td>
</tr>
<tr>
<td>Agricultural land</td>
<td>(30)</td>
</tr>
<tr>
<td>Dense vegetation and rainforest</td>
<td>(20)</td>
</tr>
<tr>
<td>Heath and sand dunes</td>
<td>(10)</td>
</tr>
</tbody>
</table>

Notes:
Catchment storage is defined as a catchment’s ability to detain or temporarily hold water within a stream’s adjacent floodplain. Water will slowly drain after flood water recedes.

Example:
Determine C_{50} for a rainfall intensity of 40 mm/h over a catchment with the following characteristics:
- Catchment relief – hilly with average slopes 4–8%.
- Catchment storage – well defined system of small watercourses with little storage capacity.
- Ground characteristics – open forest.

\[ C_{50} = \frac{16 + 5 + 10 + 40}{100} = 0.71 \]
### Table E 2: Adjustment factors for run-off coefficients for other average recurrence intervals

<table>
<thead>
<tr>
<th>Average recurrence interval (years)</th>
<th>Rural coefficient</th>
<th>Urban coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.8 $C_{50}$</td>
<td>0.80 $C_{10}$</td>
</tr>
<tr>
<td>2</td>
<td>0.8 $C_{50}$</td>
<td>0.85 $C_{10}$</td>
</tr>
<tr>
<td>5</td>
<td>0.8 $C_{50}$</td>
<td>0.95 $C_{10}$</td>
</tr>
<tr>
<td>10</td>
<td>0.8 $C_{50}$</td>
<td>1.00 $C_{10}$</td>
</tr>
<tr>
<td>20</td>
<td>0.9 $C_{50}$</td>
<td>1.05 $C_{10}$</td>
</tr>
<tr>
<td>50</td>
<td>1.0 $C_{50}$</td>
<td>1.15 $C_{10}$</td>
</tr>
<tr>
<td>100</td>
<td>1.05 $C_{50}$</td>
<td>1.20 $C_{10}$</td>
</tr>
</tbody>
</table>

**Notes:**
- $C_{50}$ determined for rural catchments using Section 6.6 – Rural Hydrology.
- $C_{10}$ determined using method described in Section 6.6.3 – Run-off Coefficient.
- Where run-off coefficients calculated using the above table exceed 1.00, they should be arbitrarily set to 1.00.

### E.2 Urban Hydrology

The run-off coefficient is calculated in accordance with the method summarised in the following steps:

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Step 1</td>
<td>Determine the fraction impervious $f_i$ for the catchment under study from Table 6.8 (fraction impervious v development).</td>
</tr>
<tr>
<td>Step 2</td>
<td>Determine the one hour rainfall intensity $I_{10}$ for the ARI 10 year event at the locality. See Section 6.7 – Urban Hydrology.</td>
</tr>
<tr>
<td>Step 3</td>
<td>Determine the 10 year C value from Table E 3 and Table E 4 ($C_{10}$ values for 0% fraction impervious).</td>
</tr>
<tr>
<td>Step 4</td>
<td>Determine the urban coefficient (frequency factor $F_Y$) for the required ARI from Table 6.2, if required.</td>
</tr>
<tr>
<td>Step 5</td>
<td>Multiply the $C_{10}$ as per Urban Coefficient (Step 4) to determine the run-off coefficient for the design storm $C_Y$.</td>
</tr>
</tbody>
</table>

In certain circumstances the resulting value of $C_Y$ will be greater than 1.0. In accordance with the recommendations of AR&R Vol. 1 (Pilgrim 2001) a limiting value of $C_Y$ = 1.0 should be adopted for urban areas.

There is little evidence to support an allowance for either slope or soil type in fully developed (non-WSUD) urban areas. If there are significant local effects, and reliable data is available, then adjustments for soil type may be incorporated within the calculations at the discretion of the designer in consultation with the relevant local authority.

The relationships shown in AR&R Vol. 1 (Pilgrim 2001) and adopted in this Guide apply to areas that are essentially homogeneous, or where the pervious and impervious portions are so intermixed that an average is appropriate. In cases where separable portions of a catchment are significantly different, they should be divided into sub-catchments and different values of C applied.

Notwithstanding the above notes and limitations, it is the responsibility of the designer to ensure each sub-catchment flow is determined using a suitable coefficient of discharge. The local government body may set specific C values to be used within their area.
### Table E 3: C₁₀ values

<table>
<thead>
<tr>
<th>Intensity (mm/h) 1₁₀</th>
<th>0.00</th>
<th>0.20</th>
<th>0.40</th>
<th>0.60</th>
<th>0.80</th>
<th>0.90</th>
<th>1.00</th>
</tr>
</thead>
<tbody>
<tr>
<td>39–44</td>
<td>0.44</td>
<td>0.55</td>
<td>0.67</td>
<td>0.78</td>
<td>0.84</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>45–49</td>
<td>0.49</td>
<td>0.60</td>
<td>0.70</td>
<td>0.80</td>
<td>0.85</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>50–54</td>
<td>0.55</td>
<td>0.64</td>
<td>0.72</td>
<td>0.81</td>
<td>0.86</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>55–59</td>
<td>0.60</td>
<td>0.68</td>
<td>0.75</td>
<td>0.83</td>
<td>0.86</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>60–64</td>
<td>0.65</td>
<td>0.72</td>
<td>0.78</td>
<td>0.84</td>
<td>0.87</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>65–69</td>
<td>0.71</td>
<td>0.76</td>
<td>0.80</td>
<td>0.85</td>
<td>0.88</td>
<td>0.90</td>
<td></td>
</tr>
<tr>
<td>70–90</td>
<td>0.74</td>
<td>0.78</td>
<td>0.82</td>
<td>0.86</td>
<td>0.88</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>

₁₀ = One-hour rainfall intensity for an ARI 10 year event & C₁₀ = Run-off Coefficient for an ARI 10 year event

### Table E 4: C₁₀ values for 0% fraction impervious

<table>
<thead>
<tr>
<th>Land description</th>
<th>Dense bushland</th>
<th>Medium density bush, or good grass cover, or high density pasture, or zero tillage cropping</th>
<th>Light cover bushland, or poor grass cover, or low density pasture, or low cover bare fallows</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intensity 1₁₀</td>
<td>Soil permeability</td>
<td>Soil permeability</td>
<td>Soil permeability</td>
</tr>
<tr>
<td></td>
<td>High</td>
<td>Med</td>
<td>Low</td>
</tr>
<tr>
<td>39–44</td>
<td>0.08</td>
<td>0.24</td>
<td>0.32</td>
</tr>
<tr>
<td>45–49</td>
<td>0.10</td>
<td>0.29</td>
<td>0.39</td>
</tr>
<tr>
<td>50–54</td>
<td>0.12</td>
<td>0.35</td>
<td>0.46</td>
</tr>
<tr>
<td>55–59</td>
<td>0.13</td>
<td>0.40</td>
<td>0.53</td>
</tr>
<tr>
<td>60–64</td>
<td>0.15</td>
<td>0.44</td>
<td>0.59</td>
</tr>
<tr>
<td>65–69</td>
<td>0.17</td>
<td>0.50</td>
<td>0.66</td>
</tr>
<tr>
<td>70–90</td>
<td>0.18</td>
<td>0.53</td>
<td>0.70</td>
</tr>
</tbody>
</table>

Appendix F  Estimation of Discharge from Rural Catchments in Victoria – Use of Area Size Factor

F.1  General

VicRoads has carried out research that indicates that peak discharges might be underestimated in certain conditions for small rural catchments in some Victorian locations (less than 5000 hectares). An area size factor is therefore applied to determine peak discharges for rural catchments.

This appendix outlines the procedure to determine peak discharge incorporating the area size factor. It is likely that the requirement for use of the area size factor in Victoria will be reassessed when the Australian Rainfall and Run-off review has been finalised (anticipated in 2013).

The procedures set out in this appendix are based on a statistical interpretation of the Rational Method, with design discharge factors determined by geographical location.

The method for rural catchments is not applicable:

1. where the total catchment area is less than 1 km² (100 hectares)
2. where comparisons are required between surfaces with different imperviousness, before and after proposed development
3. where there are flood plains with significant storage and where flow may spill over sub-catchment boundaries other than via the main channel or may be restricted by backing up from a downstream catchment
4. where detention basins are to be, or have been introduced.

In cases (1) and (2), the discharge estimation method in Section 6.7 – Urban Hydrology, should be used. In cases (3) and (4), a flood routing method is required.

F.2  Victorian Conditions

Note: Data applies to Victoria only.

Rainfall intensity/duration charts were calculated by the method given in Pilgrim (2001). Run-off coefficients were based on flood discharge data supplied by the Rural Water Commission from 325 gauged catchments having 10 or more years of continuous discharge records. The sizes of gauged catchments areas ranged between 1800 ha. and 264 500 ha. The run-off coefficient for rural catchments less than 5000 ha is increased by an area size factor, \( F_A \).
F.3 Storm Run-off Estimation

The peak flow for rural catchments is estimated by Equation A5:

\[ Q_Y = \frac{P_Y I_Y A}{360} \tag{A5} \]

where

- \( Q_Y \) = Expected maximum discharge for the \( Y \) year average recurrence interval (ARI) (m³/s)
- \( P_Y \) = Design discharge factor
- \( I_Y \) = Average intensity of rainfall (mm/h) for the \( Y \) year ARI interval during the catchment characteristic time \( t_k \)
- \( A \) = Catchment area (ha)

The formula does not represent the pattern of any particular storm event, as the times of rise, peak flows and durations of flow resulting from storms in a catchment can vary significantly. Rather, it is a means of quickly estimating a design discharge from the catchment for the design ARI. The designer should keep in mind that actual peak flows could vary from the estimate by plus or minus 30%.

F.4 Average Recurrence Interval

The ARI is defined as the average interval in years between the occurrence of a specified discharge and an equal or larger flow. However, it should be noted that there can be a significant probability of this discharge occurring in any one year. For example, although a discharge of 10 year ARI could be equalled or exceeded in a 10 year period, there is approximately a 1 in 10 probability that it could be exceeded in any one year.

Any culvert or drainage system will occasionally have to carry flows greater than the design discharge. Appropriate provision must be made for control of the excess.

When selecting the ARI for a design, the following factors should be considered:

- the consequences of flooding, such as damage to property, road, and structures
- traffic delays or extra travel distance due to road closure during floods
- road maintenance costs
- the additional cost of providing for a longer ARI.

Table 4.3 provides a guide to the selection of ARIs for flood immunity.

The ARI to be used for final design would be selected after evaluation of the factors listed above.

F.5 Definition of Catchment Area


The designer is advised to inspect the site to check the details of the area against the catchment as interpreted from the map.
Unmapped shapes less than the height of the contour interval, or features added during property development, can alter contributing areas and drainage paths.

In flat country, a site inspection may not reveal all the relevant information, and stereoscopic interpretation of aerial photographs, or catchment definition by photogrammetry could be helpful.

**F.6 Calculation of Characteristic Time**

It is necessary to calculate a typical run-off response time, called the characteristic time $t_k$ before selecting a design value of rainfall intensity.

For rural catchments, the characteristic time is calculated from Equation A6:

$$ t_k = 7.924A^{0.38} \text{ minutes} \quad \text{A6} $$

where

$$ A \quad \text{Catchment area (ha)} $$

It is important that this time be the same as that used to derive the design discharge factor, PY. It is incorrect to add a time of overland flow to the characteristic time.

**F.7 Average Rainfall Intensity**

Rainfall intensity-frequency-duration tables can be produced for any location in Victoria from the Australian Bureau of Meteorology website. Input data required to generate a table are the latitude and longitude for the city, town or location of interest.

**F.8 Design Discharge Factor**

The 10 year ARI design discharge factors for rural catchment areas greater than 5000 hectares in Victoria, except the region between Horsham and Mildura where there are insufficient stream gauging records, are shown on Figure F 1(a) and Figure F 1(b). The design discharge factor decreases with area and increases with ARI and is calculated from Equation A7:
\[ P_Y = P_{10} F_Y F_A \]  

where

\[ P_Y = \text{Discharge factor for } Y \text{ year ARI} \]
\[ P_{10} = 10 \text{ year ARI factor read from Figure F 1(a) and Figure F 1 (b)(rural catchments only)} \]
\[ F_Y = \text{Frequency factor read from Table 6.3} \]
\[ F_A = \text{Area size factor, which may be read from Figure F 2 or calculated from} \]

\[ F_A = 1.0 \text{ (} F_A \text{ not applicable, if } A > 5000 \text{ ha)} \]
\[ F_A = [1.6 - 0.6(A - 1000)/4000], \text{ if } 1001 < A < 5000 \text{ ha} \]
\[ F_A = [2.1 - (A/2000)], \text{ if } 301 < A < 1000 \text{ ha} \]
\[ F_A = 2.0 \text{ approx., if } 0 < A < 300 \text{ ha} \]
Figure F 1(a): Design discharge factor

Figure F 1 (b): Design discharge factor

**DESIGN DISCHARGE FACTOR**

**10 YEARS AVERAGE RECURRENCE INTERVAL**

*Source: VicRoads (2003).*
Figure F 2: Area size factor ($F_A$)

Austroads’ Guide to Road Design Part 5: Drainage – General and Hydrology Considerations provides road designers and other practitioners with guidance on the design of drainage systems. This Guide provides information on the elements that need to be considered in the design of a drainage system. Guidance is provided on the safety aspects of stormwater flows, environmental considerations and water sensitive treatments within a drainage system.