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National Strategic Research Program

The collection and discharge of stormwater from the road infrastructure

Allan Alderson

Research Report ARR 368

ARRB GROUP

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Abstract:

The document addresses management of stormwater runoff from roads including water quality issues. An overall view of stormwater management, including the concept of major and minor drainage systems is first presented.

The development of runoff estimates, developing a major drainage system, developing a minor drainage system is discussed and concludes with some information on treatment of road runoff before entering receiving waters.

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Allan Alderson joined ARRB Group Ltd in 1990 with a Bachelor of Engineering with honours from Swinburne Institute of Technology and has since gained a Master of Engineering Degree from RMIT.

Allan is a member of the National Asphalt Research Co-ordination (NARC) Group and National Bituminous Surfacings Research Group (NBSRG). He has had a substantial role in the development of national guidelines for the selection, testing and design of asphalt mixes and sprayed seals. Previously, Allan has been concerned with testing geotechnical materials and geotechnical investigations.

He was appointed an assessor for the National Association of Testing authorities (NATA) in 1996 and is the author of numerous papers and reports. Allan's research expertise is in the areas of materials characterisation using laboratory based techniques, sprayed seal design, asphalt mix design, training of staff in new technology related to asphalt mix design and PC based data acquisition and analysis.

PREFACE

This Guide has been prepared to provide advice on the collection and discharge of stormwater runoff from road reservations and associated infrastructure in both rural and urban areas. The Guide takes precedence over the documents listed below:

- Storm drainage design in small urban catchments: A handbook for Australian practice, Australian Road Research Board Special Report No. 34 by J Argue (1986).
- Subsurface drainage of road structures, Australian Road Research Board Special Report No. 35 by R J Gerke (1987).
- Guide to the design of road surface drainage, NAASRA (1986).

Australian Rainfall and Runoff (2001 edition) is considered to be the overarching document in Australia on the subject of stormwater. Other national documents that are complementary to, and should be read in conjunction with, this Guide are:

- Austroads 1994 publication ‘Waterway design: A guide to the hydraulic design of bridges, culverts and floodways’.
- Austroads 2003 publication ‘Guidelines for treatment of stormwater runoff from the road infrastructure’.

Where disparities exist between this document and Australian Rainfall and Runoff (2001 edition), then Australian Rainfall and Runoff (2001 edition) takes precedence.

In New Zealand, a Land Transport New Zealand research report “Integrated Stormwater Management Guidelines for the New Zealand Roading Network” (2004) provides guidance on a range of issues relating to the management of stormwater run-off from state highways and local roads in New Zealand.

The Guide is produced both for students who are new to stormwater management principles, and as a reference for experienced practitioners, and includes new information relating to environmentally sensitive disposal of road runoff.

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CHAPTER 1

1 INTRODUCTION

1.1 Scope

This Guide provides advice on the management of stormwater runoff from road reservations and associated infrastructure. The management of stormwater runoff involves two key components:

1. The management of stormwater on the road pavement, to the point where the water has been satisfactorily collected at the roadside; and
2. How the collected water is dealt with until it is disposed of at an outfall point.

Drainage systems need to be developed as part of an overall master drainage plan for a clearly defined drainage catchment that includes the road. This Guide supports the concept of major and minor drainage networks, which together provide an integrated drainage system within a drainage catchment.

The five parts of the master drainage plan are:

1. **Major / minor drainage networks:** storm drainage systems for existing development should be shown together with systems for proposed or likely 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.
2. **Development plan:** plans of all existing and proposed developments should be shown and their relationships to the major / minor networks and present or likely runoff characteristics, both in quantity and quality.
3. **Stormwater retention:** infiltration and / or detention measures: these should be identified for each sub-catchment and group of such areas and should take account of land-use, drainage networks, terrain and soil characteristics. The retention, infiltration, and detention measures adopted in each sub-catchment should aim to retain, where appropriate, as much incident storm rainfall as possible.
4. **Sediment and erosion control measures:** these should be incorporated into the planning, design and construction phases of a drainage scheme to minimise soil loss and ensure minimum downstream environmental damage from water-borne sediment.
5. **Pollution control strategy:** this strategy should be matched to the particular land use, drainage and runoff quality characteristics experienced or likely to be experienced in the catchment in the course of future development. The purpose of the strategy is to ensure that runoff entering receiving waters from the catchment meets acceptable water quality criteria.

Computer programs can be used at many stages in the management of stormwater and are in common use. It is not within the scope of this manual to discuss specific propriety software. However, the application of such software needs to be considered in terms of:

- Is the computer model appropriate for the situation?
- Have the input parameters been selected to represent the actual conditions?
- Is the predicted result reasonable?

The larger the project the more necessary it becomes to use computer modelling. Unfortunately, it is quite easy to make gross errors when using computer software through the input of incorrect parameters or applying incorrect assumptions. Errors can be very difficult to detect and results should always be carefully scrutinised.

1.2 Management Framework

Overall planning for water supply and drainage is usually under the control of a Catchment Management Authority (CMA) (where it exists or other regulatory body). In New Zealand, regional and territorial authorities consider resource consent requests for activities that affect water or stormwater. Regional Council Policy Statements, Regional and District Plans or Integrated Catchment Management Plans contain regional and local water and stormwater objectives.

State Road Authorities may have limited powers with respect to drainage, and schemes for road drainage need to comply with an overall strategy and may be subject to the approval of the CMA. The CMA or the Environment Protection 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.

The basic objectives for the management of stormwater collected in road reserves are:

- water from the road reserve shall be discharged in a manner that does not cause nuisance or damage to the adjacent landowners or occupiers
- water on the road reserve shall be managed to allow safe passage of motorists, bicyclists and pedestrians
- stormwater shall be appropriately treated before discharge.

In some States, the State Road Authority has been provided with powers by legislation to construct and maintain drains and watercourses that are used for the drainage of roads. However, it is required to comply with environmental and planning legislation specific to the State. This is embodied in the principles taken from a duty of care statement pertaining to South Australia:

"A person must not undertake an activity that pollutes, or might pollute, the environment unless the person takes all reasonable and practical measures to prevent or minimise any resulting environmental harm."

Most government bodies have a similar statement.

1.3 Design Considerations

The design of drainage systems can not be undertaken in isolation but needs to be developed with consideration for other engineering and social factors. Any drainage system should be incorporated into a master drainage plan (if one exists) for the drainage catchment.

In addition to complying with an overall drainage goal, a drainage scheme must account for differences in climate, adjacent land use and any environmental issues. These are discussed in Chapter 2 — Design Considerations.

1.4 Major / Minor Drainage Systems

A major / minor system approach is adopted for the planning and design of urban stormwater systems. The minor system is intended to collect and convey runoff from frequent storm events such that nuisance flooding is minimised. The major system is intended to safely convey runoff which is in excess of the capacity of the minor drainage system to receiving waters. The major / minor concept may be described as a 'system within a system,' since it comprises two distinct but interlinked drainage networks, as shown in Figure 1.1.

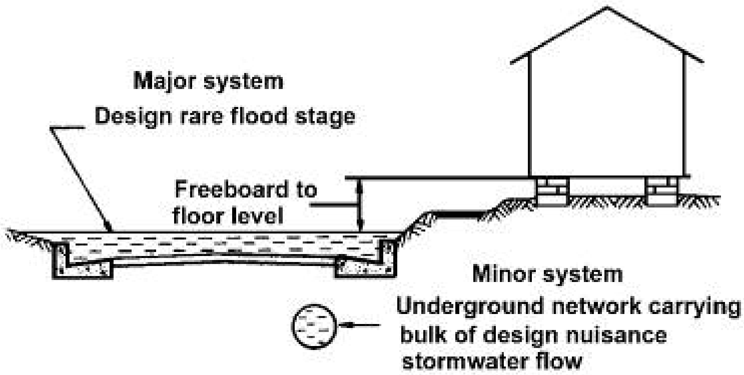


Figure 1.1: Major and minor drainage systems in the urban landscape (after ARRB Special Report 34, 1986)

The major system ensures that floodwater inundation of residential, commercial / industrial, and important public buildings occurs only very infrequently. During such events, the velocity and / or depth of flood waters in all readily accessible open channels are below prescribed limits to provide a satisfactory level of safety and security in communities. The major system typically consists of a network of overland flow paths including roads, natural channels and streams, engineered waterways, culverts, community retention / detention basins and wet and dry ponds, pumping installation and flood gate / tidal gate, which ultimately discharge into receiving waters.

The minor system provides convenience and safety for pedestrians and traffic in frequent or nuisance stormwater flows. The minor system typically consists of a network of kerbs, gutters, inlet structures, open drains and underground pipes, and on-site detention / retention facilities.

The aim is for the total drainage scheme to retain within each catchment as much incident rainfall and runoff as is possible and appropriate, given the planned use of the catchment terrain and its biotic and engineering characteristics. This is identified with the retention, infiltration and detention measures included in master drainage planning which are aimed at reducing the negative impacts of urban development on indigenous flora and fauna and predevelopment groundwater levels without loss of structural integrity of adjacent infrastructure.

The implementation of a major system is likely to pose a more difficult problem than a number of minor systems within a catchment area. This is in part due to the greater flows involved than are likely to be encountered in the design of its ‘nested’ minor system. For these reasons a major-then-minor design sequence is recommended.

CHAPTER 2

2 DESIGN CONSIDERATIONS

2.1 Introduction

The design of road drainage systems needs to fulfil two basic criteria:

- the need to provide a satisfactory level of service to road users
- minimising the impact on the environment.

When designing road surface drainage, the following factors need to be addressed:

- the nature of the catchment
- rainfall characteristics pertinent to the catchment
- determination of the climatic zone
- the consequences of the exceedence of the design flow of the drainage system
- the design average recurrence interval, for both minor and major drainage system
- consideration of environmental impacts.

The level of detail required will depend on the nature of the project.

Road drainage cannot be considered in isolation from the surrounding catchment, and the involvement of other stakeholders usually is required.

2.2 Catchment Considerations

2.2.1 Introduction

An assessment of the existing conditions should be performed well in advance of the design of the drainage scheme. This will determine the type of drainage scheme appropriate to the conditions, siting of infrastructure, the risk of erosion, and the potential for turbidity of the discharge into the receiving waters. Designers should also note whether any factors external to the catchment need to be taken into account such as whether downstream facilities can cope with any proposed additional runoff generated by the project. Factors that need to be considered during the initial assessment include:

- catchment area
- land use
- existing drainage infrastructure (including the capacity of outfalls)
- the potential impact on flora, fauna, sites of historical or cultural importance, etc.
- catchment retention or detention characteristics
- other potential stakeholders.

Some of the factors listed above would be covered in an environmental impact statement (EIS) (see also Austroads, 2003b) or assessment of environmental effects (AEE) and it is beyond the scope of this document to provide guidance on preparing such a document. Importantly, issues identified in an EIS or AEE need to be addressed in the design and construction of road drainage systems.

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

2.2.2 Catchment Area

The catchment boundary needs to be clearly defined by 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 runoff characteristics.

In most cases, catchments will have to be divided into sub-catchments because of the requirements of a hydrological model, or so that each sub-catchment is uniform in regards to runoff characteristics.

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 all of Australia and New Zealand and for most urban areas these are quite detailed. Some areas are also covered by digital maps. Whichever is used, drainage designers should seek the most current and the most detailed. Stereoscopic review of aerial photographs may reveal features not identified on contour maps, particularly in flat areas.

2.2.3 Current State of Development

Land Use

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 per cent of storm rainfall (but can be up to 70% in tropical areas) on a natural catchment is discharged as surface runoff. 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.

Rainfall that enters the groundwater storage may appear as dry weather flow in streams. Groundwater reserves support trees and shrubs in the catchment particularly in time of drought and provide dry weather flow upon which the stream ecosystem depends.

Urban development reduces the amount of vegetative cover and introduces significant impervious areas such as roofs, roads, car parks and concrete paving. This reduces the level of infiltration and evapo-transpiration, resulting in greatly increased runoff. The increased runoff can increase erosion from the land, and may cause the channel of the receiving stream to erode as the natural balance has been disturbed. Modern drainage schemes attempt to limit the discharge from catchments using infiltration techniques or peak discharge reduction using on-site detention facilities.

Urban development is also likely to increase the levels of chemical and other pollutants. The type of development will determine the likely type and quantity of pollutants. Runoff from areas near shopping centres are known to contain relatively high concentrations of gross pollutants, this in turn requires certain drainage elements to be provided within the drainage scheme (e.g. gross pollutant traps).

Urban land values are higher compared to those of rural land, with the result that urban stormwater management strategies must be space efficient. The area required for treating water in ponds and wetlands may not be available so a reduced level of treatment may be accepted.

Agricultural activity should be considered. Intense agricultural uses, such as market gardens, also increase erosion due to regular tillage, and irrigation. They create water pollution due to the use of fertilizers and insecticides. This may influence what water treatments are possible and what pollutant loads are likely to be contained in any runoff.

Land use adjacent to the road reserve should also be examined with respect to the level of flood protection required. Hospitals and other emergency services buildings are provided with a greater protection to flooding than open recreation areas. This is discussed in detail in Section 4.3.

Predictions of future developments may need to be considered (see Section 2.2.4).

Existing Infrastructure

Existing drainage infrastructure within the project site 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 be identified.

Where the project lies within an existing urban area, the standard of the minor drainage system may have to match the existing standard, with the major system being designed to cater for the gap flow. This may improve the overall level of flood protection above the existing level.

2.2.4 Future Developments

Where proposals for future development of the catchment exist, there may be a legal requirement to increase drainage facilities above that required to meet existing conditions, during initial construction. If there are no proposals for further development of the catchment at the time of initial construction, then developers may not be legally required to provide additional capacity above that needed to cater for existing conditions. The actual legal implications may vary between jurisdictions and should be checked.

In deciding whether to provide additional capacity above that required to meet existing demand, the following should be borne in mind:

- disruption to traffic during reconstruction at a later date
- whether an increase in upstream water levels due to back water will be possible at a later date; or
- 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. Little firm guidance can be offered for this task. The catchment drainage plan should be integrated with 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 / 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.

Rural catchments are also often stable for long periods. However, when changes occur to rural catchments these can often be dramatic, e.g. clear felling or a complete change of agricultural activity. It is difficult to asses the ultimate condition of a rural site, since the nature of the catchment land use is influenced by economic and political pressures that are difficult to foresee.

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 town planning schemes, proposed road networks, the Natural Resources Atlas, Australian Local Government Association, individual city and region planning strategies, etc.).

In its 'ultimate' state, a catchment, particularly in urban areas, will have evolved, almost certainly, 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, gutter 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.

The drainage designer may not have to design for an ultimate developed condition. It may be more appropriate to design for an assumed condition, provided that the appropriate development authority is aware of the implications, and is able to ensure that development is kept to the agreed condition.

2.2.5 Catchment Retention or Detention Characteristics

Some simple runoff 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 runoff 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 use existing facilities such ponds and wetlands.

2.2.6 Special Considerations

An investigation of the project area should identify any sensitive areas containing either unique flora, fauna, areas of special historical or cultural significance. These may affect the design of drainage infrastructure, particularly the nature and siting of the infrastructure and treatment devices. Each State will have legislative requirements covering these aspects and drainage designers should understand their obligations. Road authorities may also have their own objectives for stormwater management, which drainage designers must give effect to.

2.3 Rainfall Data

2.3.1 Introduction

This is a specialised topic and the procedures in ARR Vol. 1 (Ed. 2001) should be adopted. The procedures in ARR (Ed. 2001) provide a method of statistically estimating the rainfall for any area within Australia. In New Zealand, the National Institute for Water and Atmospheric Research (NIWA), regional councils and many territorial local authorities maintain up-to-date rainfall records.

However, it is possible to devise runoff characteristics based on data gathered at the site or in catchments similar to that under consideration. The resources needed to analyse historical rainfall / runoff / stream flow data are high, and require expertise not normally available to all but major projects. ARR (Ed. 2001) provides guidance on how this can be accomplished.

2.3.2 Intensity Frequency Duration

These three parameters are used to define rainfall events. ARR (Ed. 2001) provides a means of determining the likely rainfall pattern for any area in Australia and is recommended unless catchment data has been analysed.

All three of these parameters are inter-linked and are:

- intensity – the rate at which the rain fall occurs and is measured in millimetres per hour. 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. A very severe storm of high intensity would occur less frequently than a storm event consisting of light rain. Frequency is usually expressed as the average recurrence interval (ARI) or annual exceedence probability (AEP)
- duration – how long the storm event lasts.

Storm frequency is usually fixed by the drainage designer and is a function of the type of facility being designed and the consequences of the facility not coping with discharge. Evaluation of risk needs to be considered (see Section 2.5). Typically, minor drainage elements are designed for frequent storm events and major drainage elements are designed to cater for storms that occur less frequently.

Storm duration is largely dictated by catchment characteristics. The question that needs to be answered is, what storm duration produces the most severe conditions? Will a short intense storm or a prolonged less intense storm result in the greatest peak discharge?

Storm intensity then can be determined form the procedures in ARR (Ed. 2001) using the duration and frequency determined by the drainage designer as appropriate to the situation. Rainfall characteristics are discussed in greater detail in Chapter 3 — Obtaining Design Flows.

2.3.3 The influence of climate change

The capacity of stormwater systems to meet performance criteria may be affected by prediced chages in climate trends such as changes to mean annual rainfall, frequency and intencity of heavy rainfall events and sea level rise.

Considering the potential impacts of these trends during stormwater design may influence decisions about the location and capacity of stormwater infrastructure and the need for contingency measures, such as secondary flow capacity or the ability to upgrade facilities in the future.

2.4 Assessing Risk of Exceedence

2.4.1 Introduction

This section gives a brief overview of risk assessment and how it may be applied to stormwater management. Readers are advised to seek out more detailed information for specific circumstances (see AS/NZS 4360 and any additional guidance provided by road authorities on risk management).

The values given in Tables 4.2 and 6.4 for the ARI design standards should be treated with some caution. They represent design standards that may not take into account the individual circumstances. Increased flows in natural watercourses from stormwater runoff can have devastating effects on downstream flora, fauna and local amenity. Increasing water levels upstream as a result of a drainage scheme (afflux) can have a severe effect on adjacent properties. Where necessary, potential problems due to alterations to upstream and / or downstream flow behaviour should be considered during the design of drainage schemes.

For example, consider a simple culvert planned for a minor road crossing. Normally this would be a minor design task, but immediately upstream of this particular site is a large nursing home. The impact of the proposal on upstream water levels therefore becomes a matter of concern and the effort expended on hydraulic modelling must be commensurate with the risk associated with the consequences of upstream flooding.

Risk assessment goes beyond this simple example and must consider the risks and associated costs or benefits involved in providing a level of service to the community. Not every drainage scheme will require a risk management study as the resources and expertise required can be high.

Two methods are offered and comprise:

- least cost analysis
- risk factor analysis.

2.4.2 Least Cost Analysis

One approach is to undertake a least cost analysis. In this approach, a range of floods is examined and the predicted outcomes are related to costs to the community.

Increasing the level of flood immunity increases initial construction costs, and often requires higher ongoing maintenance. A high level of flood immunity reduces the costs to the community due to surcharging of the facility (i.e. flood volume exceeding design capacity) due to low frequency of exceedence and lower volumes exceeding capacity. Designs to cater for storms of lower ARI (more frequent flooding) decreases the construction and maintenance costs but increases the cost to the community due to capacity being exceeded more often and by being exceeded by greater volumes. The total cost is then correlated to the various storm ARIs and the least total cost provides the design ARI for the facility. The idealised concept is shown in Figure 2.1.

One of the main problems in use of the least cost analysis is quantifying the costs to the community of flooding. The community costs should reflect the social, commercial and environmental costs incurred. Most Levels of government have some form of sustainable development group or policy and these may provide guidance on community costs and benefits.

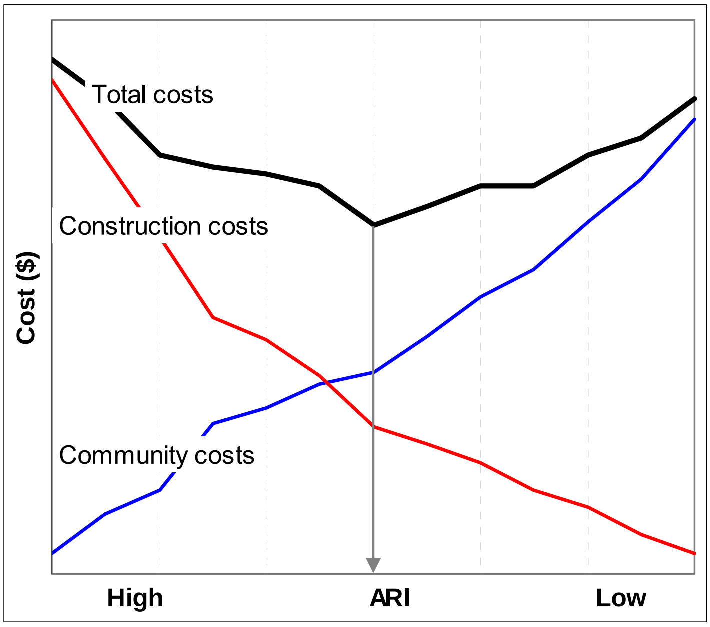


Figure 2.1: Idealised least cost analysis (based on RTA, 1999)

2.4.3 Risk Factor Analysis

Risk factor analysis involves setting appropriate assessment criteria, definition of the key elements and the risks associated with each of the elements. A number of key elements are identified in Table 2.1. However, due to the widely varying circumstances involved in provision of drainage schemes, it is not possible to provide a comprehensive list for all situations and eventualities. The elements in Table 2.1 should be viewed as typical of those that may be applied, and not considered an exhaustive list.

The process of risk assessment assigns factors associated with the likelihood of an event occurring and the consequences of that event for each of the key elements. The likelihood factor (LF) is often adopted as 1/ARI, i.e., for a design using an ARI of 20 years the likelihood factor would be 1/20 = 0.05. The consequences for each element would then be considered and assigned a factor. A typical range of consequence factors is shown in Table 2.2.

Table 2.1: Typical risk management considerations

Assessment Criteria	Key Elements	Risks
Duration of road closure Frequency of road closure Etc.	Safety	Injury to users, infrastructure elements damaged to unsafe levels
	Economic costs due to disruption	Increased user costs, damage to vehicles
	Social costs due to disruption	Community concern
	Restoration costs to drainage infrastructure	Repair or replacement costs
	Flood damage costs to others	Inundation of adjacent land
	Cost to environment	Loss of amenity, flora or fauna. Increased pollutant load or turbidity
	Impact on downstream and upstream water ways	Alteration to stream flows, siltation, scour
	Construction difficulties	Delays to work programs
	Maintenance	Higher maintenance costs
	Etc.	Etc.

Table 2.2: Consequence factors (CF)

Consequence Factor	Critical	Major	Serious	Moderate	Minor	Negligible
	0.9	0.7	0.5	0.3	0.1	0.0

A risk factor can then be calculated for each key element using Equation (2.1). The risk factors for all key elements are then summed and plotted against the costs in providing the infrastructure to each level of ARI (likelihood level). The costs and the risk factors for each ARI scenario are summed and expressed as the total for all scenarios examined. The lowest point of the 'total costs' line is when the construction costs are balanced against the risk. The plot is similar to a least cost analysis except the community cost line is replaced with a risk factor line (see Figure 2.2). The total cost curve should also incorporate ongoing costs such as maintenance costs, recycle value, etc.

$$RF = LF + CF - (LF \times CF) \qquad \text{Equation (2.1)}$$

where

CF consequence factor

LF likelihood factor

RF Risk factor

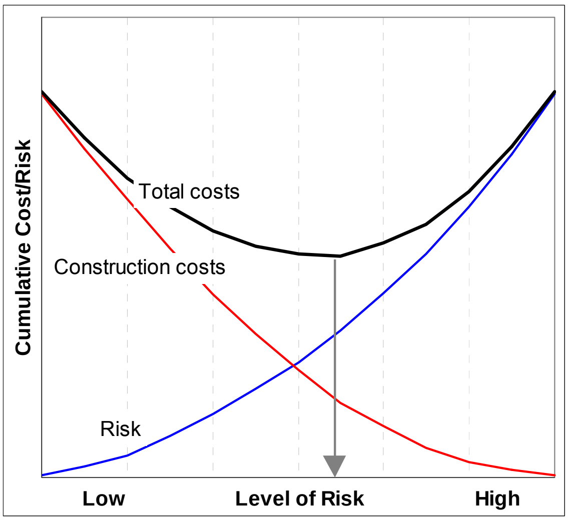


Figure 2.2: Idealised risk factor concept

2.5 ARI for Major and Minor Systems

2.5.1 Introduction

The standard of inundation protection required for the various elements of a drainage network depend on the consequences of inundation and the cost of providing the drainage elements as discussed in the previous section. In general, minor drainage elements are designed to cater for relatively frequent storms and any excess is then conveyed in the major system causing some disruption to users and the local community. The capacity of the major system is only infrequently exceeded and the associated storm events are of such high intensity that the community would be expecting some inconvenience.

2.5.2 Minor

Minor drainage system elements typically convey surface runoff and sub surface moisture away from the pavement and in their normal operation do not impinge on the user or adjacent community. Typically a minor drainage consists of elements such as:

- kerb and channel
- pits and inlets
- underground pipe networks
- surface channels (e.g. cut off drains, table drains, and the like)
- retention, detention, sedimentation, infiltration and water quality facilities.

The ARI for each element varies depending upon circumstances and design constraints but can range from less than one year to over 10 years. It should be noted that the design ARI range suggested is only applicable to situations where a major drainage complements the minor drainage system.

2.5.3 Major

Major drainage elements are only used when the capacity of the minor drainage system is exceeded. This implies that when called into use, there will be some disruption to the level of service provided by the road and to the adjacent communities. Typical elements include:

- the roadway
- designated surface channels
- some retention and detention facilities.

Flood paths to designated outfalls need to be investigated and the inundation of adjacent land estimated.

2.5.4 Regional Approach

It can be advantageous to consider individual drainage schemes in relation to a regional drainage plan. A thorough analysis of the regional flood estimates can be calculated on a one-off basis and reviewed periodically. This will reduce the need to fully examine the flood estimates on a project by project basis. The regional approach permits:

- efficient use of resources in that only a single major hydrologic analysis is required per region
- identifying areas of major risk
- provide a framework in which minor drainage schemes contribute towards an overall objective.

2.6 Environmental Issues

Some of the environmental impacts associated with road construction and operation that can be mitigated by stormwater management include:

- erosion and sedimentation during construction (refer to Section 9.2.7 Temporary Works)
- erosion and sedimentation from the modification of existing surface channels
- pollution of receiving waters by heavy metals (refer to Austroads, 2003b)
- pollution of receiving waters by hydrocarbons (refer to Austroads, 2003b)
- accidental toxic spills (refer to Chapter 8 – Water Quality & Erosion)

- the formation of a barrier to wildlife, including fish
- effects of changes to hydrology paths and flow rates on habitat modification.

2.6.1 Modification of existing surface channels

Discharging road run-off into existing waterways may increase flow volumes and velocities, resulting in erosion and adding sediment and silt to waterways. This in turn may disturb habitats for instream and riparian communities and may cause loss of species unsuited to modified conditions.

Receiving environments may be sensitive to these effects, due to the nature of biological communities or the nature of soils. Alternatively, the scale of run-off from construction or the operation of a roadway may cause adverse effects. Works should be modified or a water treatment device may need to be installed to control run-off volumes and velocities from the worksite or road.

2.6.2 Wildlife passage

Generally, provision for wildlife crossings by joint use of drainage pipes has not been totally successful. Some animals will not enter a confined space where they cannot see the other end, and will not enter a culvert when any water is flowing. Specialist advice should be sought when considering the use of drainage facilities for the movement of animals.

In some streams, there may be a requirement to maintain the connectivity of waterways for migratory fish species. In this case, the key principles for protecting fish passage are:

- maintain existing flow levels: ensure all works will maintain current water levels and avoid changing stream characteristics (e.g. don't introduce a waterfall where there isn't one)
- ensure outfalls are level with the stream's average water level: avoid the use of drop structures that introduce distance between the outfall and the stream water level
- mimic natural streambed conditions: allow natural streambed material to accumulate within culverts or drains, and consider fixing stones or other baffle structures to create resting areas for fish as they ascend waterways.

Austroads (2001) provides more detail about considering and managing the environmental impacts of road run-off and drainage.

CHAPTER 3

3 OBTAINING DESIGN FLOWS

3.1 Introduction

The information in this chapter provides some background on estimating flood levels, the concepts involved, and the setting of appropriate design standards.

For both small urban and rural catchments, the rational method is preferred. Rainfall / runoff models (intensity / frequency / duration (IFD) curves) based on statistical design rainfall are used to estimate design events. The production of the intensity / frequency / duration (IFD) curves is discussed in ARR Vol.1, Book II (Ed. 2001) and a brief summary is provided at the end of this chapter.

The procedures outlined here are those most commonly used and the local regulatory authority should be consulted to determine if an alternative design procedure is required.

3.2 General

The estimation of the likelihood of a design event is based on probabilistic or statistical analysis of historical records. The design event is linked to an average recurrence interval (ARI) or the probability of exceeding the estimated discharge. This simply means that an actual flood could exceed the capacity of a structure, designed to cater for events with an average recurrence interval of 100 years, on average, every 100 years. It is possible for floods to occur in successive years and then none for a further 200 years. Storms of lower discharge occur more frequently and hence have a lower average recurrence interval (ARI).

It is important to realise that the periods between exceedence of a given discharge are random and that storm estimation is based upon probability analysis. The calculation of the storm events for use in drainage design depends upon historical records of rainfall and resulting stream flow data. The availability of data varies from catchment to catchment and for some regions, little or no data exists.

Storms can also be described by the probability of being exceeded in any single year. The term used is the annual exceedence probability (AEP) and is the reciprocal of the ARI. The annual exceedence probability (AEP) is expressed as 1:100, i.e. there is 1 chance in 100 that the actual discharge in any one year will exceed the expected (design) discharge. The term AEP is usually applied to extreme storm events that occur infrequently (i.e. the probability of occurring in any year is less than 1 in 100). ARI is applied to events that on average occur every 100 years or more frequently (i.e. minor to rare floods that are likely to occur during the design life of most road infrastructure).

There are fundamental differences between estimating the frequency of design events and floods resulting from a storm event. The calculation of the discharge may involve the same procedures but the implications and assumptions are quite different. A design event is derived from probability analysis of historical records and is associated with an ARI or AEP. The discharge resulting from an actual storm depends upon the existing conditions within the catchment and involves deterministic analysis, rather than probabilistic analysis.

Three elements are required to design a storm water management system and these are:

- rainfall intensity-frequency-duration data

- location within a climatic zone which affects the rainfall temporal pattern
- the transformation of the rainfall input to a runoff event having the same recurrence interval.

A risk assessment can also be used to derive the design event and this is discussed separately in Chapter 2 – Design Considerations.

3.3 Rainfall Intensity – Frequency – Duration

3.3.1 Introduction

Rainfall intensity and duration are related, with higher intensities occurring for shorter durations. The relationship between storm intensity, storm duration and storm frequency has been analysed and design curves for some basic storms determined for the whole of Australia. Rainfall intensity-frequency-duration (IFD) design curves are discussed in 'Australian Rainfall and Runoff (ARR) – Vols. 1 and 2' (Institute of Engineers, Australia, 2001 & 1987).

A brief overview of the calculation of the IFD curves and their use is presented in Appendix A.1. It is beyond the scope of this Guide to derive IFD curves from basic rainfall data.

The application of IFD curves by the Rational Method requires the estimation of two variables:

Determination of a storm duration that results in the greatest discharge; and

The setting of an appropriate average recurrence interval.

Most authorities will specify their own average recurrence interval for the design of minor system elements. Any values provided in this Guide should be viewed as typical only and local values should be sought where they exist. The actual design standards adopted should be based on a risk assessment of the consequence of exceedence of the system.

The standards of downstream systems must also be considered when selecting an appropriate ARI.

3.3.2 Area Reduction Factor

The IFD curves are designed for a specific point and do not reflect the total rainfall patterns over a catchment. Reduction factors are applied to the point rainfall intensities such that the greater the area and / or the shorter the storm duration, the greater the reduction in the design rainfall. This reduction factor accounts for the improbability of intense storms occurring over a large area compared to occurring at a single point. ARR Vol. 1, Book 2 (Ed. 2001), Section 1.7 provides advice on the magnitude of the reduction. However, for catchments up to 4 km^2 no area reduction factor is required.

3.4 Climatic Zones

The Australian continent experiences a wide range of climates that significantly affect the rainfall / runoff responses of catchments. Similarly, regions in New Zealand experience varying climatic conditions. The most obvious indicator of climatic difference is rainfall. Australian rainfall patterns have been grouped into regions (see ARR Vol. 1, Book 2 (2001 Ed.), Figure 2.2). The main determinant of storm rainfall / runoff response in a developed catchment is not the average monthly rainfall but the intensity and duration of rainfall experienced during storm bursts. Fourteen discrete regions have been defined on the basis of average rainfall intensity observed in storm bursts with an average recurrence interval of ten years and a duration of one hour. For precise locations of the regions please consult ARR Vol 2.

On account of these variations, it is not practical to standardise kerb and channels, pit entries and pit sizes for Australia or New Zealand.

The application of a temporal rainfall pattern requires expertise and knowledge of catchment response. The Rational Method is proposed for most drainage projects, and assumes constant rainfall intensity for the storm duration. It is up to the drainage designer to decide whether the scope of the drainage project warrants the expenditure of extra effort to use either the unit hydrograph method or the runoff routing methods of discharge estimation incorporating temporal rainfall patterns.

3.5 Flow Estimation

There are several methods available to estimate the peak flows from catchments and these are covered substantially in Books 2, 5 and 8 of Australian Rainfall and Runoff Vol. 1 (2001). Hydrological analysis is the important first step in the design of road drainage and most design is often carried out with a hydrological design package such as StormCad, FlowMaster, ILSAX, Micro Drainage, XP SWMM, XP-RAT2000, PCdrain and Drains. Often a computer aided drafting (CAD) package, for example MX or 12D are used in the design and these packages interface with a hydrological design model.

The CAD approach enables design details (pit locations, pipe sizes, natural surface levels, etc.) to be exported to the hydrological design package for final detailing. The flow estimation methods commonly used are:

- rational Method (or the Partial Area Rational Method variant), is a version of regional flood frequency analysis
- unit hydrograph method
- runoff routing methods
- regional flood frequency analysis (e.g. Index Flood Method [Dalrymple, 1960]).

Some of the computer package based hydrological models are based on the rational method, but others predict hydrographs and incorporate detention and retention characteristics. The advantage of modelling retention and detention storage is that it can result in a more accurate prediction of runoff provided the input values reflect actual conditions.

Those methods mentioned above which interface to a CAD package are all acceptable, provided that the designer uses them correctly and selects the appropriate modelling tool for the particular situation. Where the drainage designer does not have access to a computer based package, or for small catchments where there are often no rainfall or stream gauge data available, then the Rational Method as discussed below is an acceptable alternative and can be manually calculated (or based on a spreadsheet approach).

3.5.1 Runoff Coefficient

The runoff coefficient relates peak flow discharged from a catchment to the rain falling over the catchment. The value is not constant, but varies with average recurrence interval 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 runoff coefficient.

In some parts of Australia it is common to prescribe runoff coefficients (e.g. 0.9 for urban areas and 0.35 for rural areas). However, it is worthwhile checking the sensitivity of the runoff volumes to variations in runoff coefficients and how this will affect the drainage design.

Urban areas

ARR (Book 8, Vol. 1, Ed 2001) recognises three cases relating the coefficient of runoff to rainfall intensity and the fraction of the catchment that is impervious, and these are represented by Equations (3.1, 3.2 and 3.3) below. These Equations are applicable to urban areas that are essentially homogeneous or where the pervious and impervious areas are intermixed. For catchments with areas that are different, other methods should be applied to determine runoff coefficients (see procedures under Section 3.5.1).

The relationships for an ARI of 10 years based on rainfall intensity are:

For ${}^{10}I_D \leq 25$mm/h	$C_{10} = 0.1 + 0.8 \times f$	Equation (3.1)
For ${}^{10}I_D \geq 70$ mm/h	$C_{10} = 0.7 + 0.2 \times f$	Equation (3.2)
For rainfall intensities between these limits	$C_{10} = 0.9 \times f + C^1_{10} \times (1-f)$	Equation (3.3)

where

C_{10} is the 10 year ARI runoff coefficient

f fraction of impervious area in the catchment

$C^1_{10} = 0.1 + 0.0133\,({}^{10}I_1 - 25)$ Equation (3.4)

${}^{10}I_1$ rainfall intensity for a storm of one hour duration and a 10 year ARI.

Coefficients of runoff can be determined for storms of other ARI by use of frequency factors (F_Y) as shown in Table 3.1 and Equation (3.5). If Equation (3.5) results in an coefficient of runoff greater than 1.0 then the coefficient of runoff of 1.0 should be adopted.

$$\mathbf{C_Y = F_Y \times C_{10}} \qquad \textbf{Equation (3.5)}$$

where

Y is the ARI in years

F_Y frequency factor from Table 3.1.

Table 3.1: Frequency Factors (F_Y) for the Coefficient of Runoff for use in the Rational Method (ARR Vol. 1, 2002)

Y (years)	1	2	5	10	20	50	100
F_Y	0.80	0.85	0.95	1.00	1.05	1.15	1.20

For simplicity, Equation (3.6) 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 (3.6) to developments with greater densities.

$$f = 3 \times RD - 5 \qquad \text{Equation (3.6)}$$

where

RD the number of residences per ha

Rural areas

By rearranging the rational method equation it is possible to derive a runoff coefficient Equation (3.7), for a catchment based upon observed rainfall and stream discharges. The method is applicable to individual catchments, irrespective of size.

$$C_Y = 0.36 \times Q_Y / \left(A \times {}^{Y}I_{t_c}\right) \qquad \text{Equation (3.7)}$$

where

C_Y is the coefficient of runoff from a storm with an ARI of Y

Q_Y is the discharge from a storm with an ARI of Y (L/s)

A area of catchment (ha)

${}^{Y}I_{tc}$ Storm intensity (mm/h) for a storm of duration t_c and an ARI of Y

The derivation of C_Y by the above method requires stream gauge data and rainfall data. This data is often processed by methods that differ from region to region and thus no single procedure can be applied to Australia or New Zealand. See summary Table 3.3 for the application of flood estimation methods for rural areas in the various Australian States and Territories. The general form of the modelling equations for each State is shown in Table 3.3 but the appropriate local Authority or ARR Vol. 2 should be consulted for details.

3.5.2 Time of Concentration

Time of concentration is the shortest time necessary for all points on a catchment area to contribute simultaneously to run-off past a specified point.

Urban Areas

Within urban areas, the time of concentration is largely determined by the surfaces over which the flows pass. Flows across the surface are commonly referred to as 'overland flows' and the time of the sheet flow is given by Equation (3.8) developed by Ragan and Durru (1972) and often referred to as the kinematic wave equation.

$$t_c = 6.94 \times \frac{(Ln)^{0.6}}{I^{0.4} S^{0.3}} \qquad \text{Equation (3.8)}$$

where

t_c time of concentration at a specified point (minutes)

L overland flow path length (m)

N Manning's n (see Appendix E for typical values)

I rainfall intensity (mm/h)

S slope of overland flow path (m/m)

Using Equation (3.8) requires an iterative approach because the rainfall intensity alters as the time of concentration alters. The equation should be limited to overland surface flows on pervious areas of less than 200 m. Generally sheet flows will enter some form of channel within 200 m and the flows are then calculated using Manning's Equation (6.3) for open channel flow.

For simplicity, 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 10 m and above, a three minute travel time or more, may be more appropriate Table 4.1 provides some other commonly adopted travel times for flow in gutters and surface channels to inlet pits.

Typically a minimum travel time of five minutes is adopted for any drainage element to runoff into an underground or surface system. All carriageway elements up to 200 m in length, regardless of slope, fall within this minimum.

Rural Areas

In many locations, little or no data exist and an estimate of the time of concentration is required. Table 3.2 shows typical stream velocities for varying stream bed slopes. Table 3.3 shows the various methods adopted to estimate the time of concentration throughout Australia. The general form of the modelling equations for each State is shown in Table 3.3 but the appropriate local Authority or ARR Vol. 2 should be consulted for details.

Table 3.2: Typical stream velocities for varying terrain (Department of Main Roads QLD 2002)

Terrain	Flat, Slope <1.5%	Rolling, Slopes 1.5 to 4%	Hilly, Slopes 4 to 8%	Steep, Slopes 8 to 15%	Very steep, Slopes > 15%
Flow velocity (m/s)	0.3	0.7	0.9	1.5	3.0

Table 3.3: Summary of application of rational method for rural areas

Region	Time of concentration	Runoff Coefficient	Frequency Factors
ACT	NSW – Eastern zone apply		
NSW - Eastern	$t_c = 0.76A^{0.38}$	C_{10} from ARR Vol. 2	Read off map[1] with adjustment for elevation
NSW – Western (flat catchments)	$t_c = 0.76A^{0.38}$	C_{10} from ARR Vol. 2 with adjustment for area	Read off map[1] with adjustment for elevation
NSW – Western (hilly catchments)	Regional flood frequency method applicable		
Northern Territory	$t_c = \frac{91 \times L}{A^{0.1} \times S_e^{0.2}}$	Approximate values given	na
Queensland	Average velocities (for catchments < 5km^2) $t_c = \frac{8.5 \times L}{Ch \times A^{0.1} \times S_e^{0.4}}$ (for catchments > 5 km^2)	C_{50} determined from summation of rainfall intensity, topography, storage and ground cover factors.	na
South Australia – Eastern	$t_c = 0.5A^{0.65}$	Predetermined values	na
South Australia – Northern and Western	$t_c = \frac{91 \times L}{A^{0.1} \times S_e^{0.2}}$	Values dependant upon slope	na
Region	**Time of concentration**	**Runoff Coefficient**	**Frequency Factors**
Tasmania	$t_c = \frac{91 \times L}{A^{0.1} \times S_e^{0.2}}$ for rural catchments (less than 20 ha)	C = 0.9 for paved areas C = 0.35 for rural catchments (common to apply a sensitivity check to test robustness of results).	Not used
Victoria	$t_c = 0.76A^{0.38}$	C_{10} from ARR Vol. 2	Tabulated values
Western Australia	$t_c = \Delta A^{\delta}$ where the factors Δ and δ are related to one of 7 zones	$C_{10} = \Phi.10^{\varepsilon\phi}(LS_e)^{\gamma}$ where the factors Φ, ε, ϕ and γ are related to one of 7 zones	C_Y / C_{10} factors based on the percentage of cleared land

Map is located in ARR Vol 1, Book 4 (ARR, 2001)

Ch Chezy's coefficient

S_e equal area slope

3.5.3 Rational Method

The Rational Method of flow estimation 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', t_c, of the catchment.

The basis for the 'Partial Area Rational Method' is the rainfall / runoff flow estimation procedure in which catchment contributing areas and their respective response times are represented on a time-area graph (this is discussed in greater detail in the worked example A.2.4 in Appendix A.2). Rainfall information from IFD charts may be combined with the time-area catchment representation to yield peak flow estimates. The Partial Area Rational Method is a simplified non-graphical version of this method and is the preferred rainfall / runoff model.

The IFD curves are applied to produce a hydrograph that permits the peak discharge to be estimated. Peak discharges are than calculated for storms of varying durations and these are plotted to determine the design flood discharge for a particular location.

The Rational Method is a simple method that can be solved using a hand-held calculator, though a personal computer is more often used. Limitations of the Rational method include 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 runoff coefficient. Design flow estimated using the Rational Method has about ± 25% accuracy and is calculated using Equation (3.9) as shown below:

$$\mathrm{Q_Y} = \mathrm{C_Y} \times {}^{\mathrm{Y}}I_{t_c} \times \mathrm{A} / 0.36 \qquad \text{Equation (3.9)}$$

where

Q_Y peak discharge for a storm with an ARI of Y years (L/s)

C_Y runoff coefficient for a storm with an ARI of Y years (dimensionless)

${}^{Y}I_{t_c}$ average rainfall intensity for a storm of t_c hours duration and an ARI of Y years (mm/h)

A catchment area (ha).

Explanation of the various applications of the Rational Method is best illustrated by use of examples, see example Appendix A.2. The Partial Area adaptation of the Rational Method is preferred.

3.5.4 Unit Hydrograph Method

A hydrograph is a plot of stream discharge against time. A unit hydrograph is a hydrograph resulting from a unit (typically one millimetre) depth of surface runoff produced by a rainfall of a specified duration. A runoff of two units at the same duration, would result in a hydrograph whose ordinates are twice those of the unit hydrograph, i.e. the ordinates of the hydrographs are proportional for storms of the same duration.

Derivation of unit hydrographs requires stream-flow and rainfall data. Where a unit hydrograph is needed for a given catchment, but the data used in developing the model was based upon, for example, regional data, the unit hydrograph is commonly referred to as a synthetic unit hydrograph. Where a unit hydrograph has been developed for a particular gauge station it is only applicable to that location and must be adjusted for other locations along the stream.

The response of a specific catchment to rainfall (storage, runoff coefficient, spatial variations, etc.) is accounted for in the derivation of the unit hydrographs and therefore requires no assumptions.

Using data from one catchment to predict floods in another of different characteristics is likely to produce inaccurate outcomes.

However, unit hydrographs need to be derived for storms of varying intensities, frequencies and durations. Changes within the catchment (i.e. urbanisation, road construction) can only be modelled using arbitrary processes.

This method is not accurate for small or urban catchments.

Readers are referred to ARR Vol. 1 Book 3 (2001) for worked examples for the derivation of unit hydrographs.

3.5.5 Runoff Routing Methods

Runoff from storm events is routed through various storage conditions within the catchment before it is discharged. The models used can become complicated and a doubling of rainfall may not necessarily double the runoff (i.e. non-linear response). Runoff routing methods, if handled correctly, can closely simulate observed flows.

Runoff routing permits the determination of hydrographs at any location within the catchment being modelled allowing storages to be introduced to determine cumulative effects. This approach is often used to study the effect of development on catchment characteristics.

Spatial variation in rainfall can be included in the method though this would normally only be considered for extreme flood events. It is also possible to spatially vary losses using runoff routing.

This method often relies on a mathematical model embedded in propriety software and the user then inputs the desired storage characteristics necessary to model the catchment. The input of the correct model parameters usually requires expert knowledge of hydrology and of the specific computer package. Any models derived in this manner should be verified against actual storm events. Of particular concern with the computer modelling approach is the non-linearity response component which, if not handled correctly, can have far reaching effects on the output values.

In practice, hydrographs for storms of varying durations at the ARI of interest are calculated so that the highest peak discharge can be determined. A smooth curve is fitted to a plot of rainfall duration versus peak discharge.

This flood estimation method is the most flexible and the most complex. It is applicable to both small and urban catchments and large essentially rural catchments.

3.5.6 Regional Flood Frequency Method

This method requires discharges from recorded floods for specific catchments that are then analysed using statistical processes. The methods are described in Chapter 10 of ARR Vol. 1, Book 4 (2001).

In practice, the analysis relates flood frequency data and the properties of the catchments. The relationships are determined by application of regression analysis and as such, are of limited accuracy when extrapolated to other catchments or beyond the range of data from which they were derived. The relationships are used to predict floods of required frequency.

Floods of required probability of exceedence are estimated directly from data and as such include catchment characteristics such as storage, runoff coefficients, spatial variations, etc. This means there is no need to make assumptions regarding the rainfall and the runoff.

This method cannot account for changes within the catchment such as urbanisation or road construction as it is relies upon analysis of historical data. It is desirable that at least 10 years of data be used to estimate flood frequency data for a catchment.

CHAPTER 4

4 MAJOR URBAN DRAINAGE NETWORKS

4.1 Introduction

4.1.1 Scope

The storm runoff management design practices proposed in this Guide are based on the major / minor drainage concept introduced in Chapter 1 — Introduction. Major drainage systems use the roadway reserve, drainage easement and open space or 'green belt' areas of a developed **urban** landscape to carry all major runoff flows for a specified level of rare flooding.

The primary aim of this chapter is to describe a procedure for the planning of major systems for small, urban catchments. The procedure draws on guidelines and hydrological / hydraulic information presented in other chapters.

The procedures in this chapter do not apply to rural areas.

4.1.2 Connectivity of Catchments

The major / minor drainage system designed for a specific portion of an urban landscape forms part of the master drainage plan for the entire drainage basin within which the project site is located.

The implementation of a road project potentially has an effect on major flow paths and flood peaks (see Figure 4.1). This effect must be identified and accounted for. It may be that there is spare capacity in downstream major flow paths, if not detention or retention should be provided. Alternatively, it may be appropriate and consistent with other objectives of the urban plan, e.g. traffic planning, road hierarchy, etc., to alter the gutter / pavement profile of a receiving roadway reserve and, as a result, provide greater flood escape capacity.

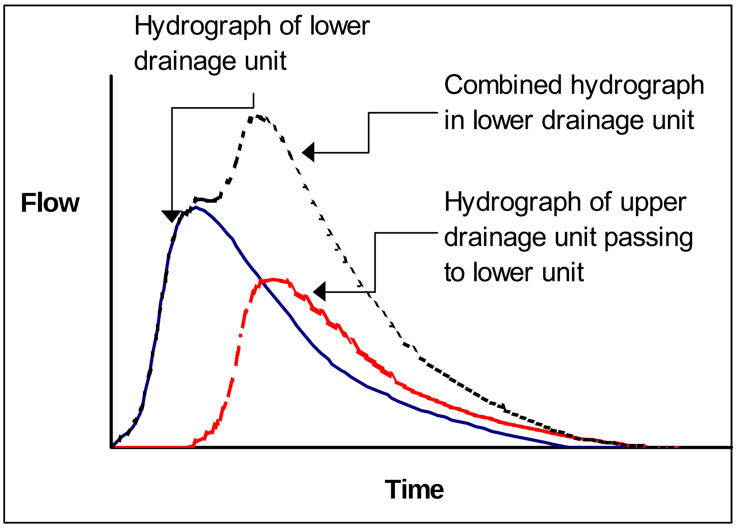


Figure 4.1: Hydrographs for slope-aligned urban drainage units (after ARRB Special Report 34, 1986)

More frequently however, drainage designers are forced to reduce the impact of storm runoff originating upstream by means such as those described in Chapter 6 – Surface Flows.

4.2 Major Urban Drainage Concepts

4.2.1 Scope

The major drainage system associated with an urban development is a network of surface flood paths taken by storm runoff during times when its subsidiary minor system is rendered partially inoperable as a result of blockage, or when the capacity of the minor system has been exceeded. In a properly planned scheme such occurrences are likely to cause flooding of open space areas and inundation of the grounds of buildings, but no indoor damage other than to buildings of secondary importance.

A worked example for the design of a major storm drainage network is provided in Appendix B.2.

4.2.2 Major Urban System Planning Procedure

There are eight steps that must be taken to plan or design a major urban drainage system for a project:

1. Catchment definition;
2. Fixing of roadway reserve capacity flows;
3. 'Gap flow' design storm selection;
4. System planning table;
5. Network review;
6. System evaluation;
7. Sub-area detailing; and
8. Final design detailing.

These are discussed in detail in the following sections and shown diagrammatically in Figure 4.2.

Catchment definition

Catchment definition involves the following:

- identify the location of the project, i.e. climatic region, rainfall relationships, etc.
- preparation of a contour map of the area (1 or 2 metre contours, or closer spacing if area very flat) ; scale 1:1000 to 1:5000
- definition of the project boundary and constraints consistent with the master drainage plan
- identification of the pattern of internal roads, relevant traffic management information
- identification of roads and streets as dual-channel (flow in two channels bound by the road crest and the kerb) or potential single-channel (flow in a single channel, i.e. no road crest – one way cross fall) flow paths and their hydraulic characteristics
- identification of major (common) land use areas (i.e. percentage of pervious and impervious surfaces)
- determination of design ARI for underground network
- identification of the likelihood of partial blockage of the underground network and pit entries
- identification of relevant environmental considerations (see Section 2.6 – Environmental Issues)
- deciding the 'natural drainage direction'
- nomination of flood disposal points

- definition of internal sub-catchments and node
- definition of flood escape networks, node sections and drainage sub-areas of each sub-catchment.

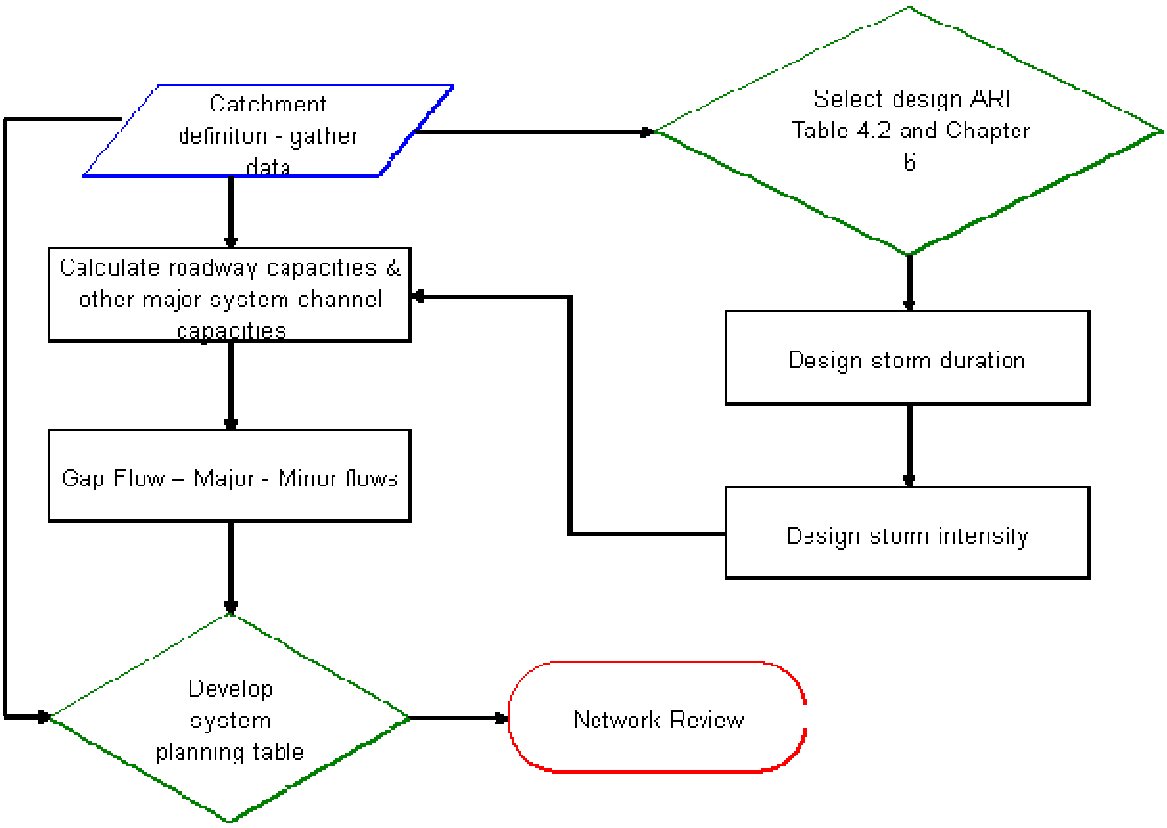


Figure 4.2: Overview of the design of a major storm network

Surface flow paths are often governed by the design of the project, including surface levels and traffic controls. It is helpful if the information is available at the time of drainage design.

Identification of the flood escape network and node sections in each catchment is part of the final task and one which draws together many interacting threads of catchment data. Catchment and drainage sub-area definition both apply two assumptions relating to the way major floodwater moves through an urban landscape:

- flow into and / or out of a sub-area takes one entry and / or exit path only; or
- the path taken by storm runoff at an intersection is along the roadway path of steepest grade.

It is not difficult to ensure, by appropriate shaping of gutter and roadway profiles, that both of these assumed behaviours do in fact occur in urban catchment runoff events of small magnitude. It is almost impossible, on the other hand, to guarantee similar performance in major storms.

A satisfactory level of conformity to the assumed behaviours can be achieved by making the bulk of roadway flood escape paths dual-channel. This has two consequences, it:

- maximises available flood escape channel capacity; and
- provides, by the presence of the roadway crown, a modest degree of flood proofing in the system.

A continuous roadway crown in a street, which has 'high' and 'low' sides, forces an uneven distribution of flow to occur in its dual-channel carriageway (see Figure 4.3). The greater flow holds to the high side kerb where it can be tolerated, while the lesser flow passes along the low side kerb, where fronting properties are usually more vulnerable to damage by roadside channel surcharge.

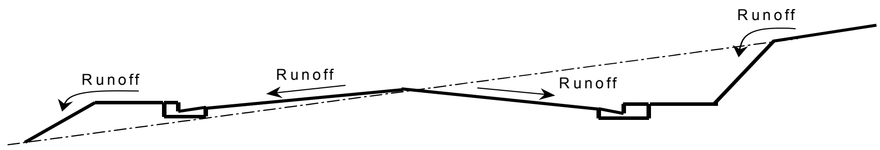


Figure 4.3: Uneven flows in dual-channel carriageway due to natural cross slope

The single-channel carriageway form (one-way crossfall) may be used with impunity in upper and / or remote sub areas of catchments. Storm runoff from such areas is, typically, well below the carrying capacities of dual-channel carriageways.

Fixing of roadway reserve capacity flows

The fixing of roadway reserve capacity flows involves two hydraulic calculations:

- calculation of roadway reserve capacity flow, Q_c, for each type of carriageway likely to be used in the catchment; and
- application of a storage correction to these capacity flows, hence Q_{sc} for each type of carriageway.

The first of these tasks employs the procedures and criteria for calculating capacity flows in open channels (i.e. roadways). Two criteria operate to limit the resulting capacity flows:

Criterion 1: limits the maximum flow level to 50 mm above top-of-kerb.

Criterion 2: limits the maximum value of Equation (4.1) to 0.4 m^2/s for pedestrian safety.

$$d_g \times V_{avg} \qquad \text{Equation (4.1)}$$

where V_{avg} mean flow velocity (m/s)

d_g kerb side flow depth (m)

Where pedestrian safety is not of concern, the maximum value for vehicle safety from Equation (4.1) should be 0.6 m^2/s.

Capacity flows, Q_c, corresponding to each carriageway type likely to be used in the development are calculated for the range of (gutter) longitudinal slopes present in the catchment (see Section 6.4.2). The outcome of the analysis is a table of capacity flows, Q_{sc}, one for each carriageway type and longitudinal slope occurring in the catchment.

A storage-correction is made to these flows. This is based on the temporary or detention storage of runoff in the surface channels and underground pipes of an urban stormwater drainage system. This depresses the peak flow discharge that emerges from the system in much the same way a flood control dam reduces the peak of an incoming flood wave. In the case of a major / minor system that includes underground pipes, the reduction is about 10%. Where gutter / pavement storage only is available, the reduction is about 5%.

An upward correction may therefore be applied to the previously calculated capacity flows. Values may be increased by 10% in catchments where underground pipes are employed, and by 5% where open channel drains only are used.

'Gap Flow' design storm selection

Four main tasks are involved in the design of major flow paths:

- select an appropriate design ARI for the storm;
- determine the flow that exceeds the capacity of the minor system, Q_{gap};
- determine appropriate design storm duration; and
- determine average intensity for the design storm.

The question of design ARI recommended for major systems, and the interpretation of this in terms of the conjunctive use of surface channels and the pipes of the underground network, if present, is discussed in Chapter 6 – Surface Flows; and values are included in Table 4.2. Information obtained from the table enables the drainage designer to focus their attention on that component of the design flood 'gap flow', Q_{gap}, (i.e. the runoff due to a major event storm which is not carried by an underground pipe network or table drains and the like) which is moving in the surface channels of the flood escape network. The drainage designer must adopt a likely blockage condition as part of this process.

The approach adopted here relies on the Rational Method where the discharge carried in the major system elements (Q_{gap}) is determined by estimating the total discharge for the catchment at the appropriate ARI for a major event (see Table 4.2) and subtracting the volume carried by the minor system (with an assumed blockage factor). Mathematically this can be represented as follows:

Q_{gap} = total runoff – runoff in minor system

$$Q_{gap} = F_n \left(CA\right)_{10} {}^{n}I_t / 0.36 - F_N \left(CA\right)_{10} {}^{N}I_t / 0.36$$

$$Q_{gap} = \left[F_n {}^{n}I_t - F_N {}^{N}I_t\right] \frac{\left(CA\right)_{10}}{0.36}$$

where

F is the frequency factors given in Table 3.1

C is the runoff coefficient (for an ARI of 10 years)

A is the catchment area (ha)

I is the rainfall intensity (mm/h)

N is the ARI applicable to a minor event (from Table 6.3)

n is the ARI applicable to the major event (from Table 4.2)

t is the critical storm duration (min).

The third task, that of fixing design storm duration, may require one or more iterations for determination of the critical storm event. This is built into the design procedure (see Section 4.2.2 — System evaluation). However, an approximate duration is needed to initiate the calculations, and guidance is given for small urban catchments in Table 4.1. See Section 3.5 for deriving estimates for larger urban or rural catchments.

Table 4.1: Design storm durations* for small urban catchments (after Argue, 1984)

Roadside channel type	Residential Sub-divisions	Commercial / Industrial developments
Kerb-and-gutter roadside channels throughout	10 to 15 minutes	15 to 20 minutes
Grassed swale or blade-cut roadside channels	15 to 20 minutes	not applicable

These storm durations correspond to catchment travel times over or in impervious areas: they incorporate a roof-to-gutter or roof-to-underground pipe travel time of 5 minutes and 10 minutes for residential or commercial / industrial developments, respectively.

The last task requires determination of the design average rainfall intensity (see Chapter 3 – Obtaining Design Flows) which is used in the calculations in the following section. Design ARI and storm duration result from decisions made in the previous three tasks.

System planning table

A system planning table is the central feature of the procedure, because it enables a drainage designer to evaluate and modify, and where necessary, trial a major drainage system.

The storage-corrected capacity flows, Q_{sc}, need to be calculated for a range of flood escape paths available in a catchment (as devised in Section 4.2.2 — Fixing of roadway reserve capacity flow). Combining this information with the design storm average intensity (selected in Section 4.2.2 — Gap flow storm selection), it is possible to determine for each flood escape path, the 'tributary (impervious) area', TA_i. This area is the impervious area which yields, in a storm of average intensity equal to that (selected in Section 4.2.2 — Gap flow storm selection), peak runoff flow matching the storage-corrected capacity flow, Q_{sc}, of that path.

The table listing values of TA_i for the full range of flood escape paths available constitutes a System Planning Table for the development.

The table is used in the 'Network Review' discussed in the next section. The data required to make use of the table are:

- flood escape path carriageway width
- carriageway longitudinal slope.

Network review

Of all the data and component information contained in the catchment definition (see Section 4.2.2 — Catchment definition), those relating to the road geometry (carriageway widths, roadway reserve cross-sections, pavement types, etc.), are the most amenable to change. The network review procedure devolves upon this fact, for it enables a trial urban plan to be tested and modified where necessary, without disturbing the plan itself in any basic way or altering the inter-relationship of its main components.

With the System Planning Table, the particular roadways and longitudinal slopes of a developed catchment can be drawn up (see Section 4.2.2 — System planning table). Using runoff coefficients applicable to its likely ultimate development (see Chapter 3 – Obtaining Design Flows), it is possible to perform a series of systematic 'test-and-modify' calculations. These can be used to assess the performance of the flood escape networks defined in (Section 4.2.2 — Catchment definition). This review is amenable to tabulation and computer based methods.

The procedure involves comparing the equivalent impervious area, (CA), computed at successive sections along each drainage path, with the value of TA_i listed for the drainage path defined in terms of its width (carriageway) and longitudinal slope in the System Planning Table. At those sections where (C x A) is less than TA_i, it follows that the flood escape path will satisfactorily convey flood runoff in the design storm (selected in Section 4.2.2 — Gap flow storm selection). Where it is greater, then the drainage designer must explore alternative flood management options. In essence, the capacity of the storage corrected capacity of the flood escape paths is fixed (but can be altered in subsequent iterations) and this permits the calculation of a contributing (tributary) area that matches this capacity. If the actual contributing area in the catchment is less than the tributary area, the capacity of the flood escape paths will not be exceeded.

The outcome of this process is a 'first approximation' major system drainage network for the development.

System evaluation

The first approximation major drainage system determined in (Section 4.2.2 — Network review) must be evaluated to ensure that the assumptions made in the course of its derivation, in particular the assumed design storm duration, are valid. This involves estimating a corrected or ‘new’ flow travel time in each catchment of the development and comparing it with that adopted from Table 4.1.

Where ‘adopted’ versus ‘new’ flow travel times differ significantly, i.e. in a particular catchment, a revised critical storm duration based on the ‘new’ estimate of travel time may be returned to (Section 4.2.2 — 'Gap flow' design storm selection;) and the subsequent steps repeated until satisfactory agreement is reached.

The qualification ‘may’ entered here covers the case where the first approximation of the major system is demonstrably conservative, but unalterable for reasons that stem from the requirements of the road / traffic plan.

The outcome of the system evaluation process is a layout of the major drainage network.

Sub-area detailing

Completion of the major drainage system planning procedure involves one further step: the detailed review of flow movement within each sub-area. This calls for careful application of the principles of hydrology and laws of open channel hydraulics to ensure that storm runoff finding its way to the roadway reserves and drainage easements of the flood escape network, moves through and from each sub-area without surcharging its defined drainage paths.

An example of the type of review referred to here is where a potential single-channel lateral street, which is not a component of the main flood escape network, is required to convey a flow exceeding its single-channel capacity. Various options must be explored by the drainage designer, e.g. change to dual-channel form, provide some form of flood proofing such as raised footpath, etc.

It must be emphasized that the earlier network review completely overlooks such sub-area internal hydraulic detailing. This must be carried out after a broadly satisfactory major drainage system has been adopted.

Final design detailing

This step involves all detailing necessary to define the system of open channels, including carriageways, roadway reserves, flood-proofing, drainage easements, roadway hydraulic geometries, etc., which will convey runoff from major system design storms through and beyond each catchment without indoor damage to residential and other important buildings.

The resulting plans form the basis for the design of the minor drainage system to be incorporated within it.

4.3 Design Recurrence Interval

4.3.1 General

All elements within a drainage system must be designed to provide immunity against some level of exceedence which is usually expressed as being able to cope with a storm of a given average recurrence interval (ARI). The standards applicable to each element will depend upon the specific circumstances and the requirements of the authority that controls stormwater management at the site. The values in Table 4.2 represent basic values that can be applied in the absence of other criteria. Users should also check the criteria in Section 6.4 for urban situations. A risk assessment procedure can also be applied to determine what is an appropriate ARI, and this is discussed in Chapter 2 – Design Considerations.

Consideration should be given to the likelihood of upgrading existing infrastructure and the impacts of adopting the existing recurrence interval on any new drainage system. Where the existing infrastructure has been designed to cope with storm events of inadequate recurrence intervals, it may be more economical to install new drainage schemes to the correct recurrence interval standard with a view to ultimately upgrading the pre-existing infrastructure at some time in the future.

Where resources are limited it may be possible to use a staged construction approach to provide varying levels of service until, ultimately, the drainage scheme is able to provide an appropriate flood immunity.

4.3.2 Standards Applicable to a Major Network

Provision of flood immunity is accomplished by ensuring that major networks are designed to maintain flood levels below predetermined levels for facilities in adjacent areas. The appropriate flood levels vary depending upon the facility in question and the general terrain.

The ARI for adjacent land use are shown in Table 4.2. The actual value adopted should comply with the requirements of the local regulatory authority, since requirements vary throughout Australia.

(See Table 4.2 over page)

Table 4.2: Tecommended minimum recurrence interval standards for major network review adjacent land use (after ARRB Special Report 34, 1986)

Priority	Situation	ARI[1] (years)
A	Floor levels of hospitals and civil defence headquarters.	500
	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.	200
B	Floor levels of residences; essential food , pharmaceutical, retail stores, department stores; centres employing a large labour force; community administration and education centres; centres of rare artefacts; venues for entertainment, dining, popular indoor sport.	100
	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.	50
C[2]	Grounds of all units belonging to priority A listed above, outdoor areas where rare artefacts are displayed or stored.	5 to 10
	Grounds of all units belonging to priority B listed above.	3 to 5
	Other open space areas, including general parks and outdoor sport and recreation areas[3].	1 to 3

States have varying standards.

An ARI of 10 years may be applicable for flat or low lying areas.

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.

CHAPTER 5

5 SUB-SURFACE DRAINAGE

5.1 Introduction

The design procedures described in this chapter are relatively simple. The hydraulic design of subsurface drainage systems (also referred to as subsoil drains) within road structures is not a precise science because soil layers within a road structure are never uniform. Water tables, when evident, are usually ill-defined and vary significantly from location to location within the road project. An elaborate detailed design is rarely warranted. Geotechnical investigations before the design of new roads or the rehabilitation of existing roads should indicate where standard subsurface drainage is needed or where an elaborate detailed design is warranted.

General guidelines for the location of subsurface drains along roads are set out in Section 5.8.

A procedure for design of cut-off drains is set out in Section 5.14.

5.2 Sources of Moisture

The main mechanisms by which moisture can enter a road subgrade and / or pavement are shown diagrammatically in Figure 5.1 and listed below:

- seepage from groundwater
- movement of a water-table under a road
- rainfall infiltration through the road surfacing
- capillary moisture from the verges
- capillary water from a water-table
- vapour movements from below the road
- lateral movement of moisture from pavement materials comprising the road shoulder.

All road surface materials are permeable to some degree, and may have defects or joints that allow water to pass into the subsurface layers of the road. Although permeability of asphalt may decrease with time due to compaction and filling with surface detritus, cracking will become a significant source of water entry as the pavement ages.

Infiltration of surface water into the road pavement is perhaps a reason for the generally poor performance record of boxed pavements in wet areas of Australia.

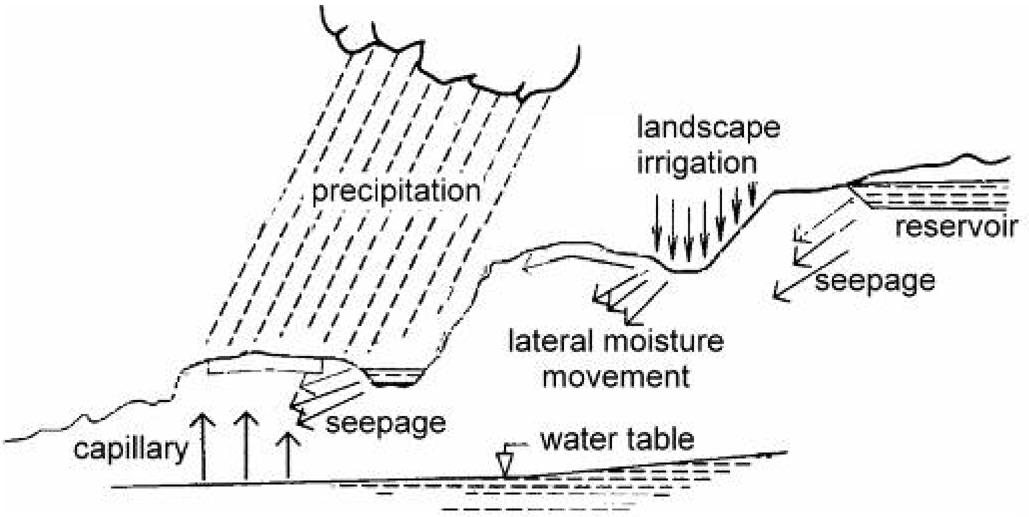


Figure 5.1: Sources of moisture (after ARRB Special Report 35, 1987)

5.3 Control of Road Moisture

The three basic techniques for controlling moisture are:

Layer Protection: For example, seal coats, plastic sheeting and other impermeable barriers placed at various levels in the pavement structure. The durability of this type of moisture control is suspect, and if the barrier lets some moisture in, pavement failure is likely.

Rendering Subgrade Insensitive: Lime or cement stabilisation are examples of this technique. The load capacity of the stabilised material does not significantly decrease with increasing moisture content. The disadvantages are additional expense and a significant reduction in permeability (providing cracks do not develop).

Subsurface Drainage: Removal of moisture from the pavement structure via a subsurface drainage system. A correctly designed and maintained subsurface drainage system is the only way of ensuring a stable moisture condition.

This chapter is concerned only with the subsurface drainage.

5.4 Objectives

The primary objective of the structural design of a road pavement is to ensure that it will withstand the design traffic loads over the design life under prevailing environmental conditions.

Subsurface drains are provided in order to avoid the following types of premature failures:

- loss of subgrade strength due to an increase in moisture content in moisture susceptible materials
- overload of the subgrade due to hydrostatic transmission of live load through a saturated pavement
- layer separation and pothole formation.

When the subsurface drains are connected to the stormwater drainage system, subsurface drains should be placed above the stormwater drainage system, so as to prevent back flow of stormwater into the subsurface drains.

5.5 Information Required

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

A study of the permeability, capillarity and dispersivity of the soils should also be undertaken. The investigation can vary from a short site visit to detailed geological mapping supported by drilling, in situ testing, sampling, laboratory testing and field inspections extending over a range of seasonal conditions. If unexpected surface water or wet conditions during inspections persist for several days or weeks after rain ceases, then a thorough assessment is usually warranted. It is desirable that investigations be completed before the grade line of the proposed pavement is fixed.

The detailed design of subsurface drains requires information about the road environment such as the coefficient of saturated permeability and capillary characteristics of potential road materials. Materials on and adjacent to the road, as well as imported materials should all be tested. Graphs that relate permeability to grading can be used for preliminary investigations, but a laboratory permeability determination is the only accurate method for assessing the permeability during the detailed design stage. Also the likely sources and amounts of moisture ingress to the road structure should be considered.

If the soil data are variable or unreliable, then the design of the drainage system may also be unreliable. Where the material is variable and sensitive to moisture changes, then the drainage design may be ineffective. In these cases a conservative pavement design approach is often the safest solution.

5.6 Types of Subsurface Drains

Subsurface drainage systems are generally installed in a road either to remove water from the subgrade and pavement materials or to intercept water before it reaches the road structure.

The former type is known as a *pavement drain*, and the latter is called a *cut-off drain* or is sometimes known as a *formation drain.*

Subsurface drain pipes may be surrounded by a single stage filter, or by two stage filters. Filter materials can consist of aggregates (ranging in size from sand to coble size), geotextiles or combinations of aggregates and geotextiles. Examples are shown in Figure 5.2. The level of filtering will be determined by the prevailing soil types and the requirements on the discharge waters. In some cases a second stage filtering may be required and this can take the form of a geotextile wrap either around the pipe or around all the filter material.

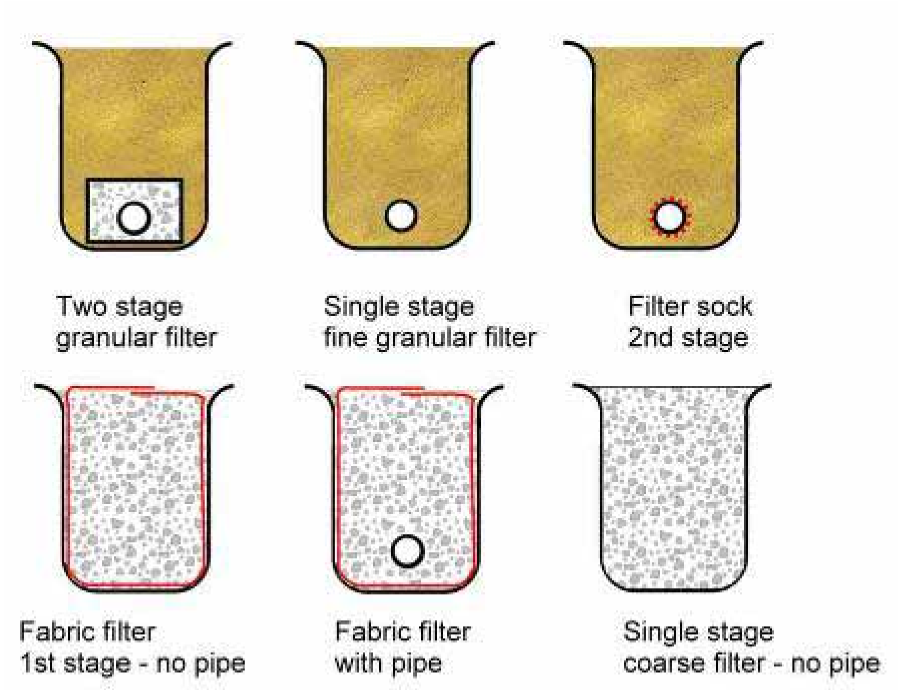


Figure 5.2: Subsurface drain types

A more recent form of subsurface drain is the geocomposite edge drain (see Figure 5.3). These are nearly always prefabricated and are often referred to as prefabricated geocomposite edge drains (PGED). They consist of a polymer core (typically in some form of 'egg box' structure) wrapped in a geotextile and are supplied complete. They can be installed in much narrower trenches than traditional pipe based drains.

They are considered to be at least equal to pipe drains though they are more difficult to install correctly due to buckling. They are impossible to clean if they become clogged and require careful selection of the filter fabric to correspond to existing conditions.

Installation as shown in Figure 5.3 lessens problems with buckling and reduces the incidence of void spaces between PGED and trench wall.

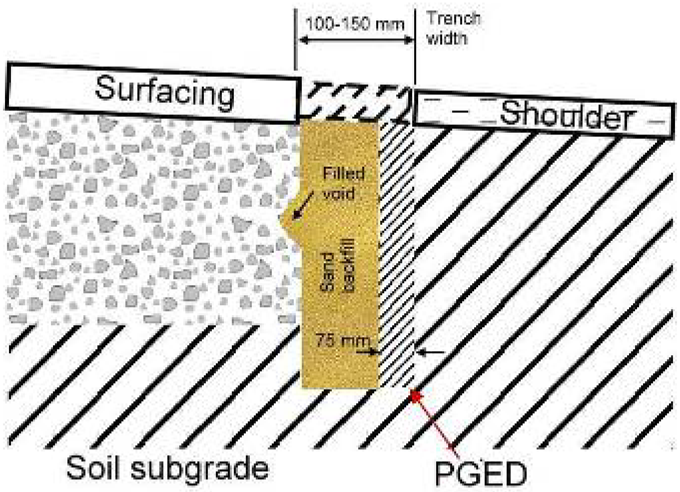


Figure 5.3: Prefabricated geocomposite edge drains (PGED) installation (adapted from Koerner *et al*., 1994)

5.7 Design Procedure

- Decide on an appropriate materials testing and site investigation program for the project.
- Arrange for pavement design. The pavement depth must be known to set subsurface drain levels.
- Select the appropriate locations of subsurface drains from Section 5.8.
- After groundwater investigation, carry out hydraulic design of cut-off drains (see Section 5.14).
- At the road grading stage, ensure that fills are high enough to inhibit capillary rise (see Section 5.15) and allow for subsurface drain outlets, and that cuts can be properly drained.
- If subsurface drains can be placed parallel to the road surface in the vertical plane, subsurface drain detailing can follow stormwater drain design. Where the road gradeline is very flat (less than 0.5% grade), and there is a need for independent grading of subsurface drains, both drainage systems could be designed concurrently if appropriate to do so.
- Identify cut-to-fill lines and locate the transverse drains.
- Detail the locations of flushout risers and outlet pits where these do not coincide with stormwater pits.

Drainage facilities should be designed and constructed recognising that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users. Investigation of potentially cracked or failed underground pipes should be carried out using a remote television camera to reduce the risk to inspection personnel.

5.8 Locations of Subsurface Drains

In areas of Australia where moisture ingress is unlikely, truck traffic is light and similar pavement designs in the vicinity have already performed satisfactorily without subsurface drains, they may be omitted. It is difficult to describe all circumstances that warrant the installation of subsurface drains. However, where soils are not free draining (i.e. clays, silts, loams) or where there is a likelihood of water ponding near the pavement, subsurface drains should be considered.

While provision of subsurface drains without design may appear wasteful, it has been found essential to provide drains extensively on arterial roads where soils are not free draining. Omission of subsurface drains on arterial roads has caused premature pavement failure and considerable expense in installing them afterward.

The following guidelines are offered where the need for subsurface drains has not been established.

5.8.1 Longitudinal Subsurface Drains

Subsurface drains should be considered:

- along the low sides of pavements of urban roads and freeways
- along both sides of the pavement near any cut to fill line
- along both sides of the pavement with kerb and channel
- along both sides of the pavement where the crossfall is flatter than 0.02 m/m in a superelevation development
- along the high side of pavement where seepage is evident, or where water may enter from batters, full-width pavement, service trenches or abutting properties

- along joins between an existing pavement and a pavement widening where pavement depths or permeabilities could create a moisture trap.

5.8.2 Medians

Subsurface drains should be considered along the:

- low side of a dished median where the median drain invert level is less than 0.2 metres below subgrade level of the adjacent pavement
- low side of a kerbed median where the cross-slope is 0.10 m/m or more
- sides of a median with a fixed watering system or wider than 6 metres
- centre of flat grassed medians without fixed watering systems and less than 6 metres wide.

5.8.3 Transverse Subsurface Drains

Transverse subsurface drains should be considered:

- on the upstream side of cut-to-fill lines
- along changes of pavement depth or permeability
- at both ends of bridge approach slabs.

5.8.4 Cut Off Drains

Subsurface cut off drains should be considered:

- Along both sides of cuts where the road is known to be below the water table, or where seepage is encountered during construction, or where seepage is expected in wet weather.
- Transversely at any seepage areas, and further downgrade if required. The transverse drains may be laid in a herringbone pattern if necessary to achieve the minimum grade.

5.8.5 Access to Subsurface Drains

Inlets and outlets for subsurface drains should be located clear of the traffic lanes. Where the inlet must be located in the shoulder, a trafficable steel cover is used. The inlet should not be located in a position where it would be possible for stormwater to enter the subsurface drainage system.

In urban areas, subsurface drains usually start and end in drainage pits. In rural areas, the flushout-riser inlet (Figure 5.4) is cheaper than a pit for intermediate access and outlets are as shown in Figure 5.5.

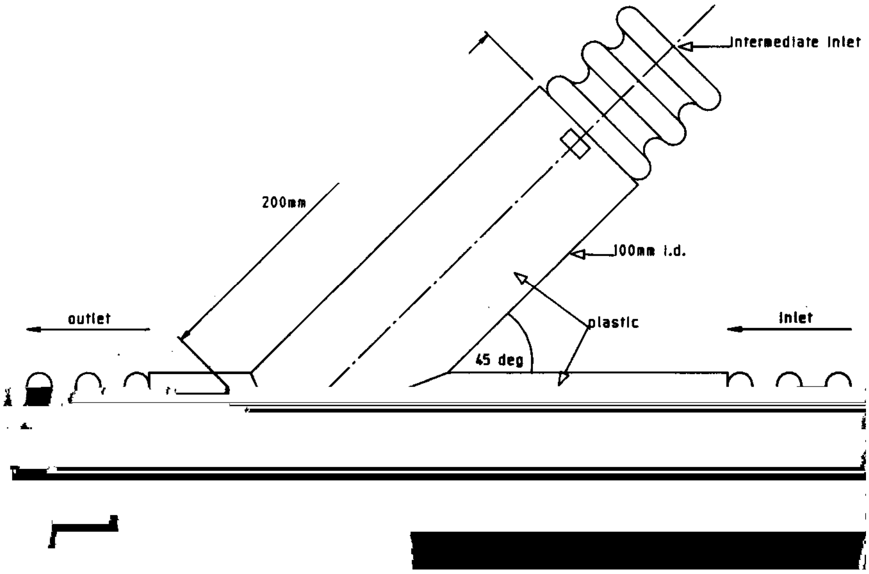


Figure 5.4: Typical flushout riser

Maximum spacing between a flushout riser and an outlet is 120 metres for inspection and flushing. In cuts where the distance to outlet may be much greater, intermediate pits should be placed with a maximum spacing of 120 metres.

Outlets should be in areas that are not subject to siltation or scouring, that are easily accessible and, where possible, visible to personnel standing on the road surface. Also, if possible, an outlet should not hinder road maintenance activities such as cleaning unlined table drains or grass cutting. Separate outlets for subsurface drains (Figure 5.5) are preferred for maintenance purposes. Outlets in pits require the pit lid to be removed for maintenance and outlets through culvert endwalls may not so readily accessible.

Outlets should be provided with some form of erosion protection commonly. Typically, this consists of either:

- a masonry or concrete apron: or
- an area of large aggregate to dissipate the outflow energy.

The outlets in Figure 5.5 are examples of poorly constructed and / or maintained outlets.

Figure 5.5: Subsurface drain outlets

5.9 Materials

Subsurface drains can be manufactured from a range of materials but all require some form of perforation to allow subsurface water to enter the pipe.

5.9.1 Corrugated Plastic

Corrugated polyethylene agricultural drain is the cheapest material. Class 1000 is used in areas subject to traffic, construction or maintenance vehicles. Pipe with a diameter of 90 mm is regarded as the minimum for roadwork.

5.9.2 Smooth Plastic

Smooth polyvinyl chloride (PVC) pipe is used to convey flows across pavement, or may be used where longitudinal gradients are flatter than 0.5 per cent. Since this material is expensive, 'herringbone grading' of corrugated pipe, that is, independent grading at slopes steeper than 0.5 per cent, may offer an alternative. Pipes sizes typically range from 100 mm diameter to 300 mm diameter. A pipe diameter of 100 mm is the minimum recommended.

5.9.3 Prefabricated Drain

Prefabricated polyethylene (PE) drain, also known as fin drain, may be laid in batters or parallel to the pavement to intercept groundwater. It may be used across a pavement if the trench is backfilled with no-fines concrete. This material has less hydraulic capacity than the corresponding diameter of pipe, so this may have to be checked. Depths of the fin drains are typically 200 mm to 450 mm in depth.

5.9.4 Steel or Fibre-Reinforced Concrete

Concrete pipes would be used where groundwater flows require diameters not available in plastic pipes. Pipe sizes in common use range from 300 mm to 750 mm in diameter.

5.9.5 Perforated Corrugated Steel

Perforated corrugated steel may be used for deep cut-off drains where the soil and groundwater are not highly corrosive. They require specific structural design.

5.10 Filters

Whether the drainage system is a vertical trench or a horizontal blanket, it is generally accepted practice that either a granular filter or a synthetic filter fabric is an essential element of the drain. A synthetic fabric may surround a granular filter.

The design of the filter material should avoid clogging with fines from the adjacent material. It is also necessary to ensure that the size of the filter material and the openings in the drainage pipe are such that entry of filter material into the drainage pipe will not occur. The filter material has to allow both the flow from the subbase and the flow through infiltration areas at the edge of pavement to be collected by the drainage pipe near the bottom of the trench. The manufacture of filter materials to meet these requirements means that they constitute a relatively expensive portion of the total cost of a drainage system.

Design procedures are not required for filters in fine graded soils such as silts and clays, and a coarse, washed sand is usually specified. Any material that would prevent migration of such particles would not posses sufficient permeability, and migration of particles is mainly prevented by the cohesion of the soil.

5.10.1 Granular Filters

Ranges of gradings are presented in Table 5.1 to cover the materials suitable for granular filters:

- Type A grading is for use with natural soil, fabric filters and pavement materials; and
- Type B grading is for use as second stage filters to Type A or with fabric filters.

5.10.2 Filter Fabrics

Filter fabrics can be categorised according to the method of manufacture. Woven, knitted and non-woven types are available, the non-woven types being further divided into needle punched, chemically bonded and heat melded types (see also Austroads, 1990).

Filter fabric design criteria are based on the equivalent opening size (EOS) and apparent open area (AOS). The EOS (O_{95}) is defined in AS 3704 as the particle diameter for which 95% of material is retained by the fabric. The AOS is defined in AS 3704 as the particle size for which a specified percentage would be retained by the fabric. Woven filter fabrics generally are available in the range of EOS values between 150 to 600 μm. Non-woven fabrics range from 20 to 500 μm with coefficients of permeability ranging from about 0.04 to 1.0 x10^2 m/s.

Table 5.1: Type a filter gradings for natural soils (VicRoads 2004b)

Percentage Passing Sieve Aperture (mm)	A1	A2	A3	A4	A5	A6
	Dune Sand	Coarse Washed Sand	Coarse Washed Sand	5 mm one-size	6-8 mm one-size	Sandy Gravel
37.5						100
26.5						ns
19.0					100	85 – 100
13.2					90 – 100	ns
9.5		100	100	100	70 – 100	65 – 100
4.75		90 – 100	90-100	70 – 100	28 – 100	45 – 82
2.36	100	75 – 100	70 – 100	0 – 50	0 – 28	30 – 60
1.18	95 – 100	50 – 98	40 – 65	0 – 10	0 – 8	15 – 40
0.6	70 – 98	30 – 80	12 – 40	ns	ns	5 – 25
0.3	30 – 60	10 – 40	0 – 16	0 – 5	0 – 5	0 –10
0.15	0 – 12	0 - 7	0 – 4	ns	ns	0 – 5
0.075	0	0 - 3	0 – 3	0 - 3	0 - 3	0 – 3
Proposed Use	Silts and Friable Clay	Silts and Friable Clay	Sandy Silts	Fine to Medium Sand	Coarse Sand	Sandy Silt
Max. pipe slot size	0.4 mm	0.6 mm	1.5 mm	3.0 mm	3.3 mm	5.0 mm
Second stage filter	B1	B2	B3	B4		B3, B4

ns: not specified

Where

0_m diameter of holes in a fabric.

0_{90m} diameter of hole that is greater than 90% of the holes in the fabric (i.e. 90% of the holes in the fabric have a diameter less than this value).

D_s diameter of particles in a soil adjacent to the fabric.

$D85_S$ soil particle diameter that is greater than 85% of the diameters of the soil particles fabric (i.e. 85% of the soil particles have a diameter less than this value).

POA Percentage open area.

For the selection of filter fabrics to act as filters against silty sands the following guidelines could be used:

0_{90m} less than 1.0 x 10_{90s} (for non woven types)

0_{5m} less than 0.2 mm (for woven types)

POA less than 10 per cent.

However, in Australia, most materials in contact with a subsurface drain contain clay particles. Only non-woven filter fabrics are suitable for use against cohesive materials. The following guidelines apply:

0_{5m} is greater than $D85_S$ (i.e. the diameter of at least 85% of the soil particles must be greater than 95% of the holes in the fabric),

and

0_{50m} is less than 70 μm.

Table 5.2: Second stage filter gradings (VicRoads, 2004b)

Percentage Passing Sieve Aperture (mm)	B1	B2	B3	B4
	5 mm one-size	6 – 8 mm one size	10 mm one size	19 mm one-size
26.5				100
19.0		100	100	70 – 100
13.2		90 – 100	90 – 100	0 – 30
9.5	100	70 – 100	40 – 70	0 – 10
4.75	70 – 100	28 – 100	0 – 15	ns
2.36	0 – 50	0 – 28	0 – 5	0 – 5
1.18	0 – 10	0 – 8	ns	ns
0.6	ns	ns	ns	ns
0.3	0 – 5	0 – 5	ns	ns
0.15	ns	ns	ns	ns
0.075	0 - 3	0 - 3	0 - 3	0 – 3
Proposed use	With Type A1	With Type A2	With Types A3, A6	With Types A4, A6
Max. pipe slot size mm)	3.0	3.3	9.0	15.0

ns: not specified

5.10.3 Aggregate Filled Drain with Filter Fabric

The filter fabric should be installed in such a manner that all splice joints are provided with a minimum overlap of 500 mm. The closure of the fabric at the top of the trench should provide two thicknesses of fabric over the top of the trench firmly secured with mechanical ties. Where an outlet pipe passes through the fabric, a separate piece of fabric of sufficient size to be wrapped around the pipe and flared against the side of the fabric of the drain should be used.

Field splices of filter fabric should be anchored with securing pins as directed, to ensure the required overlap is maintained.

Care should be taken during backfilling and pipe installation to prevent damage to the filter fabric. The aggregate comprising the backfill should be compacted by the use of a vibratory compactor to the satisfaction of the engineer. All compaction should be complete before making the filter fabric closure at the top of the trench.

5.11 Lowering of Ground Water Table

The lowering of a static water table is achieved through the use of a system of vertical trenches below the road pavement (see Figure 5.6).

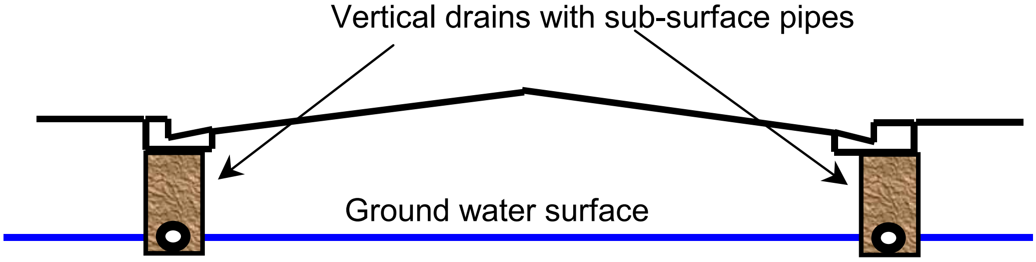


Figure 5.6: Typical subsurface drainage system

Table 5.3 is used for a preliminary assessment of the effectiveness of a proposed trench drainage system. However, if the subgrade permeability is less than 100nm/s then vertical drains on both sides of the roadway are unlikely to be effective in lowering a water table. An alternative solution to subsoil drainage should be adopted (e.g. pavement design based on saturated subgrade strength).

Table 5.3: Effectiveness of trench drainage systems (after Golden 1979)

Coefficient of Permeability, k (nm/s)	Amount of Lowering in (m) after different periods midway between two 1 m deep trenches (initial water table 1 m above bottom of trenches)					
	Trench Spacing 3 m		Trench Spacing 10 m		Trench Spacing 20 m	
	3 months	1 year	3 months	1 year	3 months	1 year
10000	1.00	1.00	1.00	1.00	1.00	1.00
1000	1.00	1.00	0.93	1.00	0.48	0.93
100	0.94	1.00	0.23	0.65	0.06	0.23
10	0.25	0.68	0.03	0.10	0.00	0.03
1.0	0.03	0.11	0.00	0.00	0.00	0.00
0.1	0.00	0.00	0.00	0.00	0.00	0.00

5.11.1 Schilfgaarde's Method

Schilfgaarde's Method can be used to determine the drain spacing that will lower the water table by m_o - m (Figure 5.7). The accuracy of the solution is extremely dependent on the accuracy of k and f (i.e. permeability and drainable pore space of the subgrade). A typical sub-surface pipe arrangement is shown in Figure 5.7 with the parameters for a numerical solution shown. The method is shown diagrammatically in Figure 5.8.

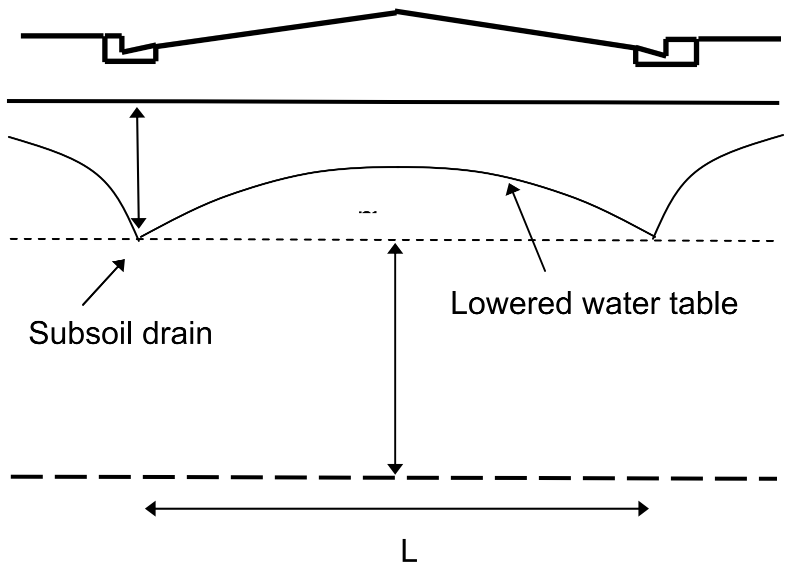


Figure 5.7: Geometry of drainage problem (after ARRB Special Report, 1987)

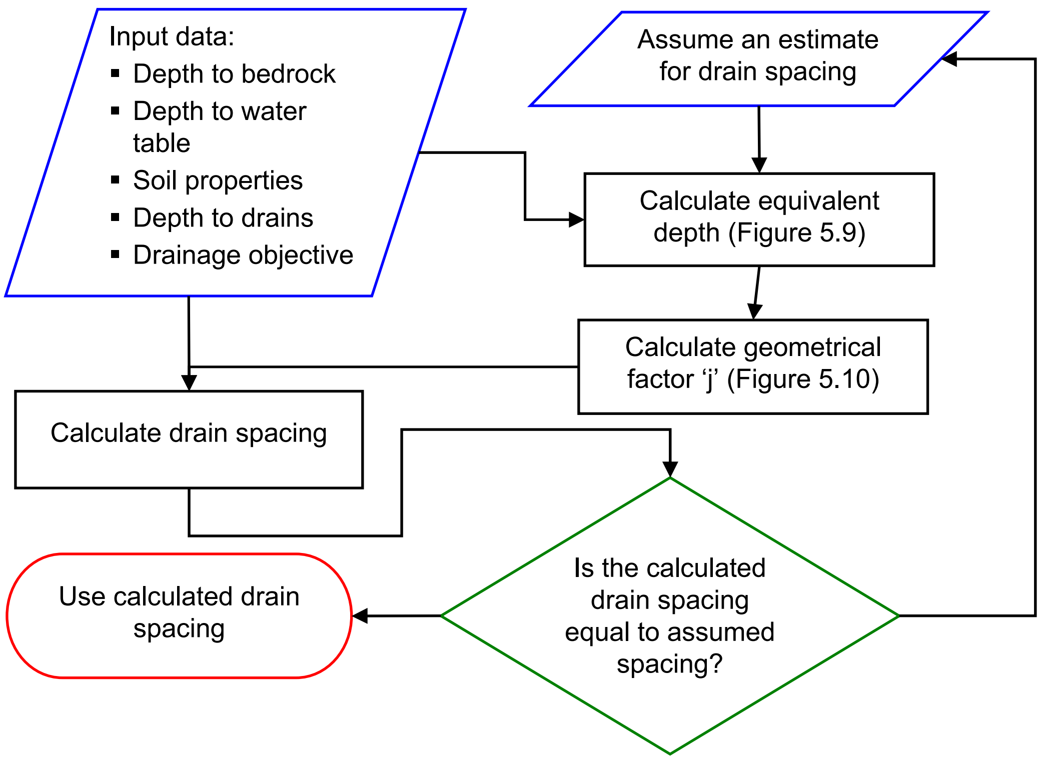


Figure 5.8: Flow chart of Schilfgaarde's method

$$L = 3j \times \left(\frac{k(d_e + m_o)(d_e + m)t}{2f(m_o - m)} \right)^{\frac{1}{2}}$$ Equation (5.1)

where

S Spacing between drains (m),

j geometrical factor (determined from Figure 5.10),

k saturated permeability (m/s),

m_0 depth of drain below original water-table (m),

m depth of drain below lowered water-table (m),

m_0- m distance water-table is lowered (m),

d height of drain above impervious barrier (m),

d_e equivalent depth of drain to impervious barrier (m). Differs from 'd' because of convergence of the flow lines,

f drainable pore space, expressed as a fraction of total volume drained at 600 mm tension (typically clays range from 0.03 to 0.11, well structured loams from 0.10 to 0.15 and sand range from 0.18 to 0.35),

t time to lower water table (sec).

The solution requires that a starting estimate of ‘L’ (drain spacing) be input with known values of d, m_o, m, k, t and f. The equivalent depth, d_e is estimated from (Figure 5.9). This is then used to calculate a convergence factor as shown in Equation (5.2). (See worked example C.1 in Appendix C.1 and a flow chart of the procedure (Figure 5.8).

Convergence Factor = $d_e/(d_e + m_o)$ Equation (5.2)

The convergence factor is then used with Figure 5.9 to estimate ‘j’. ‘L’ is recalculated and if different from the initial guess, a further iteration of calculations is commenced with the revised equivalent depth.

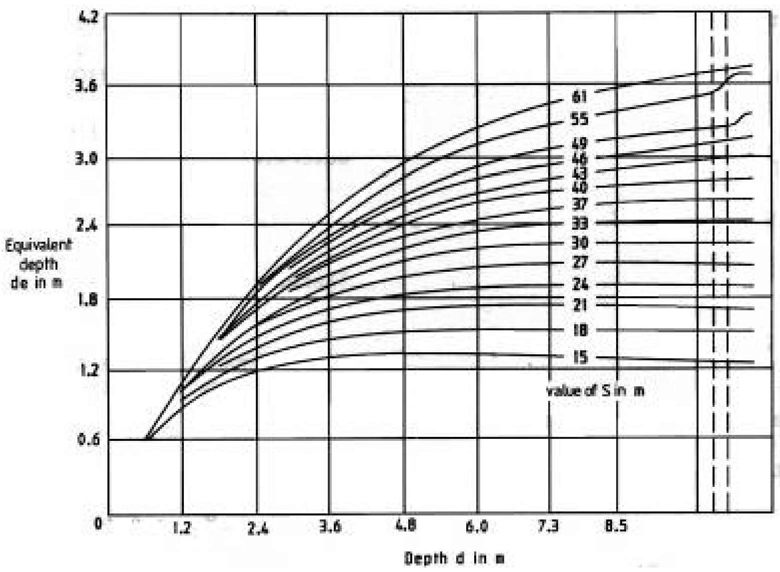


Figure 5.9: Equivalent depth for convergence correction (after ARRB Special Report 35, 1987)

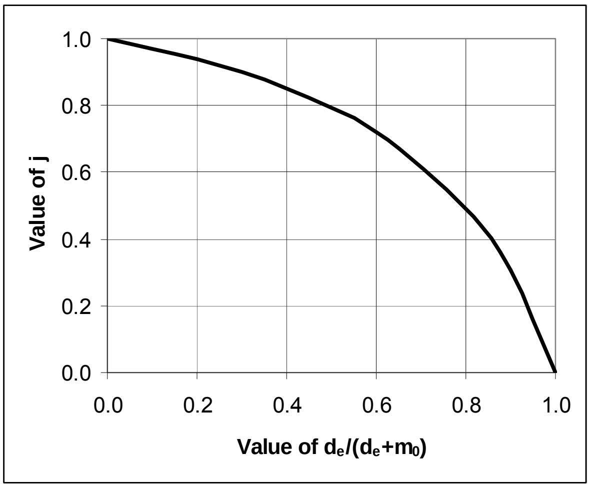


Figure 5.10: Dependence of factor j on depth to impervious layer (after ARRB Special Report 35, 1987)

5.12 Draining an Inclined Aquifer

The design of a sub-surface drainage system which intercepts an inclined aquifer (see Figure 5.10) is relatively straight forward. Darcy's Law, shown as Equation (5.3), governs the discharge from the aquifer.

$$q_m = kAi \qquad \text{Equation (5.3)}$$

where

q_m discharge per unit length of trench (m^3/m),

k permeability of the aquifer (m/s),

A area of aquifer (but in this Equation is equal to the thickness, since the discharge required is per unit metre of length (m^2)),

i slope of the aquifer (when the piezometric heads within the aquifer are equal) (m/m).

To ensure that the subsurface drainage systems intercepts all of the seepage, the permeability of the filter material and the width of the trench need to be checked. This implies that the piezometric head must drop to zero within the trench filter material. The principle is shown mathematically below.

Tan (piezometric gradient in trench) / Tan(slope of aquifer) = the ratio of the permeability of the aquifer material divided by the permeability of the filter material. Using the nomenclature in Figure 5.11;

$$\tan(B)/\tan(A) = k_a / k_f \qquad \text{Equation (5.4)}$$

Tan(B) can be approximated by W/T (width of the trench divided by the thickness of the aquifer),

Tan(A) can be approximated by the slope of the aquifer (shown as 's', in Figure 5.11),

k_a permeability of aquifer material (m/s),

k_f permeability of filter material (m/s).

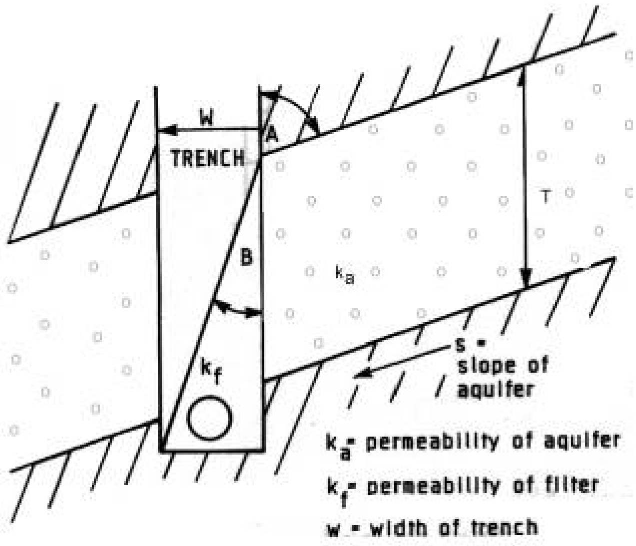


Figure 5.11: Trench excavated through an inclined aquifer (after ARRB Special Report 35, 1987)

There are two values, which can be altered by the drainage designer (W or k_f) to balance Equation (5.4). However, trench width is normally fixed to a standard value (typically 300 mm) and so it then becomes a case of selecting filter material to ensure that the ratio tan(B)/tan(A) is less than k_a/k_f. (See worked example in Appendix C.2.)

5.13 Design of a Filter Blanket to Lower a Water Table

Where a pavement is to be placed within or below the natural ground water level it may be necessary to lower the water table. This can be achieved by placing a horizontal filter blanket below the pavement. The design of the filter blanket should be undertaken using analytical procedures such as flow net procedures, finite element methods, etc. These analytical procedures are beyond the scope of this document.

5.14 Design of Cut-Off Drains

Simple analysis consisting of homogeneous layers of differing permeability rarely applies to natural conditions. Fissures, joints, faults and bedding planes in soil or rock structures can have large hydrostatic head differences over short distances that may vary rapidly. Strategic placement of piezometers and standpipes is therefore of the utmost importance. Theoretical models can give good results only if the ‘ground conditions’ input during design, are close to those in the field.

Road surfaces are more permeable than generally imagined, and the quantity of water entering a pavement (infiltration rate), may be estimated by multiplying the infiltration coefficient (Table 5.4) by the 2 year, 1 hour rainfall intensity over the surface area.

Table 5.4: Surface infiltration coefficient (after CRB, 1982)

Surface Type	Infiltration Coefficient
Sprayed Seal	0.2 –0.25
Asphalt	0.2 – 0.4
Cement Concrete	0.3 – 0.4
Unsealed shoulders	0.4 – 0.6

Once the infiltration rate is estimated (in m/s), and the coefficient of permeability has been determined by laboratory testing, the quantity of water entering the road or the inflow is determined by applying Darcy's Law (see Equation (5.3), where:

q quantity of water entering the surface (m^3/s),

k coefficient of permeability or infiltration rate (m/s),

i hydraulic gradient, i.e. head of water divided by length of drainage path (dimensionless),

A area of pavement (taken as one metre square in this application).

A hydraulic gradient of unity is suggested for rain falling on a surface. With a hydraulic gradient of unity the inflow as calculated from Equation (5.3) is equal to the infiltration rate multiplied by the surface area of the pavement.

5.15 Capillary Rise in Soils

Where a shallow formation is proposed over saturated ground, or fine-grained embankments crossing swamps, the height of capillary rise of the groundwater should be calculated to ensure that excess water does not enter the pavement. The rise in capillary water can be calculated using Equation (5.5). See example in Appendix C.3.

$$h_c = I0C/eD_{10} \qquad \text{Equation (5.5)}$$

where

h_c capillary rise (mm),

C an empirical constant that depends on the shape of the grains and varies from 0.1 to 0.5 cm^2 (For perfect spheres, C = 0.1 cm^2),

D_{10} Allen Hazen's effective grain size based on the sieve opening in cm that 10% of the material passes. The value is obtained from the grading curve.

$$e \qquad \text{void ratio} = \frac{V_V}{V - V_V} \qquad \text{Equation (5.6)}$$

where

V total volume (units),

V_V total volume of voids (units).

5.16 Size of Drain

Having estimated the amount of ground water from the previous sections in this chapter, it is only necessary to determine a suitable drain diameter. The grade line of the drain is generally known. There a number of design charts available which can be used to size the drain given the slope and the volume that the drains are required to carry. However, to avoid the system failing due to partial blocking of the drain, the drain should be designed to carry at least three times the expected flow. The minimum diameter for road works is 90 mm.

5.17 Construction

5.17.1 General

The construction of subsurface drains has an influence on the correct operation of the system. During placement and compaction of filter materials, fines can be washed into the subsurface drain. These need to be cleaned out as soon as practical after the compaction process is complete. This will also show if the drains have been crushed during construction.

A flushout of the subsurface drains at the completion of installation and backfilling should be considered part of the construction process. Water is pumped into the inlet point and the discharge at the outlet is then observed. If the subsurface drains have been correctly installed, the discharge should run clear within a few minutes. Once the discharge is clear, pumping is halted and rodent screens installed.

5.17.2 Excavation

The following are some guidelines to be observed during the excavation of trenches:

- the grade of the bottom of the trench shall be even throughout
- material excavated from the trench should be removed before placing filter materials to avoid contamination
- ensure that are no irregularities in the trench that could impede the flow of water.

5.17.3 Storage Capacity

Typically, some water is held in the subsurface drains during the flushout process. The difference between the volume pumped into the subsurface drains and the volume that is discharge is known as the storage capacity of the system. The volume of water used in the flushout process must exceed the storage capacity so as to ensure that there will be a discharge at the outlet.

The storage capacity depends upon the length of the system, the grade of the pipes and the quality of the construction. Some typical storage capacities are as follows:

- Well constructed drains on 1% grade – 300 to 500 L per 100 m of drain;
- Poorly constructed drains (20 mm depressions throughout the excavated trench) – 500 to 1000 L per 100 m of drain.

5.17.4 Compaction of Backfill

It is necessary to compact trench backfill so that it does not subside at a later date leading to possible disruption of the pavement structure or a reduction in the efficiency of the drains. Mechanical compaction is required as flooding of granular fills will not achieve sufficient levels of densification. The following guidelines should be used to select the most appropriate compaction program:

- fine to medium sands – low frequency / high amplitude compactors in conjunction with saturation.
- coarse permeable filter - high frequency / low amplitude compactors (i.e. vibrating plates) with some wetting of the materials.

The trench should be back filled in layers suited to the efficiency of compaction apparatus.

5.18 Maintenance

Subsurface drains are an essential component in ensuring pavement life and performance. As for any other component, it is necessary to review the performance of the subsurface drains on a regular basis particularly during operation.

When operating correctly, the discharge should be clear or slightly discoloured. If the time taken for the flushout water to reach the outlet is significantly longer that that noted during flushout after construction then some remedial work may be required.

If the capacity of the subsurface drains are inadequate or expensive to maintain, then additional drains should be considered to supplement the existing system or the pavement redesigned to suit the more saturated conditions.

5.18.1 Inlets

Inlets should be inspected on a regular basis, typically about once a year. Remedial action should be taken to prevent surface water entering inlets to the subsurface drains. Damage to the inlet structures should be noted and scheduled for repair as soon as possible.

It is recommended that records be kept of the location of all inlets and that marker posts be placed at inlets.

5.18.2 Outlets

Subsurface drain outlets should be inspected on a regular basis, typically about twice per year and one of the inspections should be timed to coincide with a rainfall period. Any blockage of the outlet should be noted and cleared away.

If the outlet is causing degradation of the surrounding area it should be resited or additional works scheduled to stabilise the outlet. If an outlet needs some form of remedial treatment at every inspection then consideration should be given to either relocating the outlet in a more stable location or to stabilising the existing outlet.

It is recommended that records be kept of the location of all outlets and that marker posts be placed at outlets.

It is recommended that rodent protection be fitted.

5.18.3 Flushouts

It is recommended that the subsurface drain system be flushed out immediately after construction. The next flushout should be scheduled to coincide with the inspection of the inlets. Additional flushouts should be scheduled based on whether the subsurface drainage system is stable (i.e. outlets should little signs of transport materials, outcome of the previous flushout).

CHAPTER 6

6 SURFACE FLOWS

6.1 Introduction

6.1.1 Scope

This chapter describes the procedures recommended for the selection and design of components to collect stormwater from within the road reserve. A worked example of a minor storm water drainage system is included in Appendix D. Readers should note that this is a detailed and lengthy example.

Estimation of flows is covered in Chapter 3 — Obtaining design flows.

The hydraulics of bridge waterways is a specialised topic dealt with in Waterway Design (Austroads, 1994).

6.1.2 Objectives

Stormwater management must achieve multiple objectives. For road safety and structural integrity, the objective is to remove water from the carriageway as quickly as possible. An environmental objective is to retain water in the catchment so as to emulate pre-development conditions, however, a balance must be struck in the overall system.

Typical objectives are to:

- provide convenience and safety for traffic and pedestrians in frequent or nuisance stormwater flows
- ensure that flows during major events are not impacting on development
- provide for public safety by ensuring that design velocity/depth conditions in all readily accessible open channels are below safe limits
- ensure that all drainage appurtenances are structurally adequate and provide for public safety
- regulate outflow from the catchment to levels which approximate those of its pre-developed state
- ensure pre-development levels of water flow are retained within waterways including groundwater systems
- design and construct drainage facilities realising that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users.

6.2 Elements of Surface Drainage Schemes

On rural road reserves and more frequently in water sensitive urban drainage schemes, the drainage system essentially consists of table drains and catch drains that convey surface water to the cross-culverts. Divided roads with a wide median may have an open drain in the median.

In some cases, floodways may be used to pass flows of higher average recurrence intervals.

6.2.1 Preferred Cross-sections for Surface Drains

The cross section of surface drains need to be designed to minimise the hazard posed to wayward vehicles. Where possible, surface drain cross sections should provide a traversable profile that gives wayward vehicles a chance to recover and return to the roadway. The slope of the drain batters greatly influences the ability of a wayward vehicle to return to a traffic lane. Within this zone there should ideally be no obstacles that would impede vehicles.

The change in elevation will affect the severity of accidents. High embankments or deep drainage channels are likely to increase the severity of accidents compared to low embankments and shallow channels.

Surface drain cross-sections that comply with this concept are described by the shaded areas in Figure 6.1 & Figure 6.2. Figure 6.1 applies to all V-shaped drains, trapezoidal drains with less than 1.2 m width across the bottom and parabolic drains of less than 2.4 m bottom width. Trapezoidal drains that have a width equal to, or more than 1.2 m across the bottom, or parabolic drains with a bottom width of at least 2.4 m are checked against Figure 6.2. Fore-slope is a downward slope and back-slope is an upward slope.

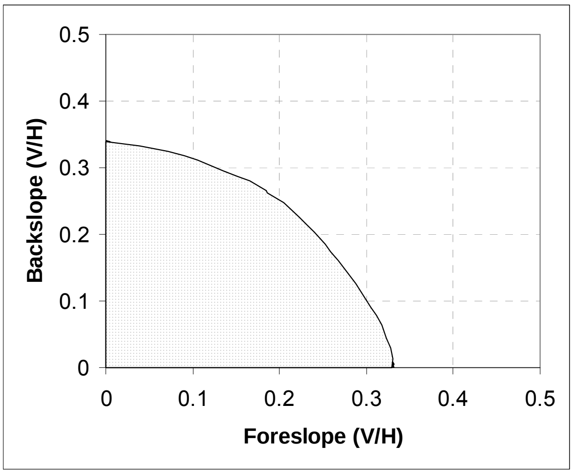


Figure 6.1: Preferred cross sections for drains with abrupt slope changes (V drains, trapezoidal shaped drains with a bottom width of less than 1.2 m or parabolic drains with a bottom width of less than 2.4 m) (from AASHTO 2002)

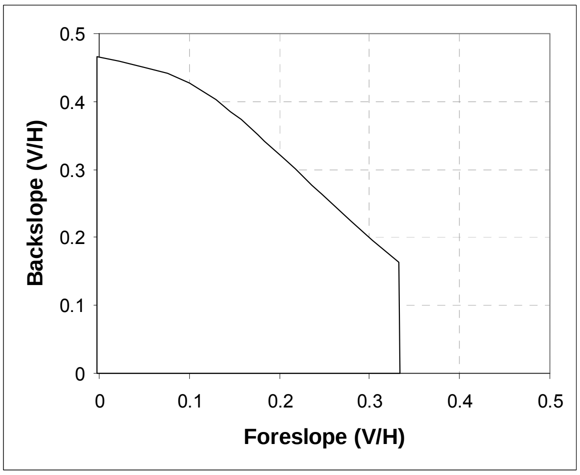


Figure 6.2: Preferred cross sections for drains with gradual slope changes (trapezoidal wider than or equal to 1.2 m across the bottom or parabolic drains wider than or equal to 2.4 m) (from AASHTO 2002)

Drain configurations that don't comply with the shaded zones in Figure 6.1 & Figure 6.2 are considered non-traversable. However in reality there are a number of factors that influence the ability of designers to provide the batter combinations. These restrictions are: economic, earthworks, impact on existing vegetation and limits on available right of way.

6.2.2 Table Drains

Table drains (see Figure 6.3) are located along the outer edge of shoulder in cuts, and beside shallow raised carriageways in fill. They collect water from the pavement, shoulders and cut batters and convey it to a suitable turnout, watercourse or culvert.

The invert of the table drain should be at least 150 mm below the bottom of the pavement to protect the structural stability of the pavement formation. Where subsurface drains discharge into a table drain, the invert depth should be deep enough to allow the subsurface water to drain away. Depth of the drain will also depend upon the design capacity required to safely convey the volume of runoff.

Where a table drain has been widened as a source of borrow material, it should be graded to a suitable outlet so that water does not pond against the road formation. The invert depth should be at least 150 mm below the formation as noted above (Underwood, 1995).

The desirable minimum longitudinal slope is 0.5%. Drains steeper than 1% may need scour protection, depending on the erodibility of the soil and the vegetative cover.

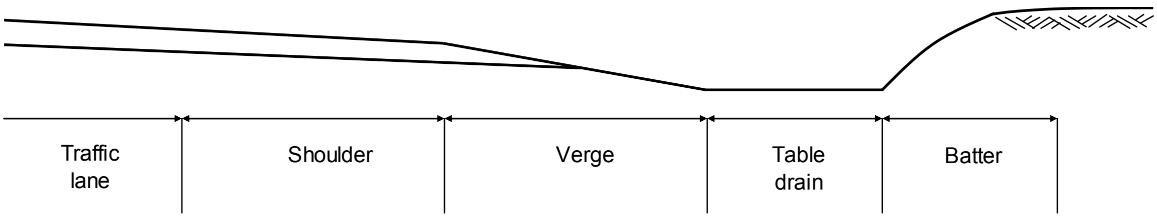


Figure 6.3: Typical table drain (adapted from Austroads, 2000)

Table drains can also function as swales and this requires that they also conform to the principles outlined in Chapter 8 - Water Quality and Erosion.

6.2.3 Catch Drains

Catch drains (sometimes known as cut-off drains, see Figure 6.4) intercept the surface water at the top of cut batters in order to prevent rilling, erosion or scouring of the batter slopes. This type of drain is usually about 0.3m deep but capacity should also be checked. Alternatively, catch drains placed at the bottom of fill slopes intercept water from adjacent properties as well as convey road drainage to an outlet. This type has a flat bottom 2.0 to 2.5 m wide and a depth sufficient to carry the design flow.

In erodible soils, the catch drain in cut may take the form of a low levee bank along the top of the batter. In such soils, a drain cut into the surface may rapidly erode and enlarge itself, or cause local slips in the batter by piping. V-shaped drains are not preferred and should not be used in erodible soils.

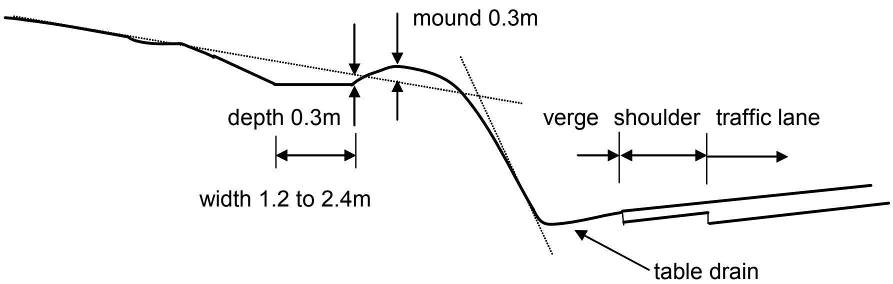


Figure 6.4: Typical catch drain

6.2.4 Median Drains

The main limitation on median drains relates to safe slopes for wayward vehicles (see Section 6.2.1). The desirable side slope of 10 per cent or flatter severely restricts the capacity of such drains unless the median is very wide. Road safety requirements may result in the median drain being augmented by grated pits and underground pipes.

6.2.5 Kerb and Channel in Cut

Kerb and channels can be used at cut locations instead of table drains (see Figure 6.5). Circumstances where this may be appropriate include where there is a need to:

- limit the width of cut to the available right-of-way
- restrict the amount of cut in order to balance earthwork quantities
- protect the formation against scour on steep grades.

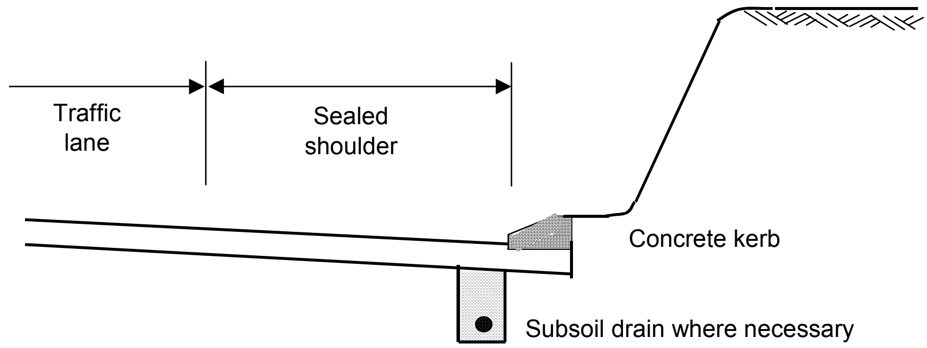


Figure 6.5: Typical use of kerb and channel in cut (adapted from NAASRA,1986)

6.2.6 Culverts

Cross drainage of road reserves is commonly achieved by means of one or more pipes, known as culverts. These are designed to convey floods with a high ARI. Where available height is restricted, strong rectangular units called box culverts may be placed close to the road surface. Hydraulic design of culverts is covered in Chapter 7 - Culverts & Energy Dissipators.

6.2.7 Floodways

A floodway consists of a length of sealed pavement on a level grade such that floodwater crosses the road at a constant depth. The facility must be identified by advance warning signs, and by water depth markers along the floodway. Hydraulic design of floodways is covered in Waterway design (Austroads, 1994).

6.2.8 Bridges

Where the catchment upstream of the road crossing is large, the construction of a bridge may be justified. Hydrological studies for bridges are beyond the scope of this Guide, and are covered by Waterway design (Austroads, 1994).

Where scuppers are used they should not discharge over the navigable parts of a stream or on to any erodible surface (see also AS 5100). For bridges over roads, railways or other facilities, flows from the deck should not discharge directly on the facility below. Drainage should be intercepted by pits placed in the verge of the road near the end of the deck. Such pits should be located so as to avoid guard-fence posts. However, any discharge into a receiving water, including that from bridge decks would need to comply with local requirements.

The width of flow on a bridge deck should not exceed that specified for its road approaches. Where the bridge cannot be drained by scuppers, and the flow width would be wider than allowable, or the average grade is less than 0.5 per cent, kerb inlets shall be provided and a drainage pipe installed to convey flow to pits near the bridge abutments.

6.3 Drainage Elements For Kerbed Roads

For kerbed roads, underground stormwater drainage systems convey surface stormwater through inlet pits and a network of pipes to an outlet. The road drainage system forms a part of the overall urban drainage system, and should be compatible with the requirements of the local regulatory authority. Major runoff flows from rare storms are conducted by a series of roadway reserve, drainage easement and open space or 'green belt' areas of a developed **urban** landscape.

As arterial roads are commonly designed to a higher Average Recurrence Interval than the adjacent system, careful attention is required to the control of overflows where the road and local system connect. In some cases, the additional volume may have to be stored for a period. Design of detention basins for this purpose is covered in Chapter 8 - Water Quality and Erosion.

The shift towards water sensitive design principles employs many of the elements previously described under Section 6.2. Conveyance of water in open channels has many water quality benefits. Water quality control and design of water treatment measures are discussed in Chapter 8 - Water Quality and Erosion.

6.3.1 Kerbs and Channels

Kerbs serve to delineate the road edge as well as to retain stormwater. Typical kerb types include barrier kerbs, semi-mountable kerbs and fully mountable kerbs.

Selection of kerb type may be based more on traffic engineering requirements than hydraulic requirements. The exception is in tropical zones, where high kerbs are used along local roads to convey the deluges that occur in the wet season.

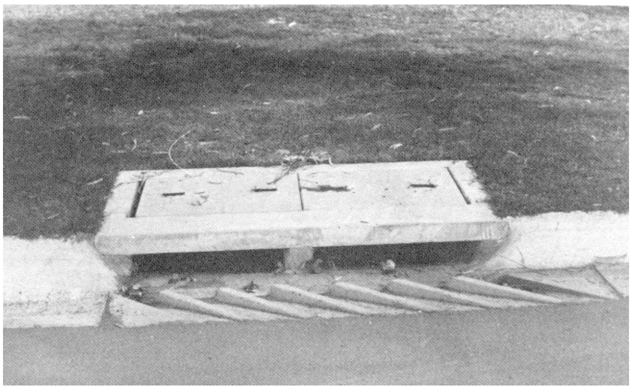


Figure 6.6: Pit inlet with gutter deflectors (after NAASRA, 1986)

6.3.2 Inlet Types and Uses

Side entry pit

Side entry pits have good entry conditions and little chance of major blockage (see Figure 6.6). To obtain maximum efficiency the throat opening may be depressed below the channel invert, and the adjacent channel slope should be steepened to direct water into the pit. However depressing the throat opening may cause a safety hazard and this should be considered. The maximum practical pit opening is about 4.2 metres and where the actual pit opening is shorter than the lintel (side entry) opening, the ledge under the lintel leading water to the pit should be sloped to avoid siltation. Where a longer opening is essential, two or more pits may be installed.

Grated pits

Grated pits are generally located clear of carriageways, such as in shoulders, medians, table drains or catch drains. The pit top is depressed at least 0.075 metres to increase inlet capacity.

Transverse steel bars must be used where pits are likely to be traversed by bicycle traffic. However, transverse bars reduce hydraulic efficiency.

As grates are prone to blockage by debris (see Table 6.10), their use in trapped low points is not advisable and are banned in some areas in Australia for this reason.

Grated side entry pits

Grated side entry pits (GSEP) may be used where the main pipe is located under and parallel to the road.

Transverse bars or patented bicycled-proof grates need to be used. On grades above 3%, the GSEP with transverse bars may have less capacity than a side entry pit, due to diversion of water from a partly blocked grate, or water overshooting the transverse bars.

GSEPs do not have significantly higher hydraulic efficiencies on grades than grated pits. Hydraulic performance of the side entry or the grate should be estimated as though they were installed separately. The component with the higher efficiency may be adopted for design purposes.

Pit locations

Various trial calculations will be required to select the most economic network of pits and pipes.

Before placing pits to control the width of flow or bypass flows, pits should be placed at essential locations such as those shown in Restrictions.

The channel capacity or the pit inlet capacity may govern locations of pits, in order to limit surface flow widths at the design ARI. It is convenient to have these both plotted on the same design chart as shown on Figure 6.13.

Restrictions

Side entry pits or grated side entry pits should not be sited where the swept paths of long vehicles may cause damage to the lintels or kerb covers.

Depressed inlets should not be used for pits adjacent to median kerbs. This will create localised depressions in the vehicle wheel path. For similar reasons, this type of inlet should be avoided in near side traffic lanes with no shoulder, regardless of what the hydraulic design and pipe locations might dictate.

When selecting bar spacing and orientation for grated inlets, bicycle and pedestrian safety, hydraulic efficiency and potential for debris collection should be considered.

Pit Sizes

Where a pipe meets a pit wall at an angle, it may be necessary to increase the pit size to accommodate the skewed diameter of the pipe plus a clearance to the pit wall.

For pipes larger than the standard pit dimensions, the size of the whole pit should be increased where the depth is less than 3.6 m. Deeper pits may be ‘haunched’, that is the bottom chamber is larger than the shaft above it.

Pits should be kept as shallow as possible, allowing for the need to connect the subsurface drainage system beside or above the obvert of the stormwater pipe. Pit costs increase significantly when the depths exceed 1.5 m, 3.6 m or 7.2 m, due to thicker walls and construction safety requirements. Pits deeper than 1.5 m need shoring during construction.

Table 6.1: Essential pit locations and general guidelines

Location	Discussion

Low points	At all low points in the roadway, median, catch drains and/or table drains
Sag curves	To minimise risk of flooding an additional pit is required within 10 m of the pit at the low point preferably at a position where the kerb lip is not more than 60 mm higher than the kerb lip at the low point.
Pedestrian zones	Pits should be located on the upstream side of kerb returns, pedestrian crossings, bus bays, left turn lanes and/or noses of islands and ramps.
Intersections	Side entry pits or grated pits should be placed to catch flows at island noses so that water does not enter intersections or interchanges. Particular attention should be given to gore areas.
Bridges	Locate pits upstream near bridge abutments, clear of guard fence posts.
Superelevation development	A pit should be located on the low side where the cross slope is 1% or where the flow width criterion is met if this occurs sooner.
Practical maximum spacing	In order to facilitate inspection and cleaning, the maximum recommended pit spacing are: — 200 m for pipe diameters > 1800 mm — 120 m for pipe diameters ≤ 1800 mm
Ramp entries or gores	Pits should be located to minimise water on the pavement and hence reduce the risk of aqua-planing.
General	Every effort should be made to space on-line junction pits as far apart as possible. Avoid the use of angled cross connections where possible
	Inspection pits spaced at intervals of not more than 120 m should be included in all long, uninterrupted pipelines of diameter 1050 mm and smaller.
	In all gutter and 'sag' inlet pits and junction pits, design water levels assigned to pits should not be higher than the gutter invert level minus 0.15 m (where 'gutter invert level' means the undepressed gutter invert level assigned to consecutive pairs of pits on main drain or lateral pipelines should differ by not less than 0.10 m in the direction of flow).
	A 3-point priority sequence should be followed in assigning pit design water levels in accordance with the above: Priority 1: junction pits along lateral pipelines including the pits where these pipelines join with main drain pipelines. Priority 2: junction pits along main pipeline branches, where these are present. Priority 3 junction pits along the main pipeline trunk. In each priority, assignment of pit design water levels should commence at the upstream extremity in priorities 1 and 2 and at the downstream extremity priority 3.
	Pit floor levels in mainline or lateral pipeline junction pits receiving flow from cross connected inlets must allow for slope or not less than 0.01 m/m in cross-connection pipes. Pit floor level should coincide with invert of the pipe carrying discharge from a junction.

6.3.3 Grated Drains

Narrow transverse grates may be used at locations such as service station driveways or at some bridge abutments. It is essential that the grate units be locked on to their channels to prevent large trucks from dislodging them.

Grated drains should not be placed in traffic lanes.

It should be noted that grated drains tend to trap litter and vegetative matter, and so lead to high maintenance costs. If trapping rubbish is a requirement, it should be done by a trap designed for the purpose, that is easy to clean, and is fail safe in the case of blockage.

6.3.4 Manholes

Manholes are required at abrupt changes to the horizontal and/or vertical alignment of a pipeline where there is no need to admit surface water. manholes are alos required when the pipe size changes.

6.3.5 Chutes

In some cases, small flows may be conveyed directly down a batter in a chute.

The kerb or entry concrete should be suitably flared and the area near the inlet may be depressed if it is in a shoulder. Open or piped chutes may be used.

For open chutes, any necessary changes of direction must be achieved before the water reaches supercritical velocity. Otherwise overshooting and scouring will result. An example of an open chute is shown on Figure 6.7. An alternative material is half-round corrugated steel pipe.

Figure 6.7: Open chute (after NAASRA, 1986)

Piped chutes are more expensive than open chutes, but are not susceptible to overtopping and scour, and they do not constitute an obstacle within the clear zone. Polyethylene (PE) pipe has good abrasion resistance and has been successfully used for this purpose.

Some form of energy dissipation and scour protection will be required at the outlet of a chute. The outflow may be discharged to a beached natural channel, a concrete pit, or to an underground system. In dry sandy areas with a low water table, sumps or soak wells may be used.

6.3.6 Pipes

General

Pipes are used to convey stormwater in underground systems. Reinforced concrete or fibre reinforced concrete are the most commonly used materials, but more flexible materials such as corrugated steel, polyethylene, or un-plasticised polyvinyl chloride may be used in certain cases.

Where the cover available is not sufficient for pipes, box culverts may be used. The larger sizes of rectangular sections are called crown units. Rectangular sections may be more suitable for conveying larger flows where width is restricted. Rectangular sections can be placed side by side, whereas pipes should be separated by about half a diameter to allow for thorough compaction of the bedding material between them.

Where possible main drain or lateral pipelines should be aligned as follows:

- Minor roads (carriageways less than 10 m). They should connect succeeding gutter inlet pits (both side-entry and grated inlet types) located along the drainage path. The alignment should favour the carriageway 'high side' in dual-channel roads and streets. 'Low side' inlets on dual-channel minor roads should be connected to the pipeline by either inter-inlet cross-connections or by deviating the alignment if necessary. The alignment should be just within the carriageway where inlets are grated and just outside where side entry inlets are used.

- The alignment in single-channel minor roads should be on the 'low side' of the street.
- Major roads (carriageways 10 m or greater). Main drain or lateral pipelines should be located within carriageways between 1.5 m or 2.5 m from the 'high side' kerb. Gutter inlet pits should be cross connected to on-line junctions.
- The alignment in single-channel major roads should be on the 'low side' of the street.
- Concrete pipes operating without pressure may be flush-jointed. Pipes designed to operate under pressure, or placed in fill, or on soft ground where differential settlement is likely, should have rubber ring joints or an equivalent.

Minimum sizes

The minimum recommended diameter for pipes along road reserves is 300 mm. Municipalities may permit smaller diameters on private subdivisions. As a practical design rule, pipe diameters should not decrease in the direction of flow.

Culverts across the formation should be at least 375 mm diameter to facilitate cleaning or 300 mm high for a box culvert.

Under very high fills, a 750 mm minimum diameter pipe should be provided for maintenance inspection, even where a smaller pipe would be adequate hydraulically.

Cover

The desirable minimum cover over concrete pipes is 0.4 metres. On projects with large-scale earthworks where large machinery and vibrating rollers are used, this cover is usually measured from the top of the subgrade rather than from the finished surface. The most adverse loading conditions for many structures occur during road construction. For information on loading conditions, refer to the Austroads Bridge Design Code (Austroads, 1992), AS 5100, or AS 3725, *Loads on buried concrete pipes*, or AS/NZS 2566.1 *Buried Flexible Pipes.* (Note: The Concrete Pipe Association of Australia provides a design manual and a software package that conforms to AS 3725.)

The desirable minimum cover to be adopted for design purposes for box culverts is 0.1 m below the subgrade level. However, the absolute minimum cover can be 0.1 m below finished surface level, that is, the culvert may be laid within the pavement layers if necessary. The maximum cover on a conventional box culvert is 1.5 metres, and above this limit, boxes or crown units need specific structural design.

Pipe strength

Rigid pipes such as steel reinforced concrete or fibre-reinforced concrete are manufactured in Classes 1 to 12 to carry specified cracking loads (for example, see AS 1342). However, a significant part of the load capacity of the pipe in the ground depends on the bedding conditions (see AS 3725). The cover depth in embankments depend upon pipe diameter and pipe support conditions (see AS 3725). Vibrating rollers can create much greater stresses on pipes during construction than heavy trucks after the road is in service. Where heavy construction equipment will be used, pipe strength should be calculated according to the procedures in AS 3725, *Loads on Buried Concrete Pipes*.

Flexible pipes such as PVC, PE and helical coiled steel, deflect to transfer stresses into the surrounding soil. Proper bedding and side support is essential. Such installations shall be designed according to AS/NZS 2566.1, *Buried Flexible Pipes.*

Pipes on curves

For pipes 1.2 m in diameter or more, changes in horizontal direction should be made on a radius of 12 m or more. Pipes are specially made with splayed ends to achieve a change in direction. Where velocities are high, curves should be used for changes in direction of pipes 750 mm in diameter or more. The grade of the curved section is made steeper than the nominal design slope to compensate for the head loss through the curve.

Flow velocities in pipes

The minimum desirable velocity of flow is 0.7 m/s to prevent siltation. The desirable minimum grade for pipes is 1 in 250, however, the practical minimum gradient for construction is 1 in 300.

High velocity flow causes cavitation and will result in surface pitting in the pipe due to surface irregularities that reduce pressure to the water vapour limit. Where design velocities in concrete pipes exceed 6m/s, sacrificial lining should be specified. The maximum velocity in a concrete pipe is 9m/s.

6.3.7 Culverts

These are discussed in Chapter 7 - Culverts & energy dissipators.

6.3.8 Energy Dissipators

These are discussed in Chapter 7 - Culverts & energy dissipators.

6.3.9 Retention, Detention, Infiltration & Sedimentation Basins

These are discussed in Chapter 8 - Water Quality and Erosion.

6.4 Design Procedures

The degree of risk in hydraulic structures is expressed in terms of Average Recurrence Interval (ARI) or Annual Exceedence Probability (AEP).

Nuisance events are commonly characterised by a storm event that has a 50% probability of occurring or being exceeded in a given year, that is, approximately the two year ARI event. For example, underground drainage systems for municipalities may be designed for two year ARI provided that an escape path on the surface is available for larger floods. The two year event is often used to size sedimentation basins.

Rare events are represented by the event that has a one per cent AEP, that is approximately the 100 year ARI event. Structures such as bridges or spillways may be designed for a 100 year ARI. No matter what ARI is chosen for the rare event, there is always a chance that a rarer and larger event could occur, and drainage designers should make allowances for relief of floodwaters and protection of structures in the event of a surcharge.

Table 6.2 contains ARI recommended for various elements of a minor drainage system and these should be adopted unless the local drainage authority specifies different values.

Table 6.2: Recommended minimum ARI to minor road drainage system elements (after Austroads, 2002)

Item	Average Recurrence Interval (years)
National highways and arterials to cross drainage (typically roads with more than 2000 vehicles per day)	100
Cross drainage on roads other than national highways or arterials (typically roads	50

with 2000 vehicles per day or less)	
Longitudinal surface drainage (including intersections and bridge decks)[1, 2]	10
Flows over the trafficable surfaces	See Table 6.5
Permanent sedimentation basins (temporary sedimentation basins)	2 (0.5)
Wide flat pavements	1
Water quality basins	1
Urban road surface drainage	See Section 6.4.1
Temporary work sites 3	1

1. If the road is sited in a town centre, the ARI should be increased to 20 years.
2. Local government policies and the local drainage authority should be consulted for the design of minor rural road drainage schemes, since these can vary across Australia.
3. A Contractor shall provide adequate drainage to side-tracks including off road drainage in order to prevent ponding of water. Culverts shall be installed at creeks, streams and well-defined low points as necessary. Culverts shall be designed for an Average Recurrence Interval (ARI) of one (1) year.

In addition to determining the water discharge volumes by adoption of an appropriate ARI, safety needs to be considered in relation to maintaining sufficient lane width during times of rainfall. This is discussed in greater detail in Section 6.4.1.

6.4.1 Water on the pavement surface

Introduction

This section covers runoff from pavements, particularly wide pavements which pose unique difficulties in providing a safe driving surface. Of greatest concern is the hazard to vehicles due to aqua-planing when a film of water on the road surface reaches a critical depth.

Other factors that are of concern to the drainage designer are:

- reduced visibility due to spray generation
- reflectance of light from surface
- reduction in roadway width.

The first two points listed above can occur at very low levels of rainfall and may persist for sometime after the rainfall has ceased. There is little that can be done in terms of drainage aspects, but a holistic approach to the roadway needs to be adopted. The provision of a porous surfacing may greatly reduce both problems.

Reduction in the road width due to ponding or encroachment of water from longitudinal drainage facilities needs to be assessed using a risk management approach. The questions one needs to ask are:

- What are the consequences of lane closures or road closures?
- What alternative routes are available?
- What level of function is the road designed to provide in the master drainage plan for the region?

The answers to these questions will guide the drainage designer to a range of acceptable options.

Scope

This section covers those aspects of the drainage of pavements, including wide pavements and bridge decks, until the discharge water is clear of the pavement surface. Once the water enters either kerb and channel, a pipe or open channel then the design of these and other downstream works follows the principles covered in Sections 6.4.2, 6.4.3 and 6.4.4.

Items covered in this section include:

- discussion on flow path lengths
- spread of flows into trafficked lanes
- depth of flow and maximum allowable depths, and
- remedial measures.

Objectives

The main objective in drainage of pavement surfaces is to limit the water depth on the pavement during storm events to tolerable limits. The design of pavement surface drainage schemes is largely controlled by the geometric design of the pavement. Superelevation and longitudinal grades need to be carefully considered during the pavement geometric design to ensure that flow length paths are not going to lead to excessive water depths.

A further requirement is to maintain sufficient lane width to enable the road to remain passable by limiting the spread of storm waters. This is discussed in detail in Section 6.4.1.

As with all runoff from road pavements, adjacent land use needs to be considered as well as other road users such as pedestrians and cyclists. Ponding of water that could lead to sprays and splashes should be minimised to provide a more tolerable service for pedestrians and cyclists. Adjacent land use is discussed in Chapter 4 - Major urban drainage networks.

Design principles

Wherever possible, the road geometry of pavement surfaces should be designed to reduce drainage path length along and across the carriageway. This will mean that two way crossfall should be used where possible and that where the high side carriageways drain to the median, the water collected in the median should not run over into the lower carriageways.

Shoulders on high side carriageways may contribute to the volume of runoff. It is not usually feasible to provide shoulders with a reverse elevation to the adjacent carriageway elevation.

Runoff from cut batter slopes or verges need to be intercepted by drains so as not to contribute to the volume of water flowing across the carriageway.

Collection of surface water

The primary means of removing water from the road is by providing a slope to the surface at right angles to the direction of travel. This slope is known as the ‘normal crossfall’, and its minimum value depends on the roughness of the surface material. Suggested values for normal crossfall are shown in Table 6.3.

Table 6.3: Minimum values for normal crossfall (after NAASRA, 1986)

Surface Type	Normal Crossfall (m/m)
Concrete	0.020 to 0.025
Asphalt	0.025 to 0.030
Sprayed seal (nominal size 14 mm)	0.030 to 0.040
Gravel or crushed rock	0.035 to 0.040

Drainage will tend to follow the steepest slope on the surface. Where the road gradient is very flat, the steepest slope is close to the direction of the crossfall. However, where the gradient is very steep, the flow path may be almost parallel to the direction of traffic, and capture of such flows is significant on the approaches to braking areas such as intersections.

Aqua-planing

Water on a pavement surface may increase accident risk by reducing friction between the pavement and vehicle tyres, affecting both stopping distance and steering. Safety of motorists requires rapid removal of rainfall from the pavement.

Development of superelevation on multi-lane roads such as freeways on flat grades may create large areas that are nearly level in both directions. This could allow accumulation of water to a depth where tyres could begin to float on a thin film of water resulting in total loss of control of the vehicle, a phenomenon known as 'aqua-planing' or 'hydroplaning'. Limiting water depth is therefore important for road safety. Aqua-planing is a complex function of tyre pressure, tyre contact area, vehicle length and speed. Partial aqua-planing may commence for some vehicles at depths of about 2.5 mm. The critical depth to cause aqua-planing is about 4 mm and above. Small passenger vehicles may be more vulnerable than heavy vehicles.

Pavement areas that may require particular attention include:

- locations where vehicle braking can be expected, such as exit lanes, approaches to signalized intersections, or sections where queuing of vehicles are common
- existing horizontal curves with an operating speed significantly different from the surrounding curves
- wide, flat intersection areas
- roundabouts within relatively high speed environments
- wide, flat superelevation development areas.

Length of flow

Length of flow is the main factor contributing to water depth, and should be kept as short as possible. Two-way crossfall on a carriageway decreases the depth of flow across the pavement. For divided carriageways on curved alignments, the drainage from the high carriageway should be collected in the median. The flow should not be allowed to cross over onto the lower carriageway.

In sags or superelevation changes, drainage may cross the pavement and return. The drainage designer should prepare a contour plan of the section of pavement to be checked. The length of flow is the path that the surface flow must take to cross the pavement and is determined by drawing a line at right angles to the contours. The slope across this line is then used as the average slope. The drainage designer should use the longest length of flow to determine the flow path length, depth of flow and the likelihood of aqua-planning.

In steep terrain, where grades can be steep, the flow path may be more along the pavement rather than across the pavement. There are limits to the crossfall that can be applied to the pavement to reduce flow path lengths in these situations.

6.4.2 Guidelines for spread of surface flows

General

Guidelines for surface flow widths should be selected before starting design. For roads other than freeways, the guidelines should be selected in consultation with the local municipality. Recommended minimum guidelines are shown in Table 6.4 and Table 6.5.

Flows in roadside kerb and channel do not entirely conform with assumptions made in relation to open channel flow. The principal assumptions made in most open channel flow analyses are those of steady flow in long, uniform channels.

Table 6.4: Surface flow guidelines — minor drainage system elements (after Austroads, 2002)

Maximum flow widths (m) in traffic lanes for Arterial roads			
Number of trafficable lanes on carriageway	Speed Environment		
	≤ 70 km/h	70 to 90 km/h	≥ 90 km/h
1	1.0	0.75	0.5
2	1.5	1.25	1.0
3	2.0	1.75	1.5
>3	3.0	2.5	2.0
General Requirements			
General	The selected surface flow widths should allow for safe passage of through traffic, and reasonable convenience for cyclists, pedestrians, and for entry to vehicles parked at the kerb. Typically the minor drainage system is designed to cater for up to a 10 year ARI storm event (see Table 6.3).		
Freeways	Surface flows should be confined to the shoulders.		
Braking Zones	Water depth and width should be restricted near braking areas such as approaches to traffic signals or freeway ramps.		
Secondary roads and residential streets	A least one lane each way on secondary roads, and at least one lane width on residential streets should be trafficable during a two year ARI storm.		
Foot path and pedestrian crossings	Maximum allowable flow width at bus stops and pedestrian crossings is 0.5 m.		
Car parks	Flow width should be restricted to two metres in a two year ARI storm.		
Residential intersections	Cross flows of less than 0.005 m^3/s are permitted at T intersections of residential streets or at the ends of small isolated traffic islands.		
General requirements			
Arterial Roads	See values in this table for multi-lane arterials. Maximum flow depth times velocity is 0.4 m^2/s.		
Kerb-side	For pedestrian safety, the maximum depth at the kerb-side should not be greater than the top of kerb, and the product of depth by average velocity (d x V) should not exceed 0.4 m^2/s. To prevent parked vehicles from being washed away the product of depth by average velocity (d x V) should not exceed 0.7 m^2/s.		

The relatively shallow and varying cross section of kerb and channel flow is far removed from that in normal channels. The flow constantly increases downstream and flow from adjacent road surfaces approaches at an angle to the kerb and channel flow and materially interferes with it. In addition to flow from the adjacent road surfaces, there will often be cases where roof runoff in conduits will discharge into the kerb and channel, thus causing further hydraulic disturbance. However, a number of open channel formulae have been used successfully for many years.

The values shown represent the maximum flow width imposition into traffic lanes. Additional shoulder widths can be fully utilised for flow conveyance.

Table 6.5: Durface flow guidelines to major system elements (after Austroads, 2002)

Trafficability Criteria (all speed environments)

Situation	Minimum provision
Access control, no pedestrians, no parking	A minimum of 3.0 m of trafficable road to remain open in each direction.
Clearways using kerbside lane, pedestrians present, bus stops.	A minimum of 3.0 m of trafficable road to remain open in each direction.
Other, parking, pedestrians, etc.	Maximum depth equal to kerb height plus 50 mm (typically 150 mm + 50 mm) and depth times velocity ≤ 0.4 m^2/s.

Flow Width

Width of flow will determine degree of water encroachment onto the adjacent roadway, and also the extent to which it may result in a pedestrian nuisance. There are various techniques available to estimate width of flow which involve consideration of channel grade, cross slope and capacity. These generally derive from modified Manning's formula Equation (6.3), for a given roughness (n) value. It is appropriate to work directly from Equation (6.3), where for given channel conditions, the depth of flow corresponding to a given discharge can be calculated. The width of flow can then be found by geometric considerations see Equation (6.1), assuming a horizontal water surface in cross section.

$$W = W_g + \left(d_a - {W_g}/{Z_a} \right) \times Z_b$$ Equation (6.1)

where (using the terminology in Figure 6.8)

W flow width (m)

W_g width of kerb channel (m)

d_a depth of flow at kerb (m)

Z_a cross slope of kerb and channel (1 in X)

Z_b cross slope of pavement (1 in X)

See Worked example Appendix D.1.

For the design recurrence interval, flow width from the face of the kerb should not unduly affect the through traffic lanes. Rainfall intensities for the design recurrence interval will generally be such that the movement of traffic will have already been impaired, drivers will be alerted, and shallow depths on a small portion of the outside traffic lane will be neither unexpected nor constitute any form of real obstacle. When checking recurrence intervals, it is suggested that roadways need to be no more than ‘passable’. Intersections and sags will be the critical areas when checking (see Table 6.6 for maximum widths of flow).

Separate calculations may be required to determine the placement of additional pits to capture bypass flows or to control flow widths in a storm event of rare ARI.

Depth of flow

The recommended design recurrence interval for a check on hydroplaning is one year or a rainfall intensity of 50 mm/h, whichever is the greater. However, runoff collection systems and downstream drainage systems should be designed according to the ARI values recommended in Table 4.2, and Table 6.2. The approximate depth of flow may be calculated using the expression developed by Gallaway *et al*. (1979):

$$d = \frac{0.0149 \times (TXD)^{0.11} \times L^{0.43} \times I^{0.59}}{S^{0.42}} - (TXD) \qquad \text{Equation (6.2)}$$

where

d depth of flow measured from the top of the surface texture (mm);

TXD average texture depth measured by the sand patch (mm) (see Table 6.7)

L flow path length (m)

I rainfall intensity (mm/h), corresponding to an ARI of 1 year or 50 mm/hr, whichever is the greater

S average slope of the flow path (m/m) (Note: Replace the factor 0.0149 with 0.10286 if the slope 'S' is expressed as a percentage.)

See Worked example Appendix D.2 in Appendix D.

Table 6.6: Indicative values of texture depth (mm) (after Austroads, 2002)

Surface Type	Texture Depth (TXD) (mm)
Burlap drag concrete	0.5
Grooved concrete	1.2
Dense-graded asphalt	0.9
Size 14 stone seal	3.7
Open-graded asphalt	1.2

Note: These are typical values to be used in the absence of site specific data.

Braking efficiency on wet pavements reduces well before the aqua-planing threshold is reached. Therefore, depths of flow in braking areas should be less than elsewhere. Suggested maximum depths of flow are set out in Table 6.7.

Table 6.7: Maximum water depths on pavements (after Austroads, 2002)

Design Speed Environment (km/h)	Design Rainfall Intensity for Calculation Purposes (mm/h)	Surface Water Depth	
		Absolute Maximum Limit (mm)	Desirable Maximum Range[1] (mm)
All speeds	50 or that corresponding to an ARI of 1 year (whichever is the greater)	5	2 - 5

The relevant State Authority Drainage Manuals should be referred to for the maximum water depths on pavements which range from 2.5 to 5 mm depending upon the design speed.

Remedial measures

Where the flow depth is found to be unacceptable, the drainage designer may consider the following remedial measures:

- increase the rate of superelevation development
- introduce one or more crown lines
- place grated drains in the pavement.

The first two measures are road design issues, and are not covered in this Guide.

Proprietary precast grated drains have been successfully used in the tapered areas near freeway noses. It is essential that the grate units be locked on to their channels to prevent large trucks from dislodging them. Design of grated drains is covered in Section 6.4.5.

6.4.3 Open Channels

The flow in open channels (table drains, catch drains, swales, etc.) is mostly determined by application of Manning's formula see Equation (6.3) and Equation (6.4) and worked example D.3). Values of 'n' can be found in Appendix E.

$$V_{ave} = \frac{R^{2/3} S_0^{0.5}}{n} \qquad \text{Equation (6.3)}$$

where

V_{ave} average velocity in channel (m/s)

R hydraulic radius (flow area divided by flow boundary length) (m)

S_o channel bed slope (m/m)

n Manning's roughness value

$$Q = V_{ave}A \qquad \text{Equation (6.4)}$$

where V_{ave} is as above and

Q discharge (m^3/s)

A cross sectional area of flow (m^2)

6.4.4 Kerb and Channel

Capacity

The most common method used to estimate kerb and channel capacity is Manning's formula Equation (6.3). The formulation of it, modified for triangular channels (Izzard, 1946) (Equations 6.5 and 6.6), has been found to be generally satisfactory. Some authorities prefer to use the Colebrook-White formula which is considered to be superior for hydraulic design of storm conduits. It can also be used for open channel conditions. The equation is not given here, but can be found in a good hydraulic text. It is difficult to solve directly, and design charts are available (Hydraulics Research Station, 1978).

Channel capacity can be calculated for composite cross sections by using triangular flow elements using Izzard's equation (see Equation (6.5) for the triangular part of the flow and Manning's Equation (6.3) and Equation (6.4) for the remainder. Values of 'n' can be found in Appendix E. Where greater precision is required, the drainage designer should consult Clarke *et al*. (1981) for correction factors. Equation (6.3) may overestimate channel capacity by 20 to 30% for a given width of flow. In terms of width of flow resulting from a given runoff, Equation (6.3) will underestimate flow width by less than 10%. As neither of these conditions are considered to be of significant practical importance, Equation (6.3) is regarded as satisfactory.

A special form of Izzard's formula is used for most kerbs and channels that have different cross-slope and roughness compared to the laneway surface (see Equation (6.6)), the terms are defined as shown in Figure 6.8).

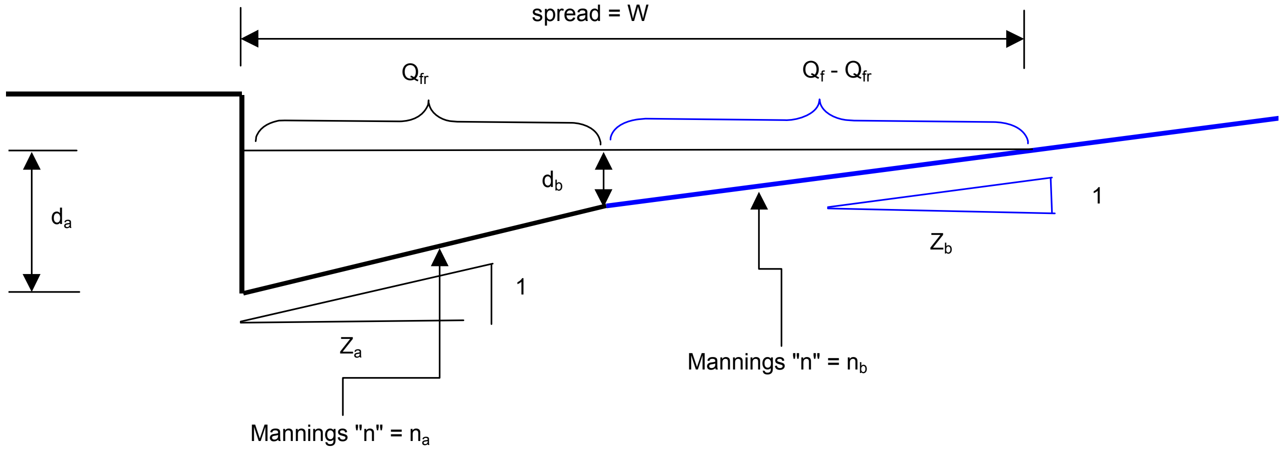


Figure 6.8: Definition of terms for kerb and channel (adapted from ARRB Special Report 34, 1986)

$$Q_t = 375\beta\left(\frac{z}{n}d^{8/3}\right)S_o^{0.5} \qquad \text{Equation (6.5)}$$

where

Q_t triangular channel flow (L/s)

shape correction factor (for simple triangular channels, factor = 0.9, for channels as shown in Figure 6.8 the factor is typically no more than 0.8)

z reciprocal of channel cross slope

n Manning's roughness value

d triangular depth of flow (m)

S_o channel bed slope (m/m)

$$Q_t = 375\beta\left(\frac{Z_a}{n_a}\left\{d_a^{8/3} - d_b^{8/3}\right\} + \frac{Z_b}{n_b}d_b^{8/3}\right)S_o^{0.5} \qquad \text{Equation (6.6)}$$

where parameters are defined in Figure 6.8 with the properties as those of Equation (6.5).

See worked example Appendix D.3 and D.4 in Appendix D.

The spread width (shown as 'W' in Figure 6.1) needs to be constrained to allow passage of users. (See Equation (6.1)and worked example Appendix D.1.)

6.4.5 Pits

Change in Vertical Alignment

Usually, drops are provided through pits (see Figure 6.9) to avoid water flowing out of upstream pits due to cumulative head losses. The drop measured between inlet and outlet pipe inverts is only equal to the head loss if the pipes are flowing full and are of equal diameter. A nominal drop of 0.1 m is usually sufficient, except for pits where major flows join, or flow direction changes, in which cases the head loss should be calculated.

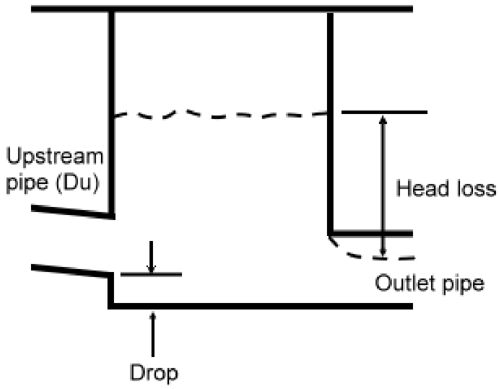


Figure 6.9: Head loss through a pit

In flat country, where the fall may not be sufficient to provide the nominal drop, the pipe inverts may lie on a continuous grade-line, provided that the bottom of the pit is shaped to match the lower half of the pipe. The hydraulic gradeline should be checked to detect any adverse effects (for method, see Section 6.4.7).

Significant changes in level through pits may be necessary in order to avoid existing public utility services, or to convey water down a batter, or as a deliberate means of reducing the energy of flow. This type of pit is called a drop pit (see Figure 6.10). Where possible, changes in elevation of pipes should be made by providing a steeper gradient between pits and/or placing pits closer together. Where the drop exceeds two metres, the pit floor should be protected by a wearing course of concrete or rock, and the outlet should be placed about 0.3 m above the floor to leave a permanent water cushion.

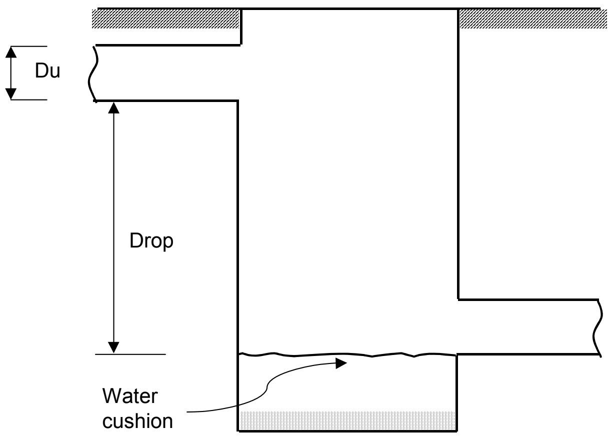


Figure 6.10: Drop pit

Length of Pits

The length of a drop pit should be increased to prevent unnecessary thrust on the pit walls. The minimum length of the pit for a given upstream pipe diameter and drop height is set out in Table 6.8.

Table 6.8: Minimum length of drainage pits (length in the direction of flow) (after VicRoads, 1995)

Drop	D_u< 600 m	D_u > 1200 m
Less than 0.5 D_u	Std Size	1.5 D_u
0.5 D_u to 1.5 D_u	1.5 D_u	2.0 D_u
1.5 D_u to 2.5 D_u	2.0 D_u	2.0 D_u
More than 2.5 D_u	2.0 D_u	2.0 to 3.0 D_u

D_u = diameter of upstream (inlet) pipe

Energy Losses Through Pits

Energy losses occur at pits due to contraction and expansion of flow, mixing of flows and change of flow direction. Losses at pits are usually expressed as the head difference on the hydraulic gradeline in terms of the average outlet velocity. The hydraulic gradeline (or hydraulic gradient) is the loss of hydraulic head per unit distance of flow. Head losses at pits can be calculated as follows:

$$H_p = \frac{\left(K_p V_o^2\right)}{2g} \qquad \text{Equation (6.7)}$$

where

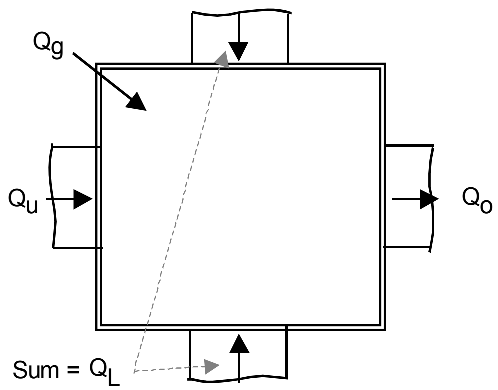
H_p head loss through pit (m)

K_p coefficient dependent on the inlet and outlet diameters, the inlet and outlet discharges, and the pit geometry

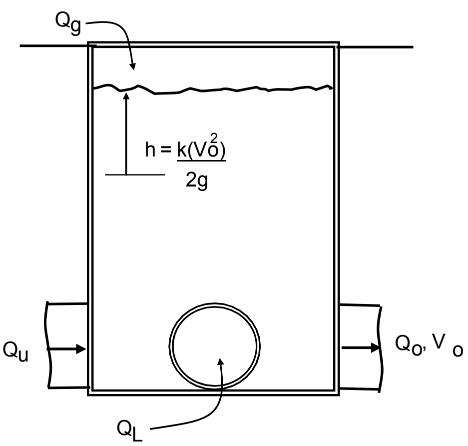
V_o average outlet velocity assuming the pipe is flowing full (= Q/A) (m/s)

g gravitational acceleration, assumed to be 9.81m/s^2

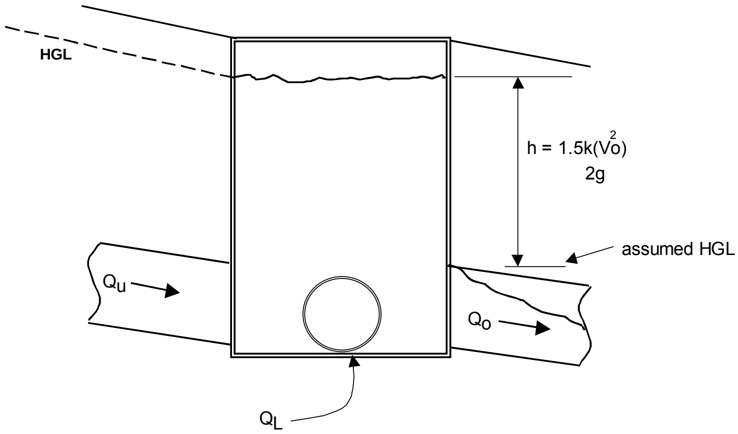
Table 6.9 shows typical values of K_p for a number of pipe arrangements as shown in Figure 6.11. For cases where small losses are critical, a more sophisticated treatment may be found in Sangster (1958).



Inlet Pit - Plan view



Inlet Pit - Side view



Junction Pit - Side view (part full flow)

Figure 6.11: Flow definitions for pits as used in Table 6.10 (adapted from ARRB Special Report 34, 1986)

Table 6.9: Head loss factors for pits (after VicRoads, 1995)

Description	Q_u	Q_L	Q_g	k
Inlet pit with one outlet pipe				
Side entry	-	-	Q_o	10
Grated pit	-	-	Q_o	5
Inlet pit on through pipe	0.9 Q_o	-	Some	0.5
	0.7 Q_o	-	0.3 Q_o	1.3
	0.5 Q_o	-	0.5 Q_o	2.1
Junction pit on through pipe	Q_o	-	-	0.2
Inlet pit on through pipe with laterals	0.9 Q_o	Some	Some	0.5
	0.7 Q_o	Some	Some	1.1
	0.5 Q_o	Some	Some	1.5
	0.2 Q_o	0.7 Q_o	Some	2.0
Junction pit on through pipe with laterals	0.9 Q_o	Some	-	0.5
	0.5 Q_o	0.5 Q_o	-	1.5
	0.2 Q_o	0.8 Q_o	-	2.0
Inlet pit on L bend	-	Q_o	Some	1.5
Junction pit on L bend	-	Q_o	-	1.3
Inlet pit on T junction with laterals	-	Q_o	Some	1.8
Junction pit on T junction with laterals	-	Q_o	-	1.6
Drop pit				
Direction change less than 45°	Q_o	-	Some	2.0
Direction change more than 45°	Q_o	-	Some	2.5

1. Q_u = flow from upstream pipe; Q_o = flow out of pit; Q_L = flow from lateral pipes; Q_g = flow from above the pit water level. k = pit head loss coefficient.
2. The pipes are assumed to operate below the water level in the pit. Flows entering from above the water surface should be added to Q_g.
3. Where part full flow occurs in the outlet pipe, tests have shown that the water surface is significantly higher. Assume the hydraulic gradeline to be at the pipe obvert, and add $H = 1.5kV_o^2/2g$.
4. Where the design flows are between tabulated values, interpolate between the k values.
5. Where flow is deflected through a horizontal angle at a pit, add the coefficient from Figure 6.12 to the k value from Table 6.10, except at drop pits. For drop pits use the values from Table 6.10.
6. If Du/Do is less than 0.9 or a better estimate of k is required, refer to the Sangster's design charts (Sangster, 1958). Du and Do are the diameters of the inflow and outflow pipes respectively.

As set out in Note 3 above, sometimes the water level in the pit is slightly higher than the adjacent hydraulic grade line owing to the reduction in the velocity head (converted to pressure head) as the water enters the pit. However, for most situations, the water level in a pit is assumed to coincide with the hydraulic gradeline level.

Refer to Vol. 1 ARR (2001) Technical Note 9, Example of Hydraulic Design Calculations.

Change in Horizontal alignment

Where a bend in the pipeline is required, consideration should be given to using splayed pipes or a pit should be constructed. The choice will depend on the curvature and radius required.

Pits can be constructed to include a deflector, straight or curved, to reduce head losses due to the change in water direction (see Figure 6.12). Losses due to angles should be added to other losses at the pit except at drop pits.

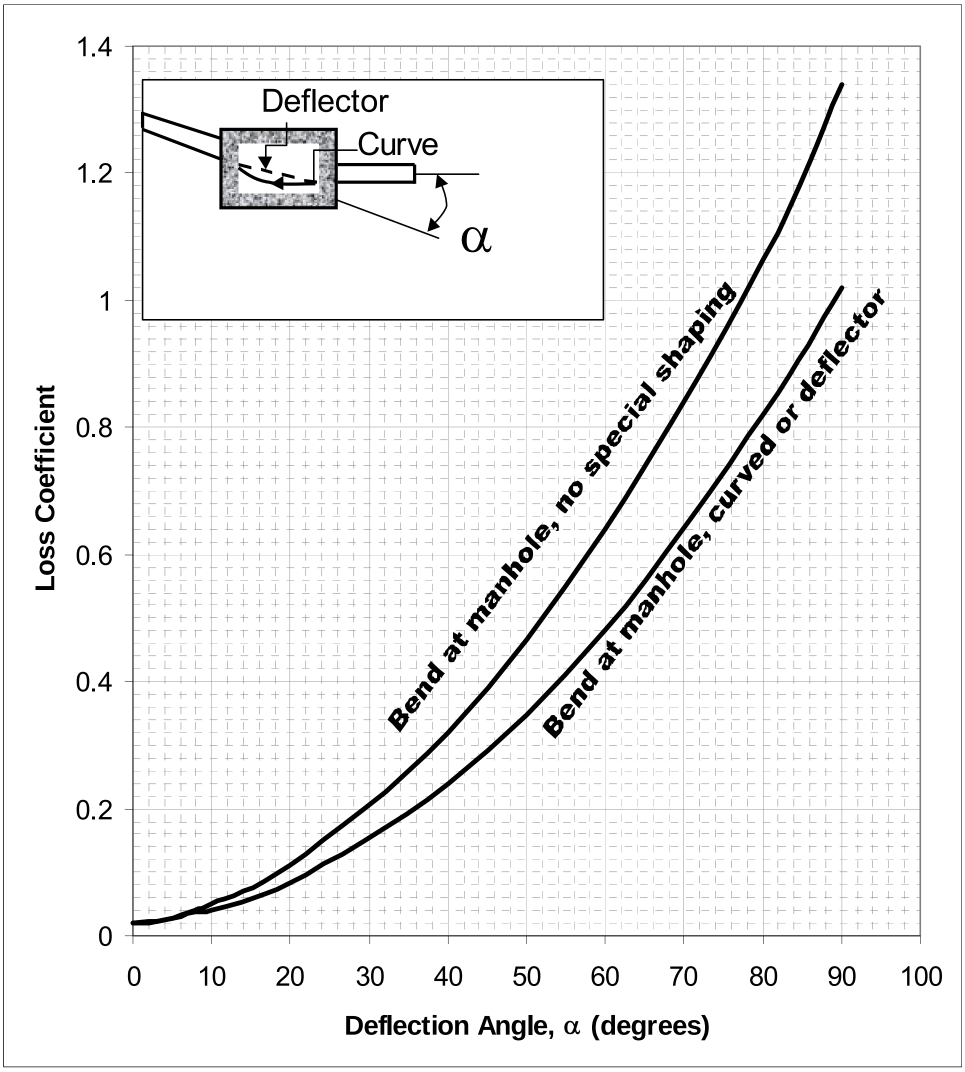


Figure 6.12: Head loss at angles in pits (after VicRoads, 1995)

Pit Capture

It is generally not economical to design for 100% capture, and values between 95 and 80% may be used, provided that bypass flows are captured at the end of the system. The capture of side entry pits has been determined experimentally and design charts are available (see Figure 6.13 for a typical example). (Note: FHWA Hydraulic Engineering Circular 22 [2001] provides some empirical relationships for pit capture efficiencies.) The ‘C’ values on the right hand side of Figure 6.13 are the capture efficiencies and Table 6.10 provides some typical values. The ‘W’ refers to the width of the surface spread. Figure 6.13 is for a specific kerb and channel configuration and other design charts would be applicable to other configurations.

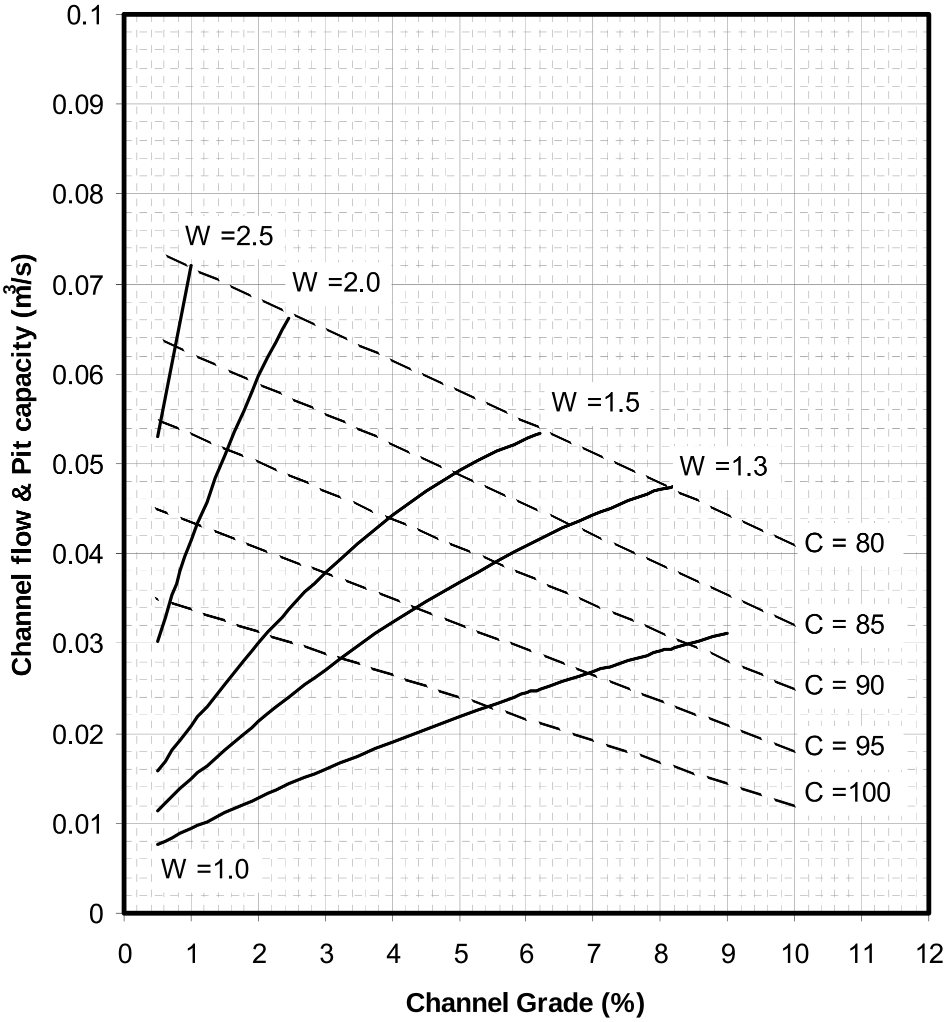


Figure 6.13: Sample design chart (adapted from VicRoads, 1995)
(based upon 1 m side entry pit, 3% shoulder crossfall)

Pit spacing should be designed to limit the spread of flows encroaching onto the trafficked lanes. The following discussion relates to the theoretical capacity of inlets. However, in practice some blockages often occur and Table 6.10 lists typical blockage conditions.

Table 6.10: Pit capture efficiency (after Austroads, 2002)

Condition	Inlet Type	Capture Efficiency (% of Theoretical)
Sag	Side entry	80
Sag	Grated	50
Sag	Combination	See note 1
Continuous Grade	Side entry	80
Continuous Grade	Longitudinal bar	60
Continuous Grade	Grated[2]	50
Continuous Grade	Combination	90[3]

1. In a sag curve, the capacity of a combination inlet should be taken to be 100% of the theoretical capacity of the kerb opening, the grate being assumed to be fully blocked.
2. Transverse bar grate or longitudinal bar grate incorporating transverse bars.
3. On a continuous grade, the capacity of a combination inlet should be taken as 90% of the combined theoretical capacity of the grate plus the kerb opening.

Pit inlet efficiency is improved by increasing the depth at the kerb face and by increasing the cross slope. Channel cross slopes are greater than the crossfall on the abutting road, usually about 1 in 12.

Gutter deflectors Figure 6.6) may increase pit inlet capture by up to 15% in some circumstances such as:

- where wide flows occur on roads with crossfall less than 2%
- on gradients less than 3%, and normal road crossfall, the increase in capture may be about 5 per cent
- where the gradient is above 5%, the flow may jump the deflectors.

The capacity of grated pits is unique to each pit design and a typical design chart is shown in Figure 6.13. Design sheets appropriate to the grated pit configuration and situation should be consulted and capture efficiency factors from Table 6.10 applied (provided these have not already been incorporated into the design charts).

6.4.6 Pipes

Capacity

The typical pipe capacity chart (Figure 6.14) is based on the Colebrook-White equation that is considered more accurate than the Manning equation. The chart is valid for long straight pipes with no interruptions to flow, flowing full but not under pressure. For pipes under pressure, refer to the Darcy–Weisbach formula in Equation (6.8) on hydraulic gradeline calculations. Some Australian States design for pipes flowing full but not under pressure based on the design ARI, and check the hydraulic gradeline for a specified rare event. Other States advocate that all design should be for pressure flows, in which case Figure 6.14 shall not be used.

The diameter of a cross connection may be selected from either Table 6.11 or may be set equal to the diameter of the pipe conveying flow from the connected mainline or lateral pipeline junction pit, whichever is smaller. In cases where the diameter of the cross connection pipeline exceeds the diameter of the connected mainline or lateral pipeline by more than one increment, the connected mainline or lateral pipeline size may be used in the cross connection provided pit head loss and water level conditions at either end are investigated for satisfactory performance.

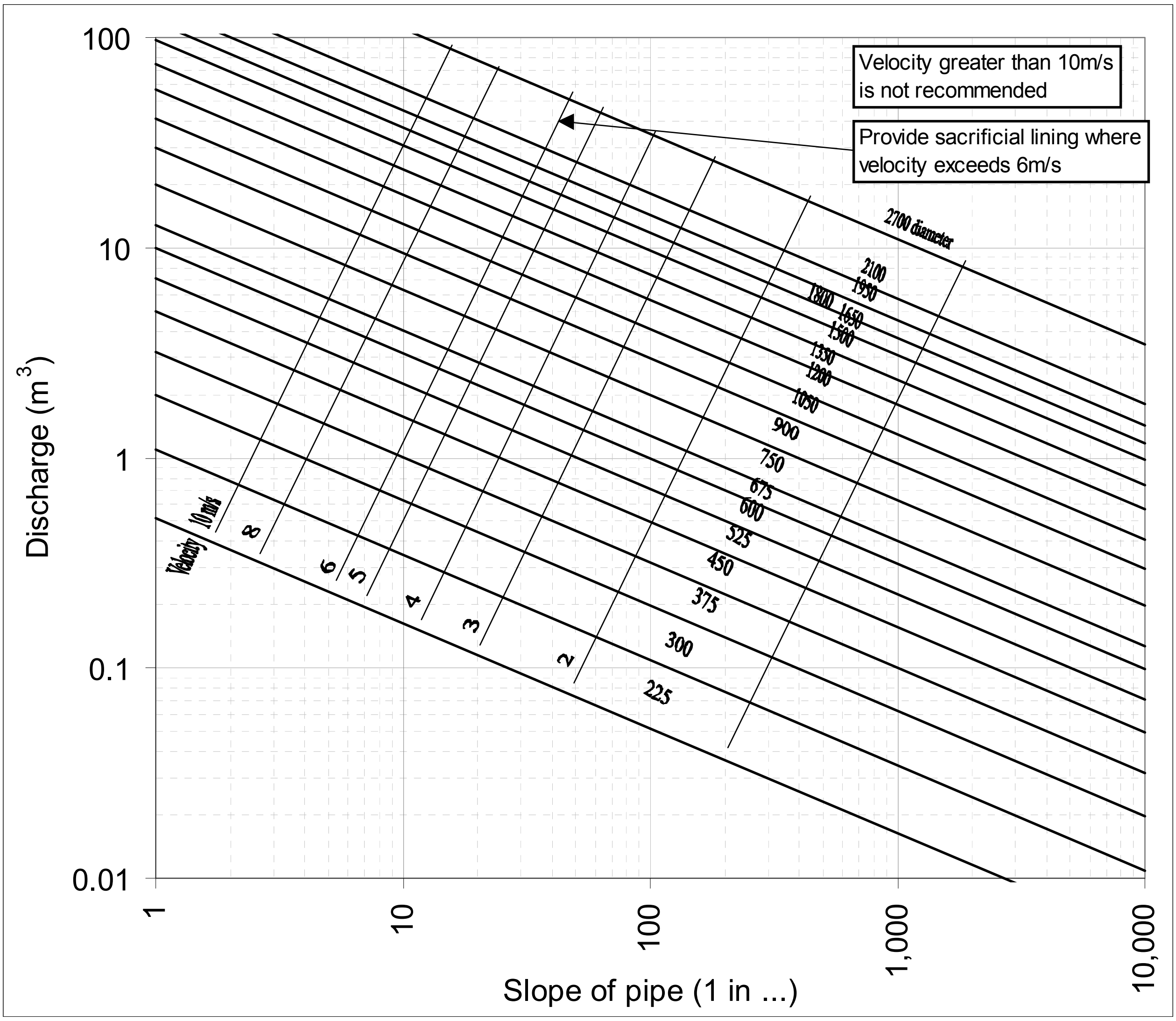


Figure 6.14: Discharge for pipes flowing full (based on Colebrook-White formula roughness of 0.6mm)

Table 6.11: Recommended pipe diameters for cross connections

Flow in cross-connection L/s	Nominal pipe diameter, D_t mm	Flow in cross-connection L/s	Nominal pipe diameter, D_t mm
< 55	300	180 to 220	675
55 to 80	375	220 to 270	750
80 to 110	450	270 to 320	825
110 to 140	525	320 to 370	900
140 to 180	600	370 to 500	1050

6.4.7 Hydraulic Gradeline

Procedure

Calculation of a hydraulic gradeline starts from a known water level downstream, and proceeds upstream, making allowances for pipe friction and losses through pits.

The design procedure assumes that flow within each storm drain segment is steady and uniform and that the average velocity throughout a segment is constant. In actual drainage systems, the flow at each inlet is variable, and flow conditions are not truly steady or uniform. However, since the usual hydrologic methods employed in storm drain design are based on computed peak discharges at the beginning of each run, it is conservative design practice to use a steady uniform flow assumption.

Tailwater Depth

The elevation of the water surface at the system outlet is called the tailwater (TW). The tailwater will be one of the cases in Table 6.. The critical depths are determined from the charts in Figure 6.15.

Table 6.12: Assumed level at outlet

Tailwater level	Assumed level in pipe outlet
Tailwater above the obvert of the outlet	Adopt TW elevation
Tailwater above critical depth	Assume either TW or (d_c + D) /2, whichever is higher.
Tailwater below critical depth	Assume TW = pipe invert level plus d_c at the pipe outlet.
Tailwater below invert	Check for supercritical flow depth. TW = pipe invert level plus dp (the supercritical flow depth) or d_c (the critical depth)

If the outlet is on a river or stream, it may be necessary to consider the probability of two hydraulic events occurring at the same time to determine the likely elevation of the tailwater in the receiving stream. If the underground drainage system has a catchment area much smaller than that of the receiving stream, its peak discharges may not coincide with that of the receiving stream. Table 7-3 of the FHWA Hydraulic Engineering Circular Number 22 provides a comparison of discharge frequencies for coincidental occurrence for a 10 year and 100 year design storm. This table can be used to establish an appropriate design tailwater elevation for a storm drainage system, based on the expected coincident storm frequency on the outfall channel.

Energy Losses

For each segment of the hydraulic grade line, all energy losses in the pipe runs and junctions must be estimated. In addition to the friction in each pipe run, energy (or head) is required to overcome changes in momentum or turbulence at outlets, inlets, bends, transitions, junctions, and access manholes.

Pipe Friction

Head loss through pipe friction is determined using the Darcy-Weisbach formula (Equation 6.8). The friction of the pipe surfaces is shown in Table 6.11 for commonly used pipes.

$$h_f = \frac{L \times V^2}{D \times 2g} \times f \qquad \text{Equation (6.8)}$$

where

h_f friction head loss (m)

L length of the pipe (m)

V average velocity in the pipe (=Q/A) (m^3/s)

D diameter of the pipe or an equivalent for a rectangular section (m)

g gravitational acceleration, assumed to be 9.81 m/s^2

f friction factor (see Equation (6.10)).

Before estimating the friction factor f, it is necessary to calculate the Reynolds Number of the flow.

$$Re = DV/\eta \qquad \text{Equation (6.9)}$$

Where D & V are as above, and

Re the Reynolds Number

η kinematic viscosity of water, approximately 1.0 x 10 $^{-6}$ m^2/s at 20°C.

The friction factor, f, can be obtained from standard charts such as the 'Moody Diagram' (see Figure 6.15) or approximated by the equation:

$$f = \frac{1.325}{\left[\ln\left(\frac{e}{3.7D} + \frac{5.74}{\mathrm{Re}^{0.9}}\right)\right]^2} \qquad \text{Equation (6.10)}$$

where

e pipe wall roughness (m) = 1000 k (see Table 6.11)

Re Reynolds Number

D diameter of the pipe or an equivalent for a rectangular section (m).

Table 6.11: Typical pipe roughness (k) in millimetres (after ARR Vol. 1, 2002)

Pipe material	Pipe conditions		
	Good	Normal	Poor
Concrete			
Precast with, 'O' Ring Joints	0.06	0.15	0.60
Spun precast, 'O' Ring Joints	0.06	0.15	0.30
Monolithic construction against steel forms	0.30	0.60	1.50
Monolithic construction against rough form	0.60	1.50	
Fibre Cement	0.015	0.03	
UPVC			
With chemically cemented joints		0.03	
With spigot and socket joints		0.06	

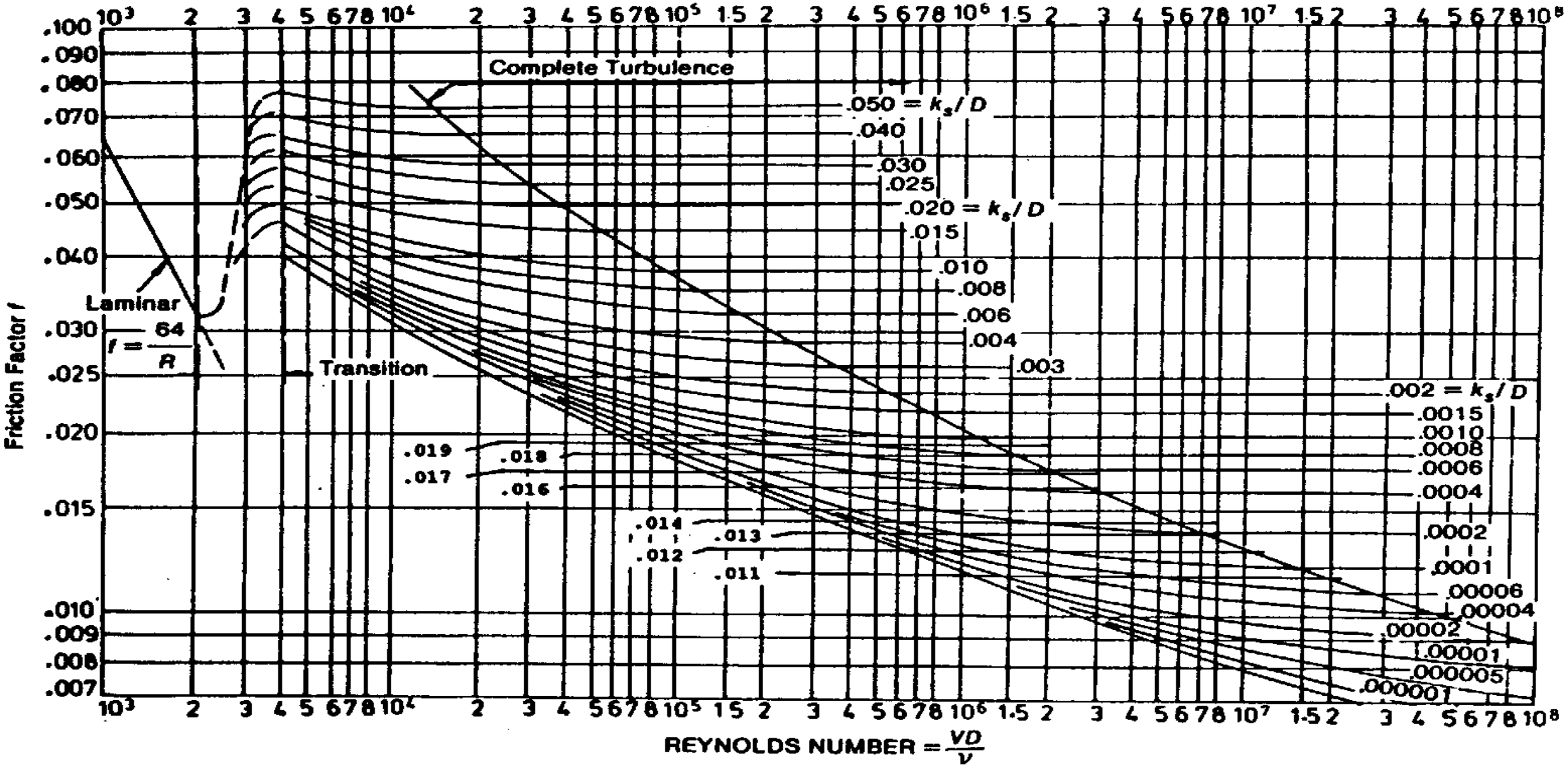


Figure 6.15: Moody diagram (after ARR, 2001)

If the hydraulic gradeline plots below the invert of the pipe at the upstream pit, the flow is not under pressure and Figure 6.16 should be used to estimate the part full depth in the pipe. That depth should be added to the invert level of the pipe to represent the HGL along the length of the pipe.

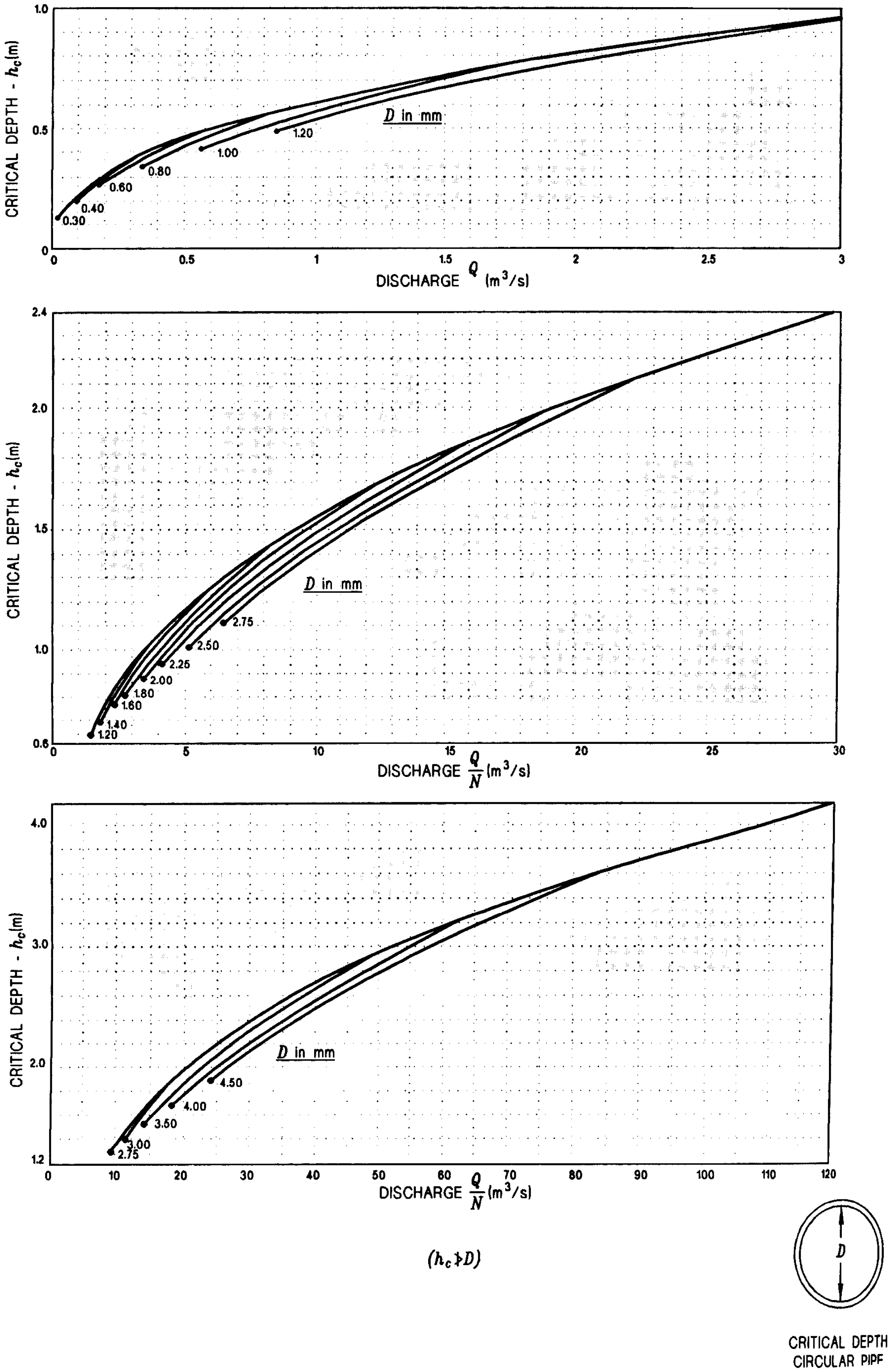


Figure 6.16: Critical depths for circular pipes (after Austroads, 1994)

CHAPTER 7

7 CULVERTS & ENERGY DISSIPATORS

7.1 Culverts

7.1.1 Introduction

Cross drainage of roads is by means of pipes or box culverts, collectively known as culverts. There are two prime concerns in the design and location of culverts, namely:

- potential for upstream flooding
- potential scour downstream due to increased stream velocities.

If the culvert restricts flows, it will cause a rise in upstream water levels (afflux) and this should be considered in relation to adjacent properties. The available headwater upstream of the culvert should be assessed to determine the limit on flood height. The potential damage is greater in urban situations due to the likelihood of affecting a greater number of people.

In rural situations, it may be possible to constrict flows purposely to attenuate peak discharges, though the available headwater height will need to be checked against potential damage to neighbouring properties. The headwater depth should not be a potential hazard to persons or stock.

The headwater should not damage the roadway, the pavement or the culvert. The potential for scouring due to increased stream velocities should be examined (see Section 7.2). Conversely, flow velocity within the culvert should be maintained above a minimum to prevent siltation.
Debris can significantly reduce the capacity of a culvert, or may cause forces that threaten the integrity of the structure. Either debris racks should be provided upstream, or the culvert should have sufficient freeboard to pass floating objects.

7.1.2 Siting

To limit costs, it is preferable to site a culvert at right angles to the centre-line of the road. It is also preferable to site a culvert within a natural drainage channel. Culverts should be laid on a straight alignment and grade to facilitate construction. The channel immediately upstream and downstream of a culvert should not include sharp bends as these are subject to erosive forces. Realignment of the natural watercourse within the vicinity of the road should be a priority.

The longitudinal profile of the culvert should follow the natural water course profile where possible. Alterations to the stream bed slope may be required for engineering reasons (e.g. prevent siltation or scouring), however, the effects on the water course should be examined to ensure that they are not degraded.

7.1.3 Culvert End Treatment

The culvert end treatments need to:

- guide the channel flow to the pipe and to transition the cross-sectional area of flow
- prevent erosion of the stream and/or the embankment
- not pose a hazard to road users
- prevent fill from falling into the culvert

- anchor the culvert against uplift forces (applicable to light weight culverts, such as corrugated pipe culverts)
- provide strength to flexible culverts
- where possible culvert headwalls should be flush with the adjacent batter to reduce the likelihood of causing errant vehicles to become airborne.

Culvert endwalls consist of a shaped retaining wall, a floor or apron, and a cut-off wall, as shown in Figure 7.1. Where the culvert lies at an angle to the road, the headwalls may be placed parallel with the road edge, resulting in a skewed headwall, as shown in Figure 7.2.

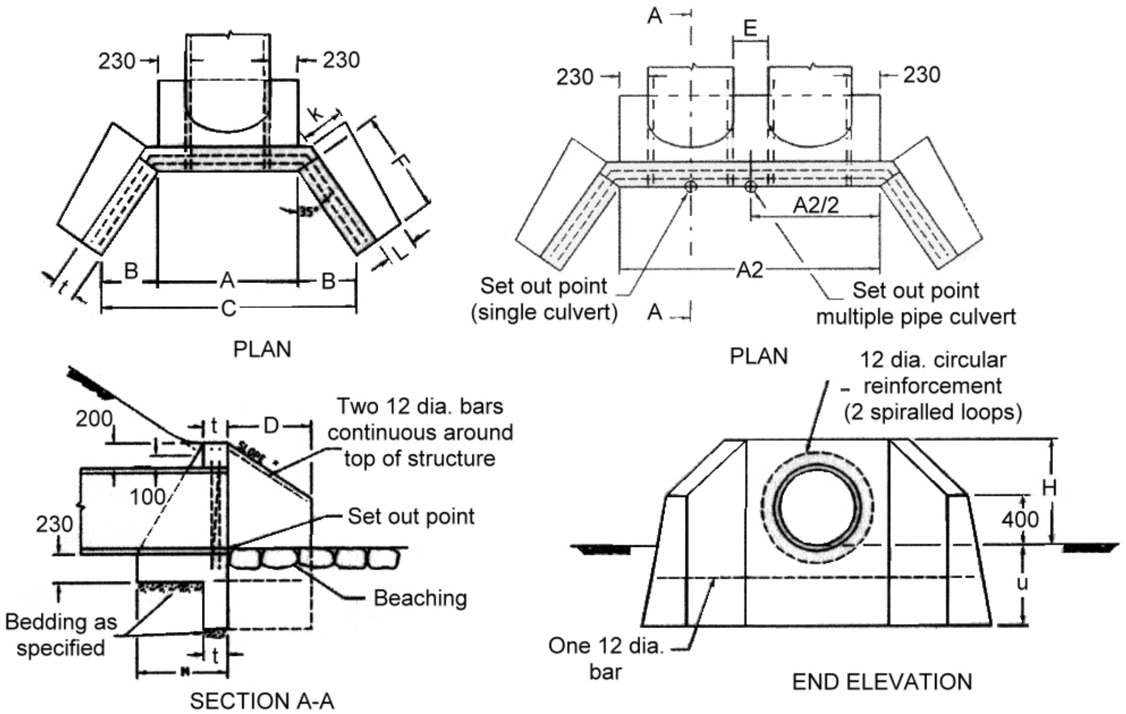


Figure 7.1: Mass concrete headwall (Note: Dimensions not shown in diagram and will vary depending upon pipe diameter) (after VicRoads standard drawings)

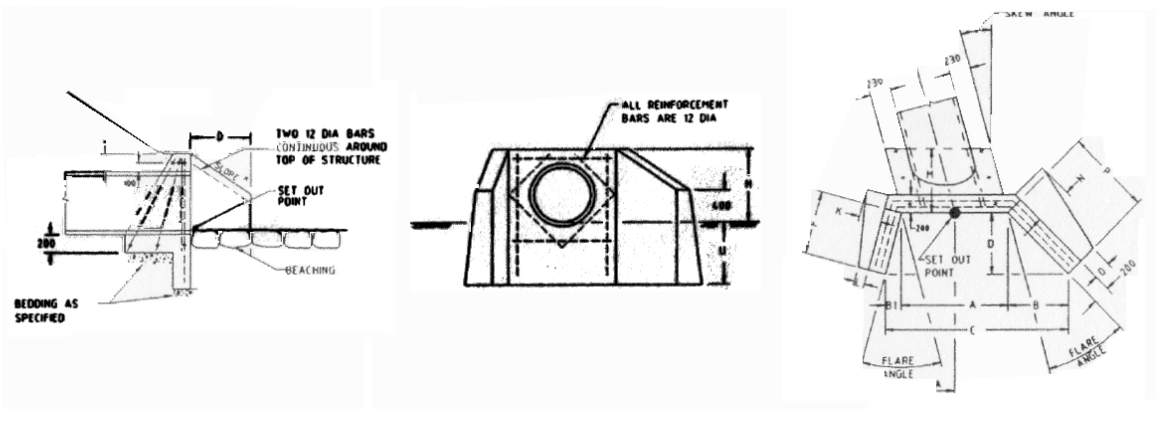


Figure 7.2: Mass concrete skewed headwall
(Note: Dimensions not shown in diagram and will vary depending upon pipe diameter) (after VicRoads standard drawings)

The headwall of culverts may pose a roadside hazard. Where economical, small culverts should be extended to the limits of the appropriate clear zone. Where headwalls are at right angles to the direction of traffic, for example on culverts under driveways, driveable headwalls should be used (see Figure 7.3) designed to AS 3600. Headwalls on pipes between 450 mm and 1050 mm diameter should be made driveable or traversable. Above this limit, headwalls should be protected by a safety barrier or marker posts.

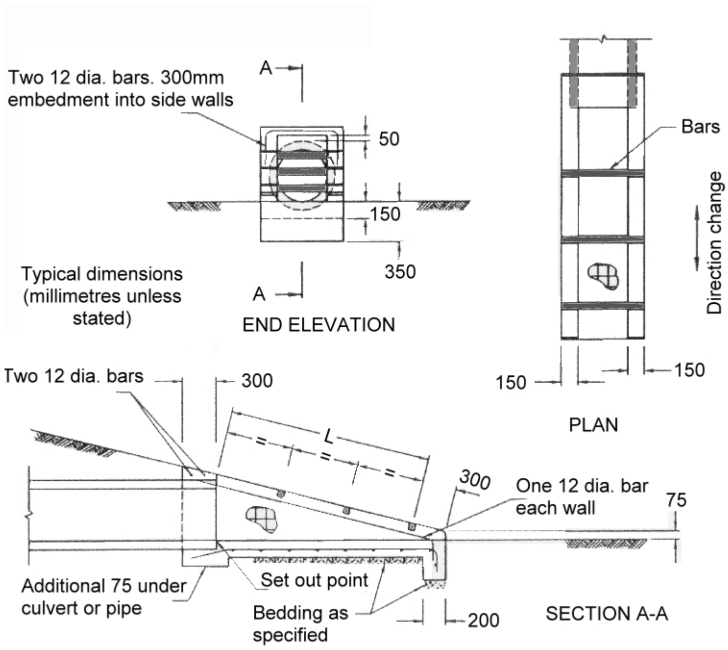


Figure 7.3: Typical driveable culvert endwall (after VicRoads Standard drawings)

On low trafficked roads, or where main roads traverse broad flood plains, culverts may be used in combination with floodways. The optimum waterway areas for the culvert and the floodway should be decided after an appropriate investigation.

Corrugated steel culverts should be provided with headwalls and adequate anchorage. Examples have been reported in the Western Australia of damage to culvert ends by vortices, or bending of culvert ends or uplift of the entire culvert and transport downstream by buoyancy forces.

7.1.4 Piping

Piping is the forming of a hollow in the material around a culvert by water seepage. It may eventually cause failure of the culvert bedding or the road batters. Piping may be expected where ponding above the culvert occurs for a long time, such as where the road embankment forms a dam or a detention basin. Piping may also occur through open culvert joints due to differential settlement. Where conditions for piping exist, the pipes should be rubber ring jointed. Where the embankment forms a dam, the culvert should have seepage collars cast around it at regular intervals (these can be cast in place flanges, steel flanges, etc. that increase the seepage path length on the outside of the pipe). The collars should increase the minimum seepage path length along the pipe by about 30%.

7.1.5 Abrasion

Abrasion may occur where culvert inlet velocities, greater than 1.5 m/s, are combined with frequent passage of hard rock fragments. Remedial measures include reduction of designed pipe velocities or sacrificial lining.

7.1.6 Corrosion

Corrosion causes pipe wear due to electrochemical currents, and is more significant in corrugated metal pipes. The main factors are, the electrical properties of the pipe materials and protective coating, the pH values of the surrounding soil and ground water, and the soil's electrical resistivity. These properties should be measured wherever metal pipes are being considered. The corrosive properties of the soil may rule out the choice of metal pipe. In mildly corrosive conditions, the thickness of metal plate could be increased, or aluminium alloy pipes could be considered. Additional information is contained As 4058.

Corrosion of reinforcement bars in concrete pipes and structures should also be considered, especially in or in the vicinity of marine environments.

7.1.7 Minimum Parameters

For minimum size, minimum grade, minimum cover and bedding conditions for culverts, see Section 6.3.6.

7.1.8 Culvert Design

In some States, standard box culverts have been designed for a maximum cover of two metres. A comprehensive exposition on culvert design is set out in Austroads (1994) including a flow chart (Figure 7.11 in Austroads, 1994), to which the reader is referred. The Austroads process is summarised as follows:

- determine site details (levels for road, contours, flood limits for adjacent properties upstream and downstream, historical flood levels, etc.) (see Chapter 2 – Preliminary Investigation)
- determine design flows (see Chapter 3 – Obtaining Design Flows)
- select trial culvert size
- determine headwater depth for inlet control
- determine tailwater depth for outlet control
- determine headwater depth at inlet under outlet control conditions
- determine controlling headwater depth
- calculate outlet velocity.

A high tailwater may cause the culvert to flow under outlet control, thus increasing the headwater depth required to pass the design discharge.

A low tailwater may allow the culvert flow to accelerate at the outlet and to cause scour. Dissipation of flow energy to match downstream conditions must be carefully managed.

Selection of a trial culvert size

Use the flow equation (Q=AV) and balance with Manning's equation to estimate the design discharge for the trial culvert size. The procedure is:

- adopt a maximum velocity from Table 8.6 as a guide and also consider higher flow velocities where there is good grass cover downstream
- calculate the area required by rearranging the flow equation to A = Q/V and choosing a pipe or box culvert that approximates the required area.

Inlet or Outlet Control

Inlet control refers to the situation where the flow in the culvert is governed by the capacity of the entrance acting as a weir or an orifice. The roughness, length and slope of the culvert are not factors in determining culvert capacity.

Outlet control refers to the situation where the cross-section determining the energy required to drive the design discharge through the pipe is at or near the outlet. The culvert may be submerged, or flow full, or flow part full. While typically outlet control will occur for long culverts on flat grades with high tailwater depth, outlet control may also occur where the streambed is very flat and both the inlet and outlet are not submerged.

Determine headwater depth for inlet control

Use the nomograph (see Austroads, 1994) appropriate to the culvert type and headwall arrangements to determine the head water depth under inlet control conditions.

Determine tailwater depth for inlet control

Use the nomograph (see Austroads, 1994) appropriate to the culvert type to determine the critical water depth. Take the average of the critical depth and pipe diameter (or box section depth) and compare against known records of tailwater depth. Take the larger of the two depths.

Determine headwater depth for outlet control

Use the nomograph (see Austroads, 1994) appropriate to the culvert type and headwall arrangements to determine the entrance loss coefficient and the energy losses through the culvert. Calculate the head water depth under outlet control conditions by summing the energy losses and the outlet head, and subtracting head lost due to slope.

Determine controlling headwater depth

If the headwater determined under inlet control is greater than the headwater determined under outlet control, the headwater is taken as that determined from the inlet control calculations. If this is not the case, then the headwater is taken as that determined from the outlet control calculations.

Calculation of the outlet velocity

The average velocity is calculated from the design discharge, Q, divided by the area of flow near the pipe outlet.

If the culvert is operating under outlet control, the depth is taken as:

- Tailwater depth above pipe obvert (Use the full cross-sectional area of the culvert).
- Tailwater depth between critical depth and pipe obvert (Calculate the partial area of the culvert corresponding to the tailwater depth. For pipes, use Figure 7.4 as a design aid).
- Tailwater depth less than critical depth (Use the area corresponding to the critical depth. For pipes, use Figure 7.4 as a design aid).

For a culvert under inlet control the velocity can be based on the normal flow depth using Manning's Equation ().

The calculated flow velocity is compared against the values permitted in Table 8.6.

However, if the difference between the tailwater depth and the critical depth is 0.3 metres or more, either approximate the drawdown by multiplying the pipe velocity by 1.3, or calculate a backwater curve.

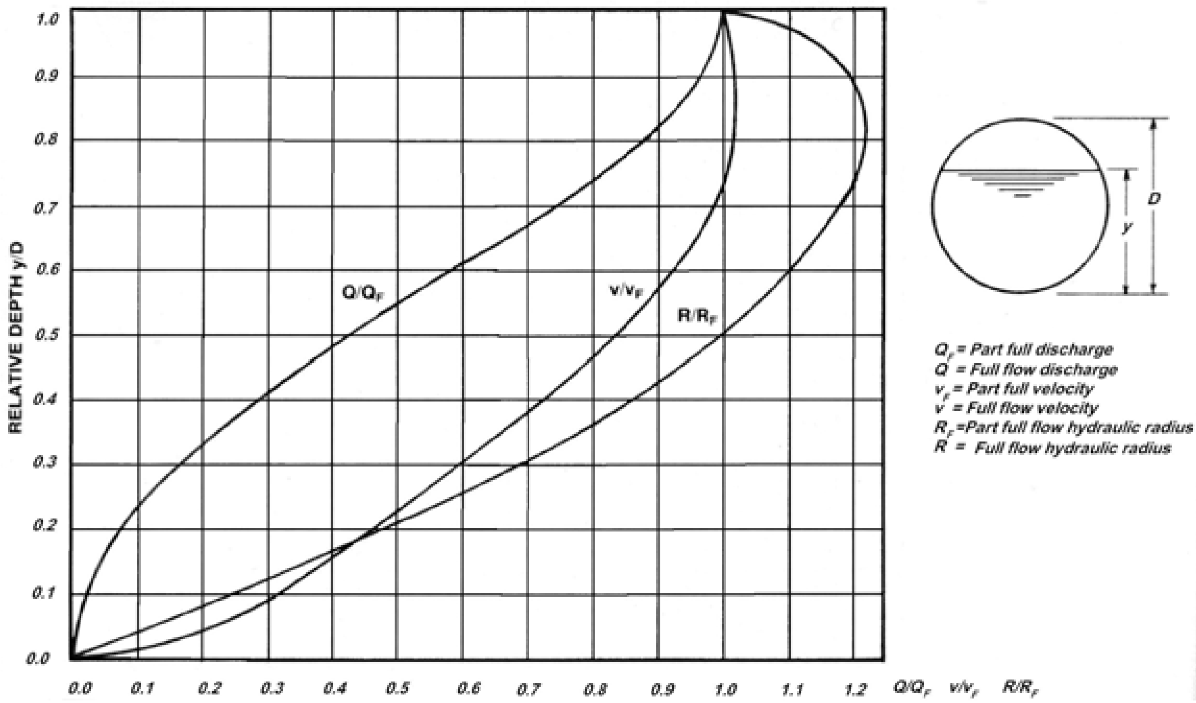


Figure 7.4: Velocity and discharge in part full pipes (after Hydraulics Research Station, 1983)

7.2 Energy Dissipators

7.2.1 Introduction

Where the velocity of outflow from a culvert is greater than the erosive resistance of the soil and vegetation downstream, an energy dissipator is required to allow a return to pre-existing drainage conditions as quickly as possible.

The energy of flow is reduced by a combination of:

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

The simplest measure is rock beaching (see Section 8.5.1 for minimum length of rock beaching). Where the calculated length of beaching exceeds 12 metres, an energy dissipator may be warranted (VicRoads 1995a).

Four types of energy dissipators are shown in Figure 7.5.

Care should be exercised in selecting a dissipator type to suit a particular site. Also, each dissipator requires certain conditions for effective operation. For instance, if the tailwater is not of sufficient depth, the flow may sweep sediment out of the dissipator instead of forming a hydraulic jump to reduce energy. The drainage designer should check a range of flow conditions to ensure that the device will operate as intended.

As energy dissipators are expensive, all reasonable alternatives should be considered. For example, drop pits can control small flows. Pipe velocities can sometimes be reduced by laying pipes deeper and flatter using a drop inlet and this is less costly than an energy dissipator.

The impact type dissipator is most effective where the Froude number of flow is more than 4 and where the tailwater is not as deep as necessary for a drop-stilling basin. The maximum permissible discharge volume is 11 m^3 (VicRoads, 1989)

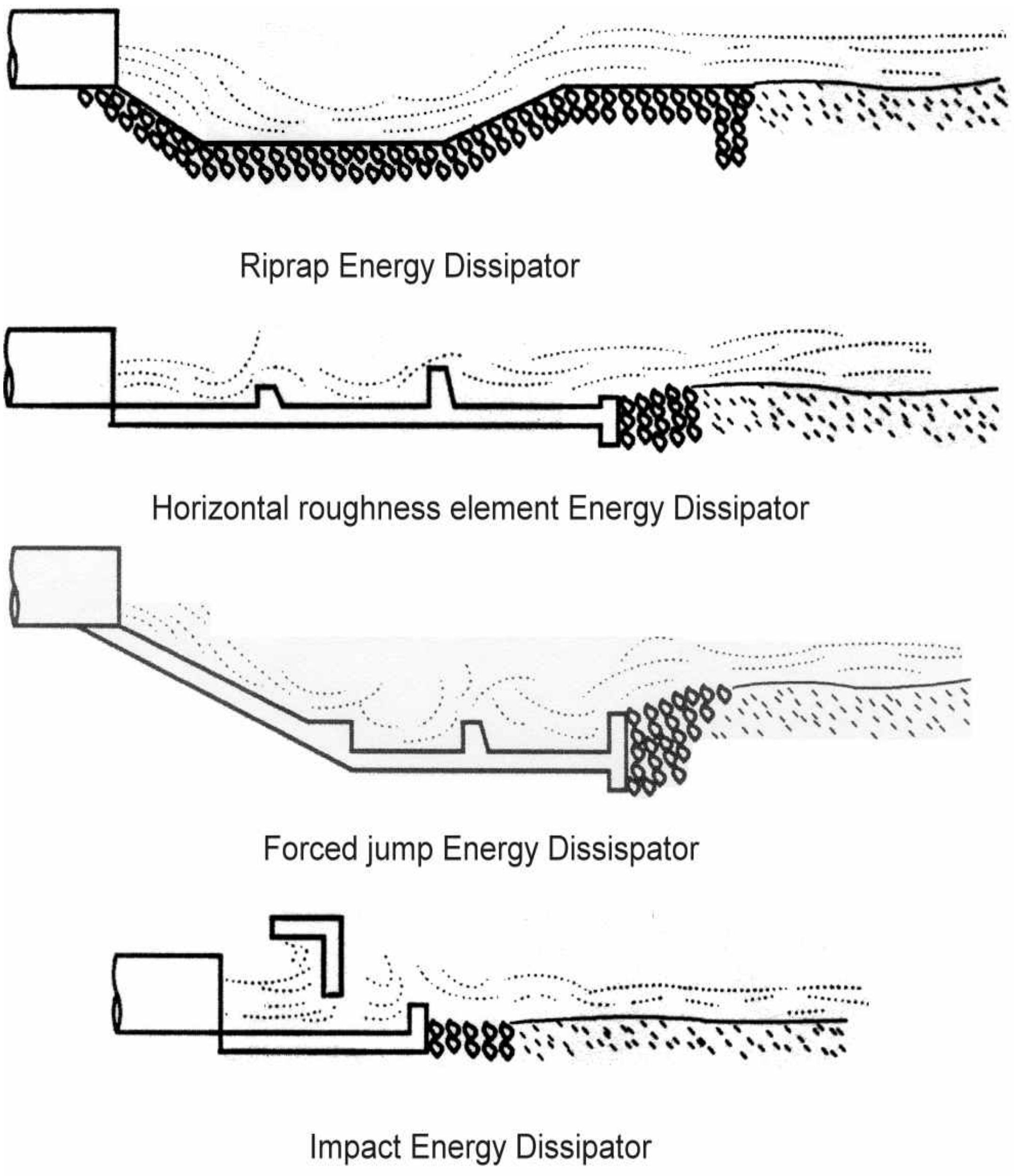


Figure 7.5: Energy dissipators (after RTA, 1993)

A riprap energy dissipator typically is most efficient with low tailwater depths and readers are advised to seek further information on design from RTA (1993). The horizontal roughness dissipater is sometimes known as a ‘Contra Costa” dissipator and is most suited to small to medium sized culverts. Again readers are advised to seek additional information from RTA (1993). The forced jump dissipator is only suitable for outlets with some tailwater depth. Additional pipes are required to ensure that the basin drains during low flow periods (see RTA, 1993).

A drop-stilling basin (see Figure 7.8) may be useful where flows are to be dropped through heights up to 4.5 metres, beyond which, the structure tends to become uneconomic. It is essential that this device have sufficient tailwater depth to support the hydraulic jump in the basin.

7.2.2 Information Required

Before starting calculations for a dissipator, the drainage designer requires:

- the design discharge for each average recurrence interval to be considered
- the flow depth and velocity in the upstream pipe for each case
- the tailwater depths in the receiving channel for each case
- a longitudinal section of the culvert and for a reasonable distance downstream.

7.2.3 Selection of Energy Dissipator Type

To aid selection of an appropriate dissipator type, it is necessary to calculate the Froude number of the culvert flow, and the downstream depth required to support a hydraulic jump.

The depth used to determine the Froude number is the 'hydraulic depth', that is, the waterway area divided by the top width, A/B. For pipes, hydraulic depth may be estimated with the use of Figure 7.6. For rectangular sections, hydraulic depth is the flow depth, d (m).

For a pipe, calculate the depth ratio d/D, where

d is depth of flow (m)

D is diameter of pipe (m)

Enter Figure 7.6 from the left with known ratio d/D to meet the θ line, proceed vertically to meet the A/BD line. Proceed horizontally to read the A/BD ratio on the left scale. Multiply this value by D to obtain hydraulic depth, A/B.

Calculate the Froude number of the flow using Equation ().

$$F_r = \frac{V_p}{\left(g \times A/B\right)^{0.5}} \qquad \text{Equation (7.1)}$$

where

F_r Froude number

V_p the velocity of the pipe flowing part full (m/s)

g gravitational acceleration (taken as 9.81 m/s)

A/B hydraulic depth (from Figure 7.6 and pipe diameter)

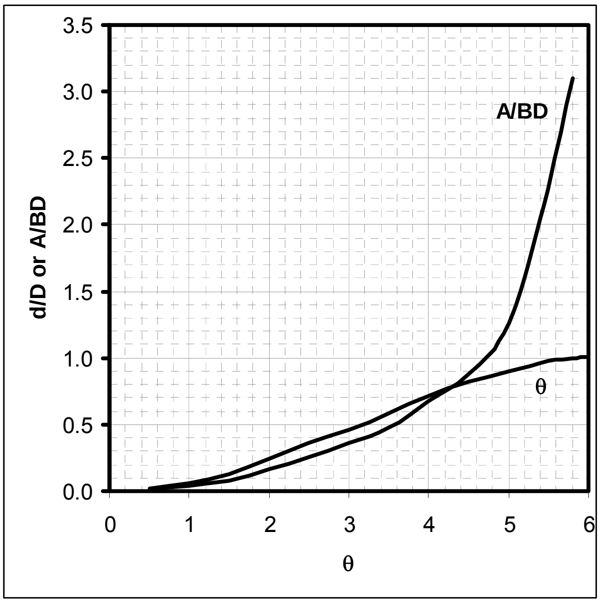
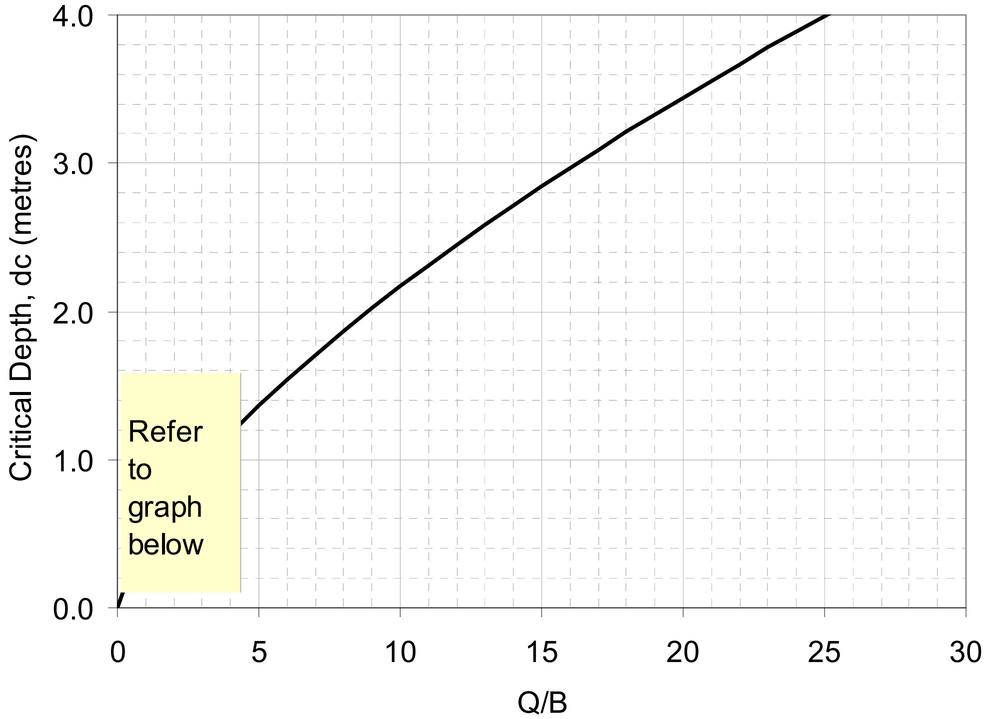


Figure 7.6: Estimation of hydraulic depth (after VicRoads, 1989)

A Froude number less than 1 indicates subcritical flow that probably does not need an elaborate structure. For a Froude number greater than 1, the next step is to estimate the depth after a hydraulic jump.

Determine the critical depth of flow, d_c (m) from Figure 7.4 for pipes or Figure 7.7 for box culverts. The depth after jump is approximately $2d_c$.



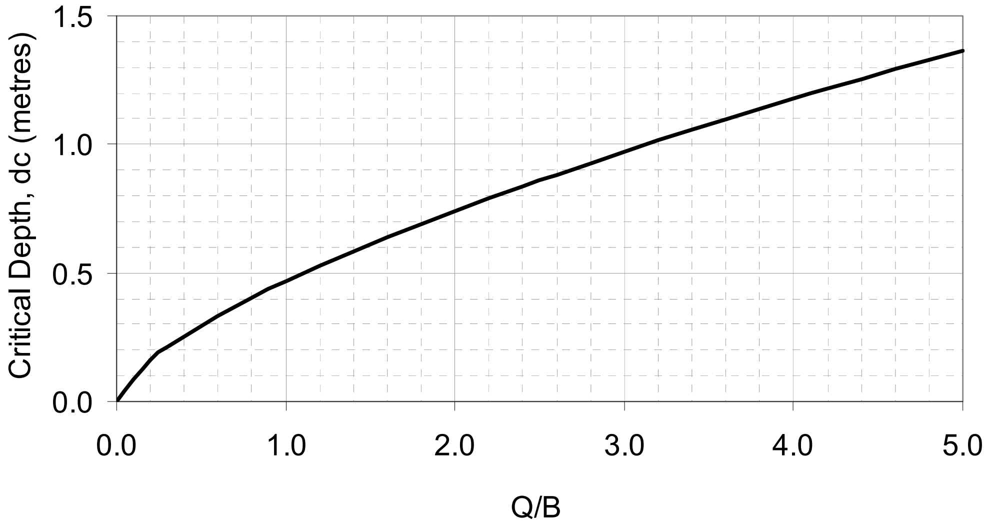


Figure 7.7: Critical depths for box culverts (after VicRoads, 1989)

Consider the following factors when selecting dissipator type:

- drop stilling basin

  where the Froude number less than 7 is desirable to limit the basin length

  A tailwater depth greater than $2d_c$ is essential. Part of this depth can be achieved by placing a low sill at the end of the basin (see Figure 7.8).

- Impact type dissipator, installation is recommended when:

  the Froude number is more than 4

  there is insufficient tailwater depth for a drop stilling basin; and / or

  there exists a need to minimise basin size for environmental reasons.

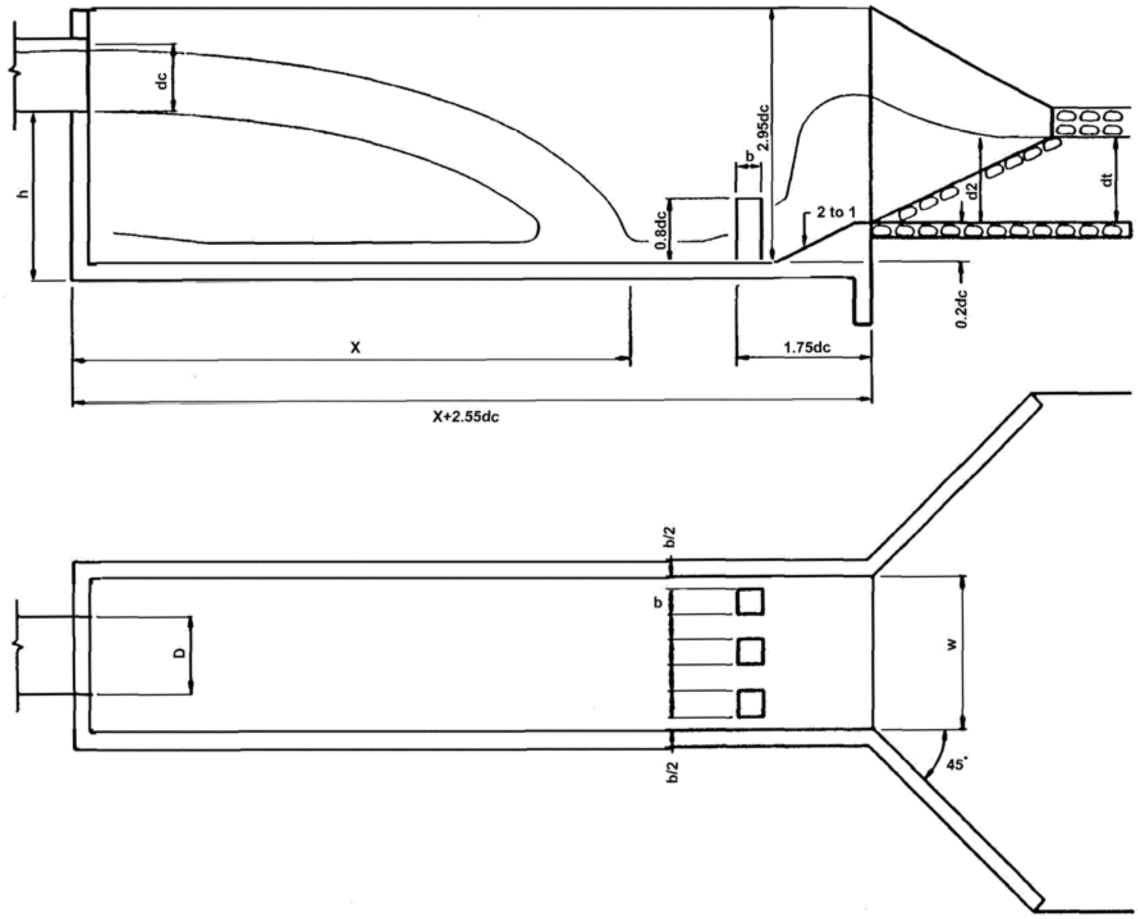


Figure 7.8: Drop stilling basin layout (after VicRoads, 1989)

7.2.4 Design of an Impact Type Dissipator

This Section gives design methods for an impact type dissipator. For the rip rap dissipator or the horizontal roughness dissipator, refer to RTA (1993). For more options, refer to US Federal Highway Administration 1975, or Rural Water Commission 1991, or Keller, 1989. The general design principles are shown in Figure 5.9.

In this device, energy dissipation is due to the flow striking a vertical baffle and being deflected upstream by the horizontal part of the baffle and the floor, creating strong eddy currents. The notches in the baffle concentrate the outflow into jets that clean sediment from the structure. The downstream sill evenly spreads and slows the outflow.

Where the incoming pipe is steeper than 1 in 4, a pit and a horizontal length of at least 8 metres should be placed immediately upstream of the dissipator. If this could cause a hydraulic jump in the pipe, a vertical air vent about one-sixth of the pipe diameter should be installed upstream of the jump.

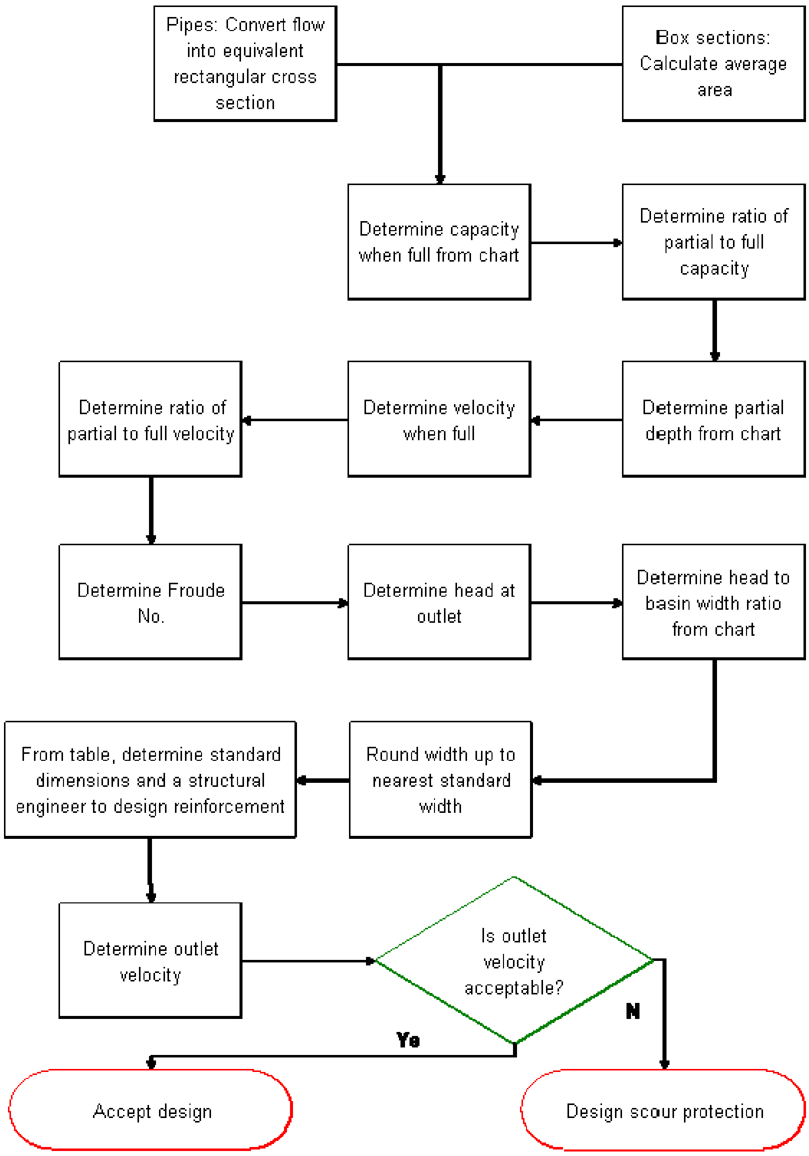


Figure 7.9: Design chart for impact energy dissipater

The proportions of the elements of the structure are shown in Figure 7.8, but a structural design may be required for the reinforcement detail.

To simplify the design procedure, the approach flow is converted to equivalent rectangular cross section in which the width is deemed to be twice the depth of flow.

From the design discharge in the pipe, calculate the flow area.

$$A = Q/V_p, \qquad \text{Equation (7.2)}$$

where

Q design flow (m^3/s)

V_p velocity of the partial depth flow in the pipe (m).

Calculate the equivalent depth from $d_e = (A/2)^{0.5}$ (m)

Calculate and approximate the Froude number (Equation. 7.1) of the approach flow, Calculate the specific head at the end of the pipe,

$$H = d_e + (V_p)^2/2g \qquad (m) \qquad \text{Equation (7.3)}$$

Using the Froude number, read the value of H_o/W from Figure 7.11. Divide the specific head, H, by H_o/W to obtain an approximate width, W (m). Then use the nearest standard value of W from Table 7.1, read the rest of the dimensions from Table 7.1.

Calculate the average outlet velocity (Section 7.1.8). Determine the length and size of beaching as set out in Section 8.5.1.

Table 7.1: Impact dissipator – basin dimensions in metres (see Figure 7.10) (after VicRoads, 1989)

W	L	L1	L2	h1	h2	h3	h4	w1	w2	t1	t2	t3	t4	t5
1.5	2.0	0.9	1.1	1.00	0.60	0.25	0.6	0.13	0.4	0.15	0.15	0.15	0.15	0.075
2.0	2.9	1.2	1.7	1.70	0.80	0.35	0.9	0.15	0.6	0.15	0.15	0.15	0.15	0.075
2.5	3.3	1.4	1.9	1.90	0.90	0.40	1.0	0.18	0.7	0.15	0.18	0.18	0.15	0.075
3.0	4.0	1.7	2.3	2.30	1.10	0.50	1.2	0.23	0.8	0.20	0.20	0.23	0.20	0.075
3.5	4.5	2.0	2.5	1.25	0.55	1.40	1.4	0.25	0.9	0.20	0.23	0.23	0.20	0.100
4.0	5.3	2.3	3.0	3.00	1.50	0.65	1.6	0.30	0.9	0.20	0.28	0.25	0.25	0.100
4.5	6.0	2.5	3.5	3.50	1.70	0.75	1.9	0.35	0.9	0.20	0.30	0.30	0.30	0.125
5.0	7.0	3.0	4.0	4.00	1.90	0.85	2.1	0.40	0.9	0.23	0.23	0.30	0.30	0.150
5.5	7.3	3.1	4.2	4.20	2.00	0.90	2.3	0.40	0.9	0.23	0.33	0.33	0.33	0.180
6.0	8.0	3.5	4.5	4.70	2.30	1.00	2.5	0.45	0.9	0.25	0.35	0.35	0.35	0.200

(See over page Figure 10)

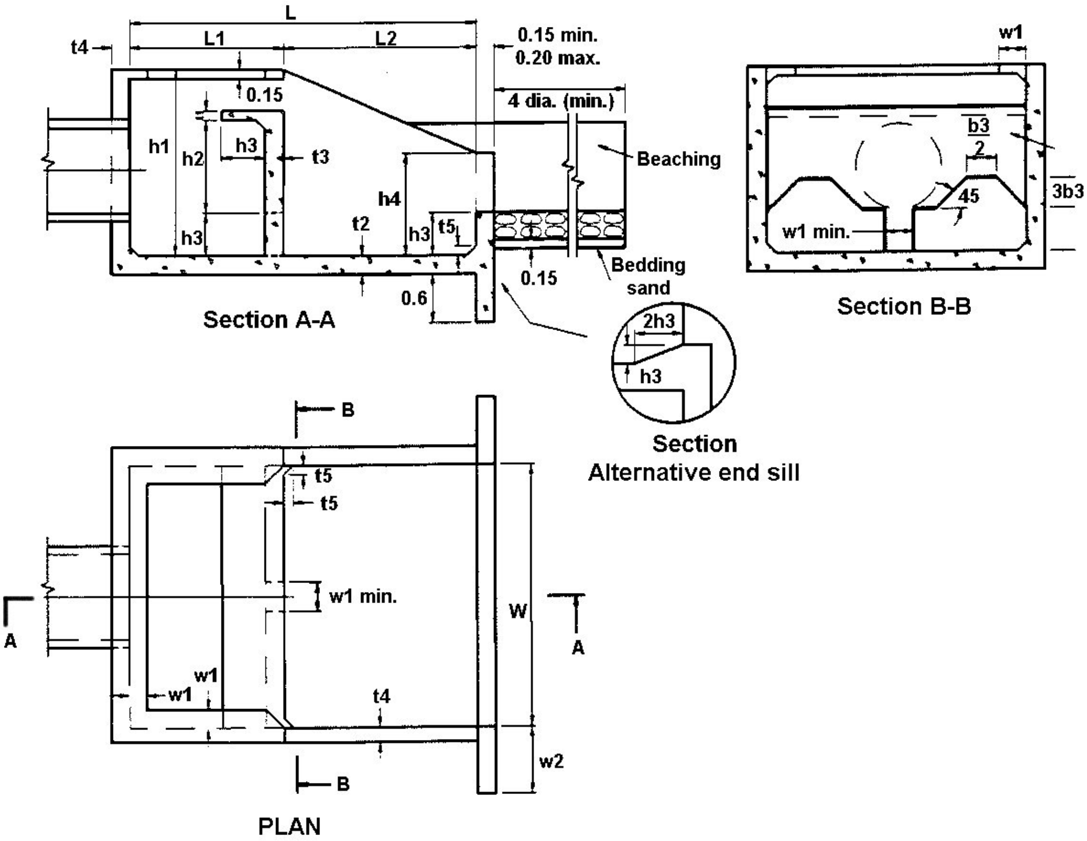


Figure 7.10: Impact type dissipators (after VicRoads, 1989)

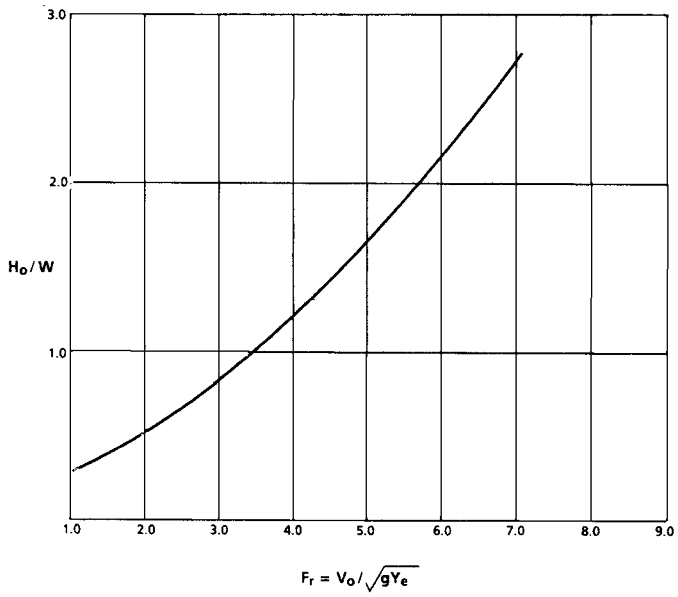


Figure 7.11: Width ratio for impact dissipators (after VicRoads, 1989)

7.2.5 Drop Stilling Basin

A chart for the design of a drop stilling basin is shown in Figure 7.12.

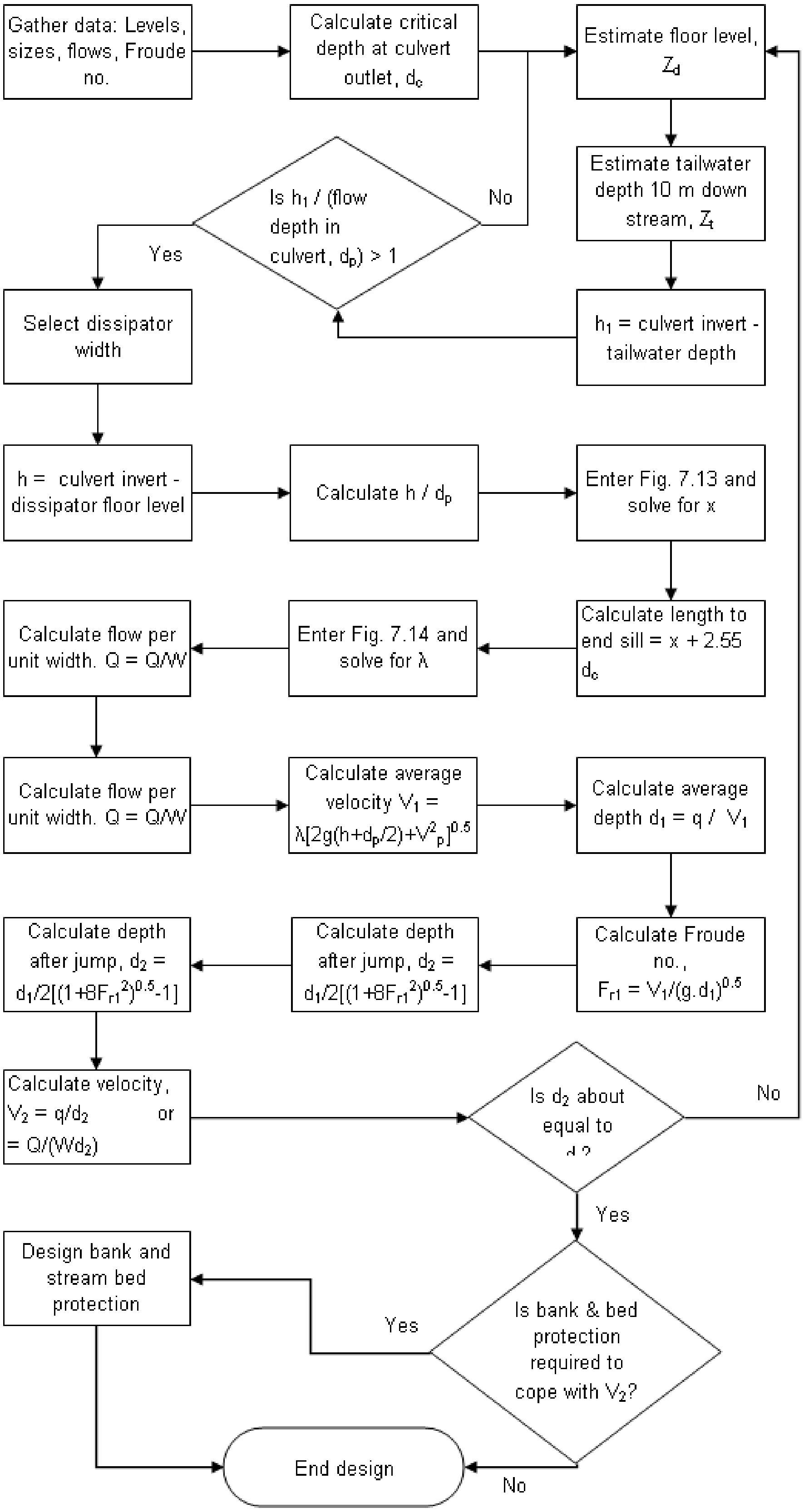


Figure 7.12: Chart for the design of a drop stilling basin

Floor Level and Width

The floor level must be selected so as to keep the tailwater below the culvert invert. Otherwise, there is a tendency for the culvert flow to stay separated on top of the still water, later causing damage downstream by high velocity and wave action.

- Determine the tailwater elevation 10m downstream, Zt (m) and the critical depth of the culvert flow, d_c (m), then estimate a trial dissipator floor level (Zd) from:

  $Zd = Zt - 2.2d_c$

- Check the difference in level between the culvert invert, Zc and the tailwater level, Zt. Compare this difference against the partial depth in the culvert outlet, d_p (m).
- Calculate, $h_1 = Zc - Zt$, it is preferable that $h_1/d_p > 1$. Otherwise, lower the floor level to achieve at least equality.
- Next, select a dissipator width in the range 1.4D<W<2D for a single cell culvert, or (N+0.4)D<W<(N+1)D for multiple cells, where D is pipe diameter (m) and N is the number of cells.

Length to End Sill

The length to the end sill comprises the average horizontal distance for fall of the flow plus the distance required to form the hydraulic jump.

- Calculate the drop from the culvert invert to basin floor, h = Zc – Zd.
- Calculate the ratio h/d_p. Using this ratio and the Froude number of the culvert flow, read $x/F_r.d_p$ from Figure 7.13.
- Multiply the graph value by $(F_r.d_p)$ to find 'x' (m), the horizontal component of the upper nappe.
- Calculate the length to the end sill, L, in metres.

$$L = x + 2.55d_c \qquad \text{Equation (7.4)}$$

where d_c is the critical depth in the culvert. (For d_c refer to Figure 7.4 or Figure 7.7.)

Initial Depth of Flow in the Dissipator

The initial depth of flow is calculated by applying a loss factor, λ to the energy of flow at the top of the overfall, in order to estimate the average velocity on the floor of the dissipator.

- Enter Figure 7.13 with the ratio h/d_p, and read the loss factor, λ.
- Calculate flow per unit width of the dissipator, q = Q/W (m^2/s)
- Calculate the average velocity at the base of the overfall

$$V_1 = \lambda\,(2g(h+d_p/2) + Vp^2)^{0.5} \quad (m/s) \qquad \text{Equation (7.5)}$$

- Calculate the average depth, $d_1 = q/V_1$ (m).

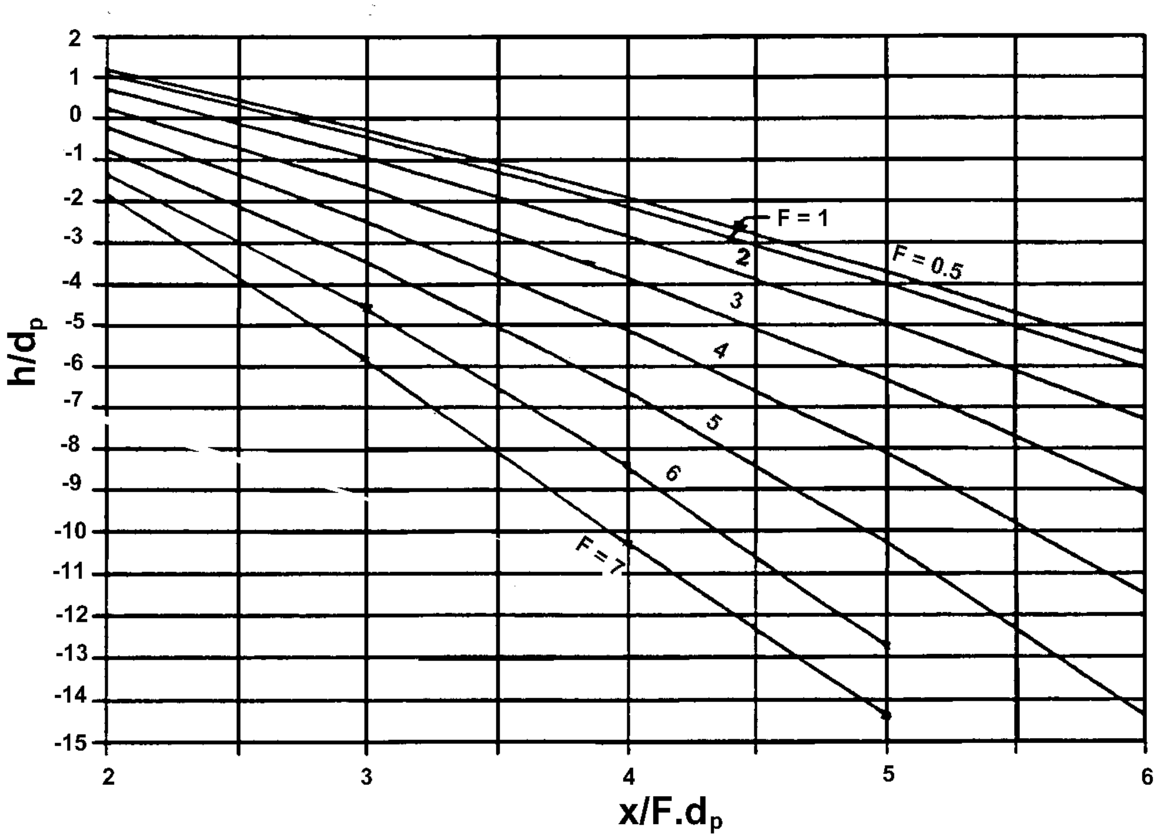


Figure 7.13: Design curves to determine end sill length (after VicRoads, 1989)

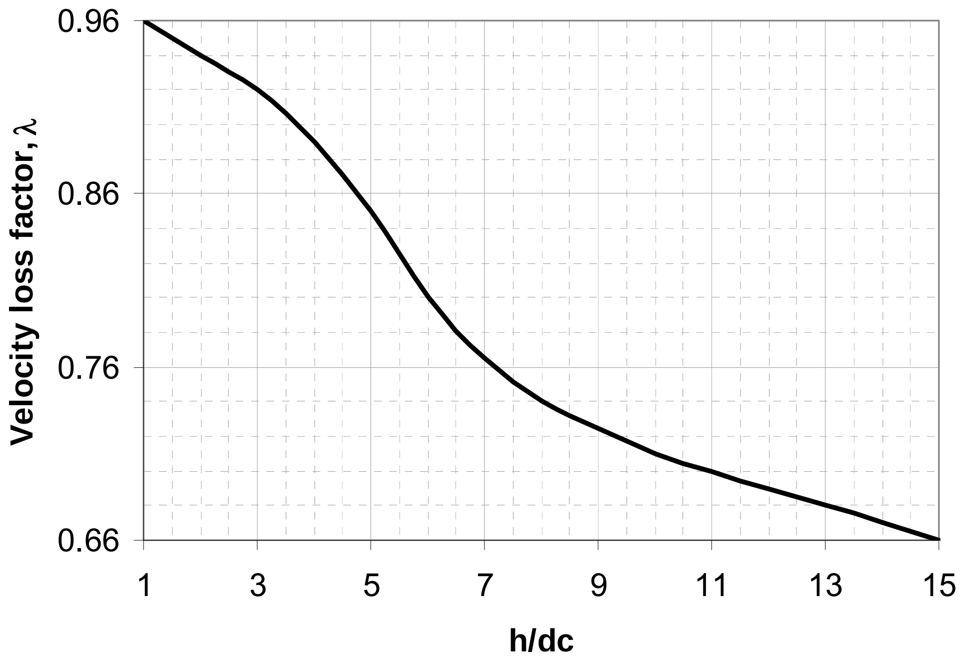


Figure 7.14: Determination of the velocity loss factor (λ) (after VicRoads, 1989)

Depth after Hydraulic Jump

- Calculate the Froude number at the base of the overfall

$$F_{r1} = \frac{V_1}{(g \times d_1)^{0.5}} \qquad \text{Equation (7.6)}$$

where from the previous step

V_1 is the velocity at the base of the overfall (m/s)

d_1 is the average depth(m)

- Then the depth after the jump, d_2 (m) may be found from

$$d_2 = d_1 \times 0.5\left(\left[1 + 8F_{r1}^2\right]^{0.5} - 1\right) \qquad \text{Equation (7.7)}$$

Design Review

Check the calculated parameters against the initial assumptions.

- Ensure that the elevation of the jump, (Zd + d_2) is very close to the tailwater elevation, Zt.
- Also check the velocity at the dissipator outlet against the velocity in the downstream channel. Provide rock beaching where warranted.
- Adjust the floor level and the width of the dissipator and recalculate the parameters until optimum performance is achieved.

CHAPTER 8

8 WATER QUALITY AND EROSION

8.1 Introduction

8.1.1 Scope

This Chapter covers the means of collecting, treating and discharging stormwater from the road reservation and its surroundings. Water quality control is an integral part of the drainage design process.

Detailed consideration of the treatment of stormwater runoff is given in the Austroads Guidelines for the Treatment of Stormwater Runoff From the Road Infrastructure (Austroads, 2003b)

8.1.2 Objectives

The objective of this chapter is to provide information so that a long term solution to stormwater runoff quality can be realised. In order to achieve this goal, drainage designers need to consider strategies to:

- Reduce pollutants and sediments in the stormwater runoff from roads before discharge into receiving waters;
- Control the outflow of stormwater so as not to erode or cause siltation of the receiving waters; and
- Reduce the volume of stormwater to be discharged.

A significant proportion of stormwater pollutants are transported as sediment-bound contaminants. Removal of suspended solids is a critical part of stormwater management. This chapter provides guidance on how a total system can be selected and designed, consisting of a number of individual elements.

Erosion and runoff quality are also discussed in relation to road and/or drainage construction.

8.1.3 Fencing of Stormwater Management Ponds

In built-up areas it is recommended that all stormwater management ponds be reviewed with respect to public access and liability.

In the absence of requirements from local jurisdictions the following guidelines are offered:

1. Ponds with batters steeper than 1 in 6 should be fenced. The fence should be fitted with a locked gate to provide access for maintenance plant. The type of fence used will be site specific and depends on the level of protection required but should generally be 1200 mm chainwire or 1800 mm chainwire fence. If a pond is located outside the road reserve on local government land then local government requirements will also need to be considered. See point 3 below when site amenity is an issue.
2. When a pond is located close to the carriageway and steep slopes or water depth are likely to be a hazard to motorists, a safety barrier should be considered.
3. A safety margin consisting of a shallow perimeter (< 600 mm and a slope of no steeper than 1 in 10) one or two metres wide may also be considered to provide additional safety, particularly where the pond has a high amenity value and a fence would detract from this.

4. Where required, warning signs should be displayed prominently and these should state that the water may be contaminated from road runoff.

8.2 Background

8.2.1 Sources of Pollution from Roads

Land development and road construction can lead to soil erosion and consequent sedimentation. 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.

Road operation contributes litter, suspended solids, toxicants, oil and other hydrocarbons to storm runoff.

Tyre wear contributes to road grit and traces of the zinc used in tyre 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 to stormwater.

Large car parks near regional shopping centres are the sources of significant quantities of litter including plastic containers, packaging, and plastic bags. Carparks are also major sources of oil, grease and other hydrocarbons.

Residential streets, in addition to any pollutants from road operation, contribute rotted 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 have a greater impact on the catchment hydrology.

However, the quality of runoff depends on traffic volumes. Pollution loading increases as traffic volumes increase and is also increased in areas of acceleration and/or braking. Although divided arterials and freeways may only represent a minor proportion of catchment imperviousness, the runoff from them can be a significant source of pollutants. There is evidence to suggest that the contribution of high-volume roads to increased pollutant concentrations in runoff, can be similar to that found in industrial and commercial areas and an order of magnitude higher than medium density residential areas (Wong *et al*., 2000).

A significant amount of pollutants, ranging from litter to solid or soluble particles, accumulate on roads and car parks and are conveyed into stormwater drainage networks during storm events. Between runoff events, pollutants enter drainage networks as a result of ineffective street sweeping, wind action and 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 storm water 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 *et al*. (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.

8.2.2 Erosion Estimates

An assessment needs to be undertaken to estimate the susceptibility of the soils to erosion. Soil maps exist for Australia and New Zealand and these 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 runoff volume (retention), promotes infiltration and decreases flow velocities (retardation). The level of erosion protection varies depending on the vegetative cover provided (Table 8.1). Protection can also be provided using various other coverings (Table 8.1).

Table 8: Protection provided by vegetative cover

Description	Protection Provided
Well established and stable vegetative cover	High
Grassed areas with or without intermittent taller growth	High to Intermediate
Lightly timbered area with some grass cover	Intermediate
Re establishing growth with soil visible in places	Intermediate to Low
Bare earth with little or no vegetative cover	Low

Table 8.1: Cover and management factor (after VicRoads, 1995a)

Description	Factor
No cover	1.00
Straw secured with netting	0.20
Mulch	0.10
Fibre glass rovings	0.05
Bitumen emulsion	0.02
Grass (few trees)	0.04
Shrubs	0.01
Trees (few shrubs)	0.01

Erosion estimates are required for the design of erosion protection measures, such as temporary sediment basins during construction (see also Section 2.2). 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 Rainfall runoff erosivity factor (it can be estimated from $R = 164.74 \times 1.118I \times I^{0.644}$; where I is the 2 year, 6 hour log-Pearson type III rainfall intensity (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 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; This is the product of the slope factor and the steepness factor. Slope length factor is 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.

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

C Cover management factor allows for erosion protection practices such as provision of vegetation or other erosion protective measures, (e.g. mulching), see Table 8.2: for other common practices.

The detailed method may be found in VicRoads (1995a), or a computer-assisted method called SOILOSS is published in Rosewall and Edwards (1988).

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 8.2: .

Table 8.2: Typical erosion estimates for erodible soils (A_s) (after VicRoads, 1995)

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

8.2.3 Water Quality Standards

The water quality standards to be achieved in receiving streams and for outflows from construction sites may be prescribed by the local regulatory authority and/or the environmental protection agency, usually supported by State legislation. Water quality indicators and objectives may differ according to the proposed use of the water, for example, agriculture, recreation or potable water supply.

The indicators most often specified and monitored for road construction and operation are pH, litter, suspended solids and turbidity. Testing for traces of heavy metals and polycyclic aromatic hydrocarbons is expensive, and would be carried out only where a problem is anticipated. A simple criterion for hydrocarbons is that stormwater outflows should not carry visible oil slicks.

Stormwater runoff from residential areas and streets may also need to be tested for Biological Oxygen Demand (BOD), Total Phosphorus (TP), and/or Total Nitrogen (TN).

Standards for road operation are more likely to be specified in terms of annual pollution loads, whereas during construction the requirements may be in terms of daily runoff events. For example, in relation to suspended solids, the long-term target may be 80% retention of the annual load. During construction the requirement may be effective treatment of 90% of daily runoff events and achievement of an average concentration of 50 mg/l (Victorian Stormwater Committee, 1999).

Runoff quality treatment measures need only cope with relatively frequent events as this provides a significant improvement over the do-nothing scenario. Provision of treatment measures to cope with a small event ARI (up to say 1 year ARI) can remove up to about 98% of the sediments from stormwater. However, as stated earlier, the discharge water quality will often be stipulated by a controlling authority and it is up to the drainage designer to provide the means of achieving the desired result.

8.2.4 Stormwater Treatment Sequence

There is no single device that can economically treat stormwater for all pollutants. Road runoff requires two or more 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 complete 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 BOD.

Not all of these steps may be taken in practice, or they may occur at different locations in the system. It is more efficient to trap the larger objects and particles close to the source.

Typical treatment measures include vegetated buffer strips, swales, litter traps, sediment basins, wetlands and regional lakes. A detailed discussion of treatment measures is set out in Section 8.3. For effective operation of devices, a regular maintenance program is essential. Also, they should be inspected and maintained after large storm events.

8.2.5 Capture of Toxic Spills

Occasionally trucks overturn, spilling their loads on to 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 at any time, 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 m^3 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.

8.3 Water Treatment Devices

8.3.1 Buffer Strips

A buffer strip is a densely vegetated area separating the road from the receiving stream by at least 20 metres, through which stormwater passes as overland flow. Eucalypt forest is not as effective as grassed buffer strips, possibly due to the lower density of grass growth under the eucalypts.

The length of buffer strip, slope, vegetative characteristics, catchment characteristics and runoff velocity are all factors that may affect the pollutant removal efficiency of buffer strips. The method is not so effective when slopes exceed 17%, since steep slopes lead to the development of rills and scour and subsequent erosion of the slope.

A buffer strip can achieve 98% removal of 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, perhaps due to the high association of phosphorus with clay sized sediment. For the same reason, buffer strips are not suitable if removal of heavy metals, polycyclic aromatic hydrocarbons and nutrients is required.

8.3.2 Swales

Background

Swale drains are gently sloping vegetated drains that are preferable to conventional kerb and channel for water quality purposes. They differ from conventional table drains and catch drains in density of vegetation, and the depth of flow should be below the top of the grass for the 1 year ARI event. However, urban use of swales is limited to low-density residential areas where road crown levels are at or slightly above local natural surface levels and space is available within the road reserve.

Flow times in these drains are about 5-6 times longer than in concrete kerb and channels, so they serve to detain the runoff and to increase infiltration. The lower velocities allow heavier fractions of the suspended particles to settle out. Grass and other vegetation in the drains act as a filter, removing many pollutants, with reported removal efficiencies of 83% for sediment, 75% for hydrocarbons, 67% for lead, 63% for zinc and 63% for aluminium.

Design principles

The location and side slopes of swales should be designed in accordance with Figs. 6.1 or 6.2. Longitudinal gradient should be between 2% and 4% to promote uniform flow conditions across the cross section of the swale. Swale base width should be not more than 2.5 m, unless measures are used to ensure uniform spread of flow, such as check dams. Check dams (see Figure 8.1) can be constructed from a range of materials including straw bales, filled bags, timber or earth and should be installed if slopes exceed 4%.

Figure 8.1: Rock check dams

Maximum flow velocity should be less than the scouring velocity of the soil shown in Table 8.3 or less than 0.5 m/s (whichever is the smaller) for the 1 year ARI event and not more than 1.0 m/s for the 100 year ARI event. The discharge from a swale is given by Manning's Equation;

$$Q = A \times R^{2/3} S^{1/2} / n \qquad \text{Equation (8.1)}$$

where

Q discharge (m^3/s)

A cross sectional area of inundated swale (m^2)

R hydraulic radius (m)

S slope (m/m)

n Mannings n (see Appendix 6B, but usually taken as 0.2 for flow depths below the height of the vegetation, reducing to 0.15 at 100 mm above the top of vegetation, and 0.03 for flow depths 400 mm above the top of vegetation).

The velocity of the flow in the channel is essential to ensuring water quality and preservation of the channel, and is determined using;

$$V = Q / A \qquad \text{Equation (8.2)}$$

where

A cross sectional area of inundated swale (m^2)

Table 8.3: Maximum velocity over surface (after Austroads, 2001)

Soil description	Maximum velocity (m/s)	Soil description	Maximum velocity (m/s)
Bare Earth Surfaces			
Sand	0.4	Sandy Loam	0.5
Silty Loam	0.6	Stiff Clay	1.0
Fine Gravel	0.8	Coarse Gravel	1.0
Vegetative grass covered surfaces			
Grass Type	**Slope (%)**	**Maximum velocity (m/s)**	
		Erosion resistant soils	**Erosion susceptible soils**
Grass mixtures	0 to 5	1.5	1.2
	5 to 10	1.0	0.8
Kikuyu	0 to 5	2.5	2.2
	5 to 10	2.2	2.0
	> 10	2.0	1.8
Couch, bent or fescue	0 to 5	2.2	2.0
	5 to 10	2.0	1.8
	> 10	1.8	1.5

Worked examples are found in Austroads (2003b).

8.3.3 Infiltration Systems

Infiltration is believed to have high pollutant removal efficiency and can also help recharge the ground water, thus restoring low flows to stream systems. Infiltration basins can be challenging to apply on many sites because of soil requirements. Infiltration basins can have relatively high failure rates compared with other stormwater management practices and need to be designed and constructed to high standards.

Infiltration systems are most commonly employed as a source control measure and as such can be constructed on a small scale to capture runoff from small contributing areas. However, infiltration basins can cater for contributing areas up to 10 ha in size.

When assessing the applicability of infiltration systems, the following need to be considered:

- The depth to the ground water table.
- Soil conditions (i.e. is the soil sufficiently permeable?).
- Pre-treatment (need to prevent siltation and clogging).
- Available area (infiltration basins can be large).
- Proximity to other facilities (will the additional soil moisture adversely affect foundations of the pavement or other nearby structures?).
- Slope of site (need to placed with very small slopes).
- Ponding depth of infiltration basins should be no more than 0.6 m.
- Construction issues.
- Can the systems easily be maintained? (Can back flows be used to flush the system, or the filter bed replaced?)

Permeable pavements

Permeable pavements (i.e. total pavement structure is Permeable not only the surfacing) are not practical for arterial roads.

Permeable pavements are commonly used overseas in open car parks and driveways. They are constructed from modular or lattice paving. These paving blocks are used to provide structural support, while retaining a large proportion of the 'paved-area' pervious for infiltration of rainfall and ponded storm water. Permeable pavements have also been used but this form of construction requires good drainage conditions to ensure adequate subgrade bearing capacity.

There are no data available on the effectiveness of Permeable pavements in reducing the quantity of storm water runoff. Their most appropriate applications would be as a source control measure, used in conjunction with a storm water infiltration system through a sand-mix medium.

It is difficult to prevent pollutants from entering groundwater when using Permeable pavements and the effect of this should be considered. Permeable pavements should not be placed in areas that are liable to experience a high pollutant loading.

Infiltration trenches

Infiltration trenches (see Figure 8.2) are widely used overseas, particularly as source controls for urban allotments. There are proprietary products available that encompass a range of filter mediums and storage capacities. Generally the capacity of infiltration trenches is not sufficient for runoff from roads except for short sections.

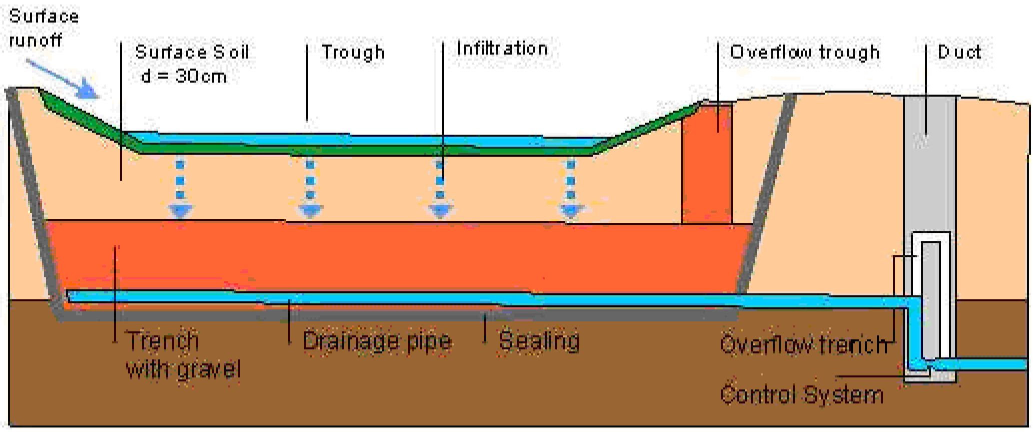


Figure 8.2: Typical infiltration trench installation (after, Sieker, undated)

For the system to operate effectively the permeability of the soil needs to be at least 3 x 106 m/s (coarse silts and coarser).

The design approach is based on capturing the first flush, or a percentage of runoff from the road reserve. Typically, 95% of all runoff would be treated, which corresponds to a storm event with an ARI of about 6 months. The inflow rate (for the design event) would be balanced against the storage capacity and the outflow via infiltration, to determine the length of trench required. The overflow system would need to be designed to cater for storm events greater than the design event.

Infiltration basins

An infiltration basin is an impoundment that is designed to infiltrate stormwater into the ground water. 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 runoff 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 runoff
- to attenuate channel (stream) discharge; or
- risk management.

Typically, on-site detention and infiltration basins are designed with an ARI of between two to ten 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. A design outline is shown in Figure 8.3.

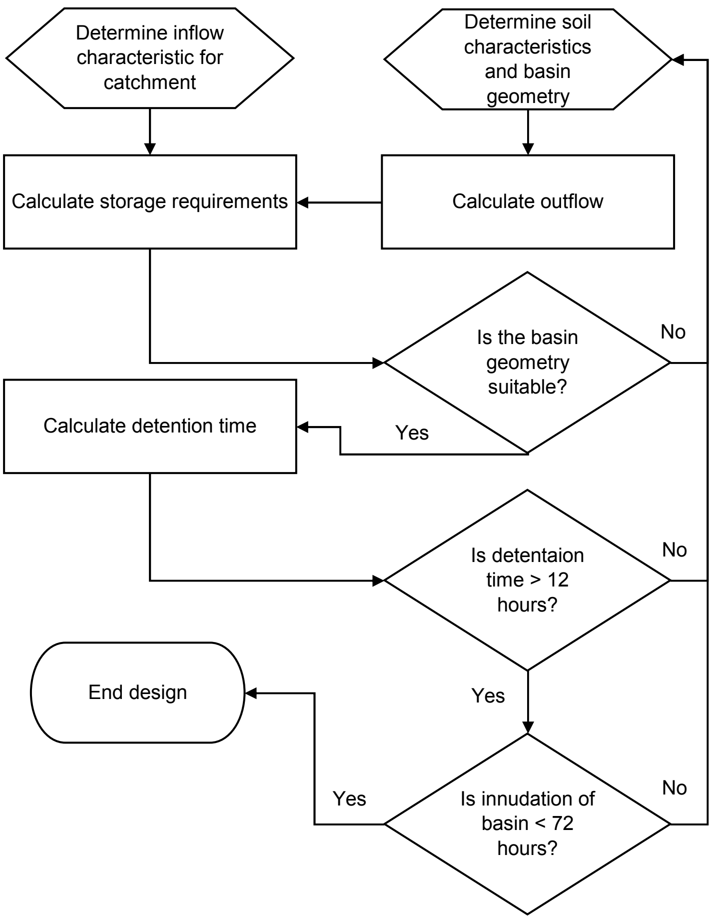


Figure 8.3: Design procedure for an infiltration basin

An inflow hydrograph is constructed for the basin catchment area. The outflow (Equation (8.4), see Austroads, 2003a for more detail) is then subtracted from this and would yield the maximum storage capacity of the basin. The size of the basin could then be determined on the basis that the depth should be limited to 0.6 m to prevent compaction of the basin floor. See example in Appendix F.

$$\text{Outflow} = \frac{K\pi T\left(H^{'} + d\right)^2}{\ln\left(R/r\right)} \qquad \text{Equation (8.3)}$$

where

Outflow (m^3) typically the outflow would be calculated at time 2T, i.e. when there is no more runoff entering the basin

K soil permeability (m/s)

T storm duration (s)

H’ half depth of water in the basin (m)

d distance from floor of the basin to the water table (m)

r half wetted perimeter (m)

R radius of influence from the centre of the basin.

$$R = r + 50(H + d)K^{0.5} \qquad \text{Equation (8.4)}$$

where

H depth of water in basin (m)

Where the infiltration basin is installed above natural ground level, or the slope of the natural ground is sufficient, then an underground drain fitted with a valve should be installed to facilitate draining of the basin (see Figure 8.4). This would enable the basin to be drained in the event of water ponding (clogging). Some pre-treatment may be required to ensure that most of the litter, sediments and pollutants are removed from the runoff prior to the infiltration basin. However, 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.

If the infiltration basin is excavated in flat country then the installation of an underground drain would be impractical.

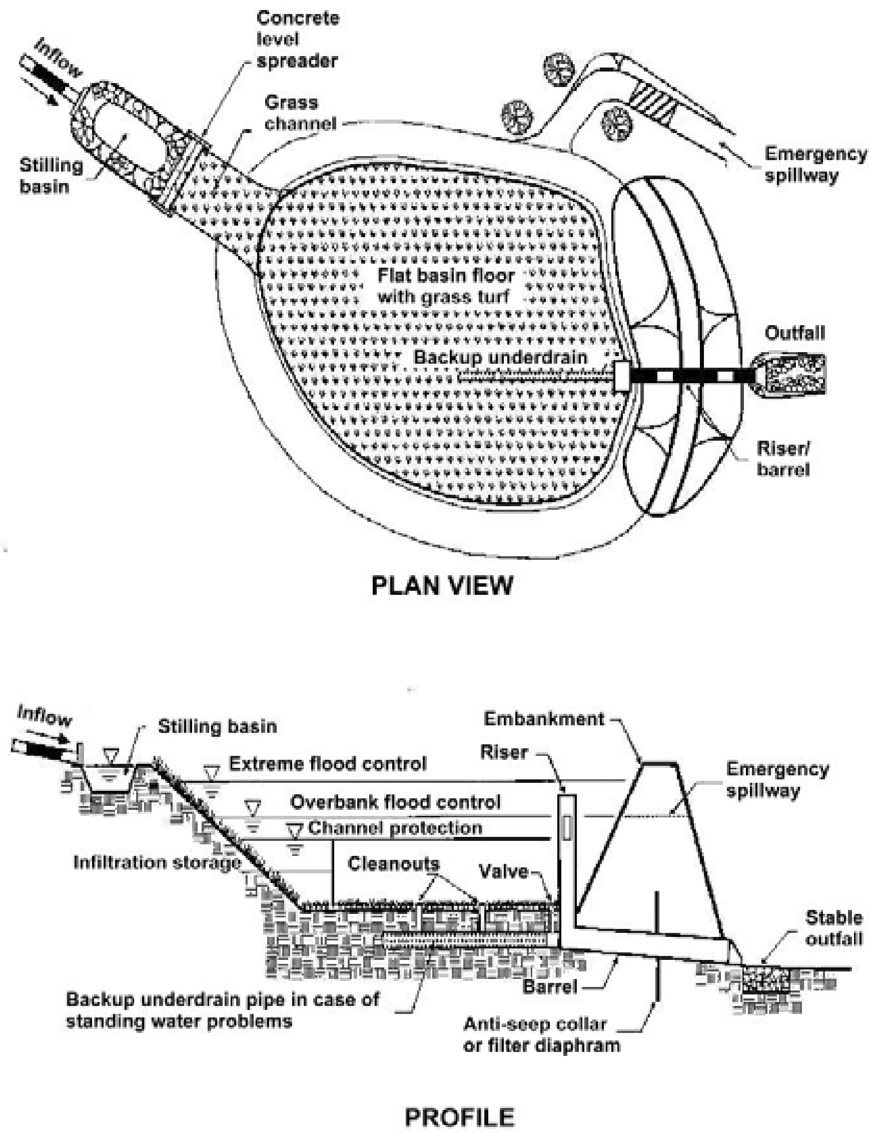


Figure 8.4: Typical layout for an infiltration basin (pre-treatment zones not shown) (after Young *et al*., 1996)

The basin floor should be covered with grasses to assist in maintaining permeability within the upper soil layer. The grass selected should be capable of withstanding periods of inundation. The basin should drain within 72 hours of the storm event to prevent mosquitoes from breeding. Detention time is determined from;

$$\text{Detention time (s)} = \frac{d}{K\left(\frac{H'+d}{d}\right)} \qquad \text{Equation (8.5)}$$

Where the parameters are those for Equation (8.3) & Equation (8.4).

It is also desirable that the detention within the soil above the groundwater table should be at least 12 hours. For this case, H' in Equation (8.5) would be set to zero.

8.3.4 Grated Pits

Grated pits are favoured in water quality systems for their propensity to catch vegetation, litter and debris. Clearly such pits must also be designed to achieve adequate hydraulic capacity (see Section 6.3.2).

They may facilitate rubbish trapping in rural areas on facilities where underground drainage is provided, such as rest areas, truck parking bays and wayside stops. Maintenance requirements, particularly in rural areas can be high and this should be considered when designing a water treatment system. If trapping rubbish is a requirement, it should be done by a trap designed for the purpose, which is easy to clean, and is fail safe in the case of blockages.

8.3.5 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.

Side Entry Pit Baskets

Litter baskets are not required in every pit. Traps should be placed where the quantities of litter are expected to be sufficient for economical maintenance such as near:

- regional shopping areas
- local shopping areas or corner stores
- fast food outlets
- schools; or
- sports grounds.

Litter baskets should have 10 – 20 mm wide openings. Plastic moulded traps with perforations tend to block easily with leaves and fine materials. There must be sufficient waterway around the basket to bypass the basket if it becomes blocked or full.

Trash Racks

Trash racks are commonly placed in-stream to trap the largest gross pollutants. They may be incorporated into other forms of gross pollutant traps, and are an essential first stage at pumping stations. A bar spacing of 60 mm will trap most cans and bottles, but may become blocked if there is a significant amount of vegetative matter.

Upward sloping of the trash rack in the direction of flow may allow rubbish to be pushed up the rack during higher flows, leaving the lower section clear for passage of water.

The trash rack should be designed to have about 0.3 m freeboard above the design discharge so that floating materials are retained in the design event. However, an allowance should be made for the design discharge to flow over the rack without causing damage downstream in the event that the rack becomes fully blocked.

Racks require regular maintenance, and consideration must be given to the means of providing vehicular access, and to the method of disposing of the accumulated rubbish.

8.3.6 Gross Pollutant Traps

Background

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.

A typical gross pollutant trap is shown in Figure 8.5 and consists of five elements:

- an inlet area designed to spread the flow uniformly across the width of the trap
- a sediment trap designed to settle coarse particles out of the flow
- a trash rack to capture floating rubbish
- access ramps to allow machinery to collect and remove accumulated materials; and
- an outlet to return the flow to the stream or water quality control pond.

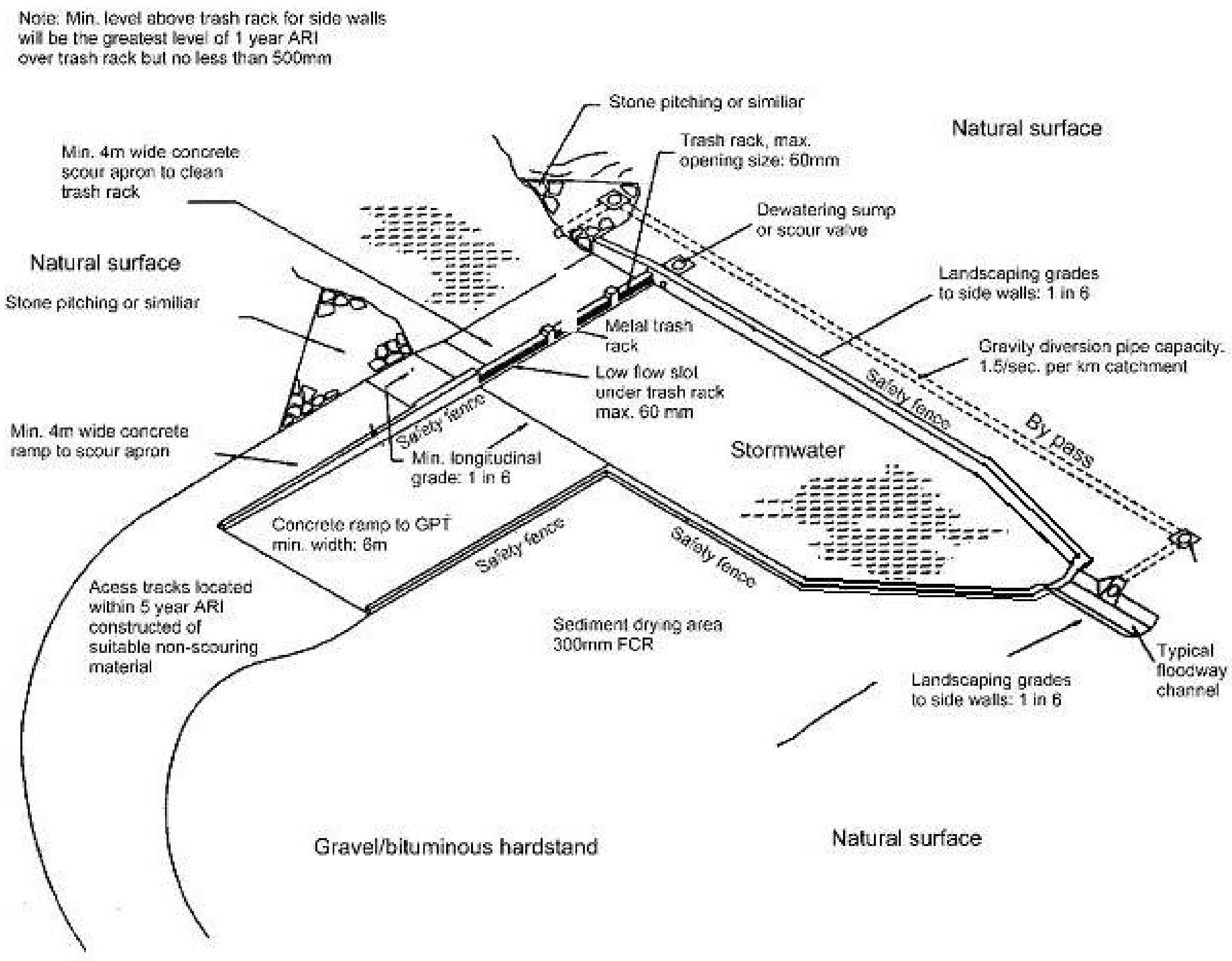


Figure 8.5: Typical gross pollutant trap (adapted from Department of Urban Services, 1987)

Design principles

Two types of hydraulic design criteria apply:

1. The trap must collect rubbish and deposit sediment to the specified requirements in a 1 year ARI event.
2. The trap must be structurally and hydraulically safe in an extreme event, usually taken as the 100 year ARI storm, even though the litter and sediment trapping functions may become ineffective in such an event.

Other design criteria for non-proprietary pollutant traps include:

- The angle of width transition at the inlet should be about 30 degrees.
- The inlet should have a smooth hydraulic transition to achieve uniform spread of flow.
- The sediment basin should capture 75% of particles larger than 0.05 mm.
- The maximum flow velocity for a 1 year ARI event should be less than 0.3 m/s, to avoid stirring up the accumulated sediment.
- The ratio of length to width should be about 3:1.
- The minimum trap depth is 0.8 metres, including 0.4 metre storage zone.
- The trap should hold about 3 month's deposit of sediment. The actual volume of silt to be deposited will be a function of the size and erosivity of the catchment.
- The sediment trap should be able to be completely drained by gravity flow for cleaning.
- The maximum slope of an earth batter in contact with water should be 3 to 1. Where public access is readily available, the side slope should be around 6 to 1.
- The structure should not adversely affect flood levels upstream or downstream.

Gross pollutant traps will only remain effective if they are cleaned regularly. A program for inspection and maintenance is essential. They should be inspected on a regular basis and after every major storm event. Sediments and trash removed from a GPT should be considered as commercial waste and need to be disposed of in a responsible manner.

Variants of a gross pollution trap include the commercially available continuous flow separator. The supplier usually provides the design for proprietary devices.

An in-line device (Figure 8.6) may be designed for larger flows, and successfully trap vegetative matter and coarse sediment, while the system ensures that the separating screen remains unblocked.

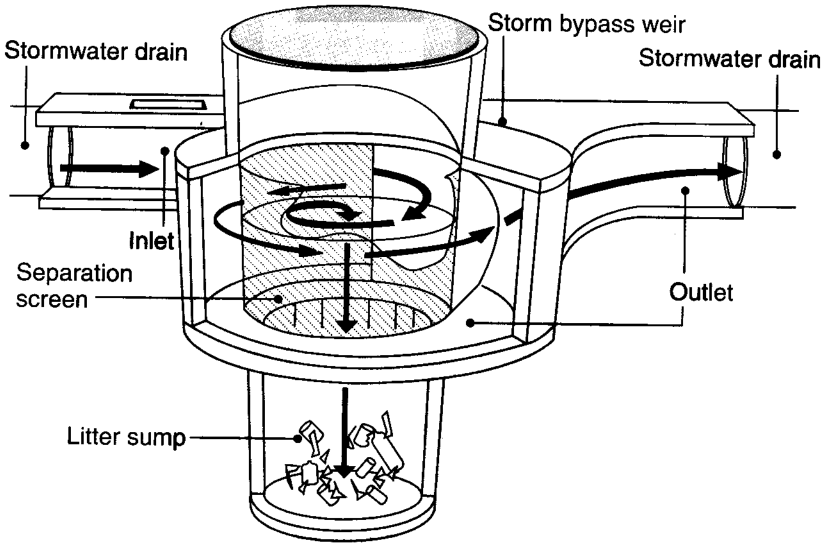


Figure 8.6: Continuous deflective separation trapping system (after EPA, 1998)

8.3.7 Separators

Separators can be ‘in-line’ or ‘off-line’. In-line refers to the placement of the unit in that it is occurs within the flow of the storm water. Whereas off-line refers to the practice of shunting the stormwater to a separate area for separating out the pollutants.

The cylindrical unit shown in Figure 8.7 (Humeceptor, there are other propriety products that fulfil the same purpose) is an ‘in-line separator’ and is designed to trap oil, floating materials and coarse sediment for small flows such as those from an industrial site or a short length of arterial road. Such units can be expensive but may be expedient where there is no space for a more conventional and expensive sediment basin. The supplier usually provides the design for proprietary devices.

These type of devices need to be checked and cleaned out periodically, particularly after a substantial storm event.

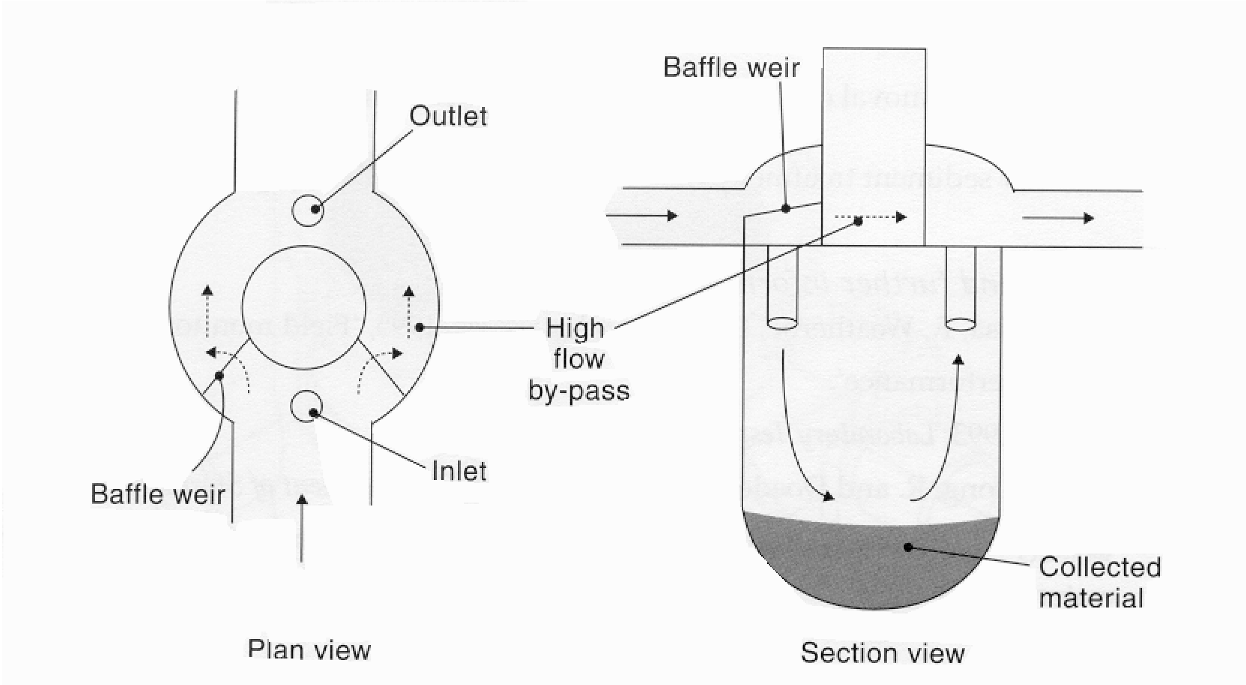


Figure 8.7: Circulating settling unit (after EPA, 1999)

8.3.8 Sedimentation Basins

Background

Sedimentation basins are designed to remove suspended solids from runoff. 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 8.8.

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.

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 over time, fill up with sediment. Consideration needs to be given 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.

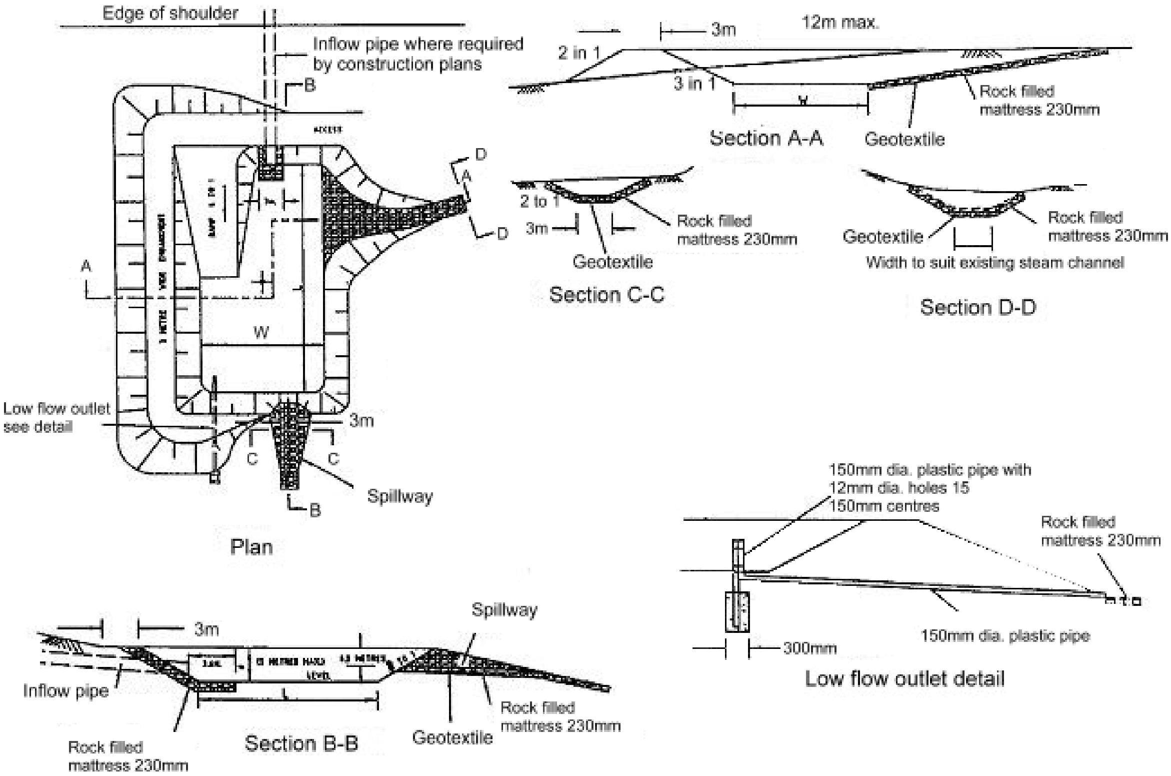


Figure 8.8: Typical sedimentation basin (after VicRoads, 1995a)

Two hydraulic design criteria apply:

- the trap must deposit sediment to the specified requirements in a design event, for example a 6 month ARI for a temporary basin or 2 year ARI event for a permanent basin
- the trap must be structurally and hydraulically safe in an extreme event, usually taken as the 100 year ARI storm.

Design Procedure

A comprehensive design procedure for determining the size of sedimentation basins is given in '*Managing urban stormwater – Soils and construction*, (Dept. of Housing, 1998)' However, this guide was developed for NSW and specific site conditions would need to be checked against the assumptions within the guide. An alternative procedure is described below and illustrated by the flow chart in Figure 8.9.

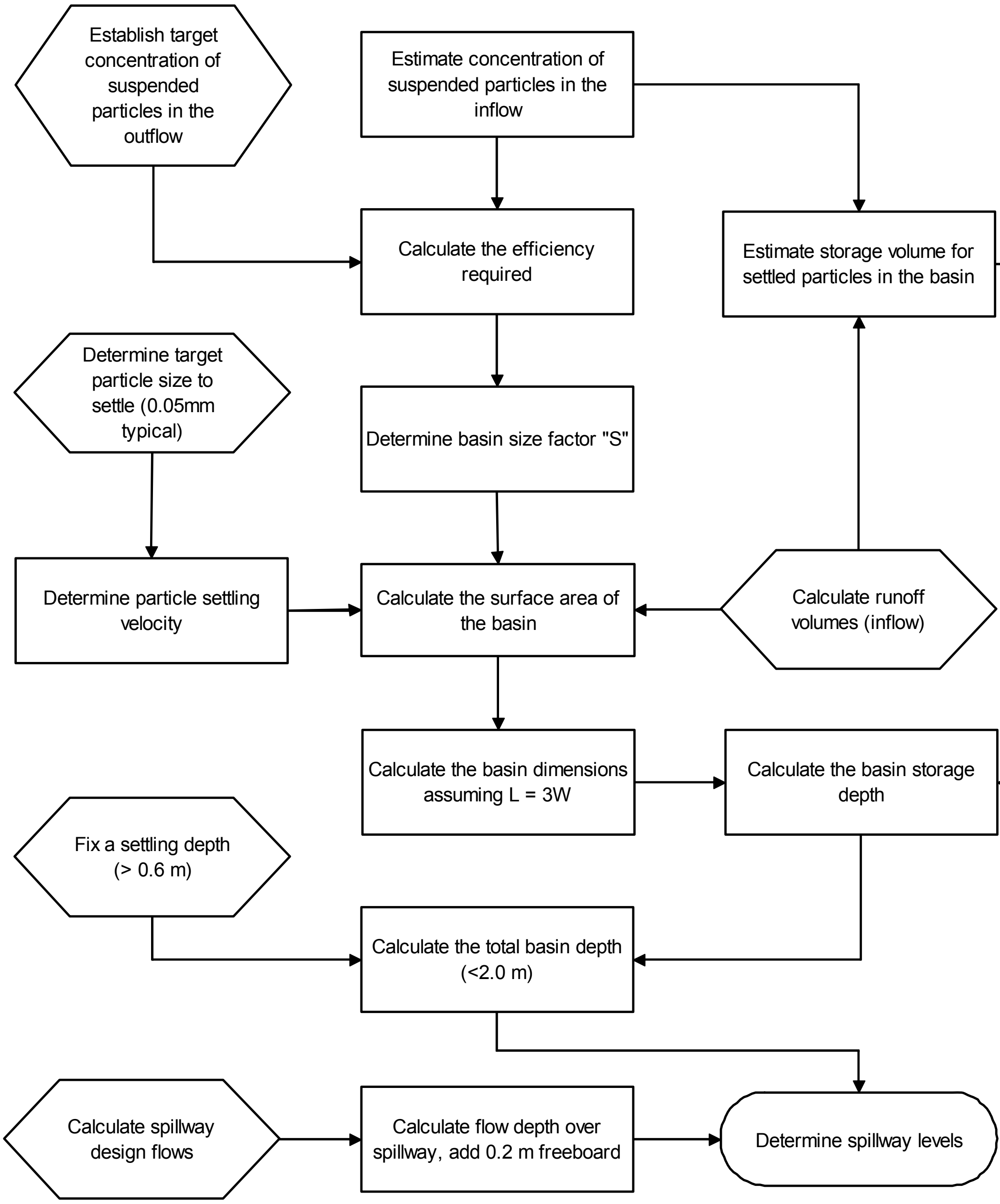


Figure 8.9: Design flow chart for sedimentation basin

- Calculate the design flow for the basin by the Rational Method, typically 2 year ARI.
- Calculate the spillway design flow, 100 year ARI for the catchment time of concentration.
- Determine the size of the target particles. A particle diameter of 0.05 mm is recommended for a target size for all areas except catchments with coarse soils where a larger size particle can be adopted.
- From Figure 8.10 read the average vertical settling velocity of the target particles, V_s.

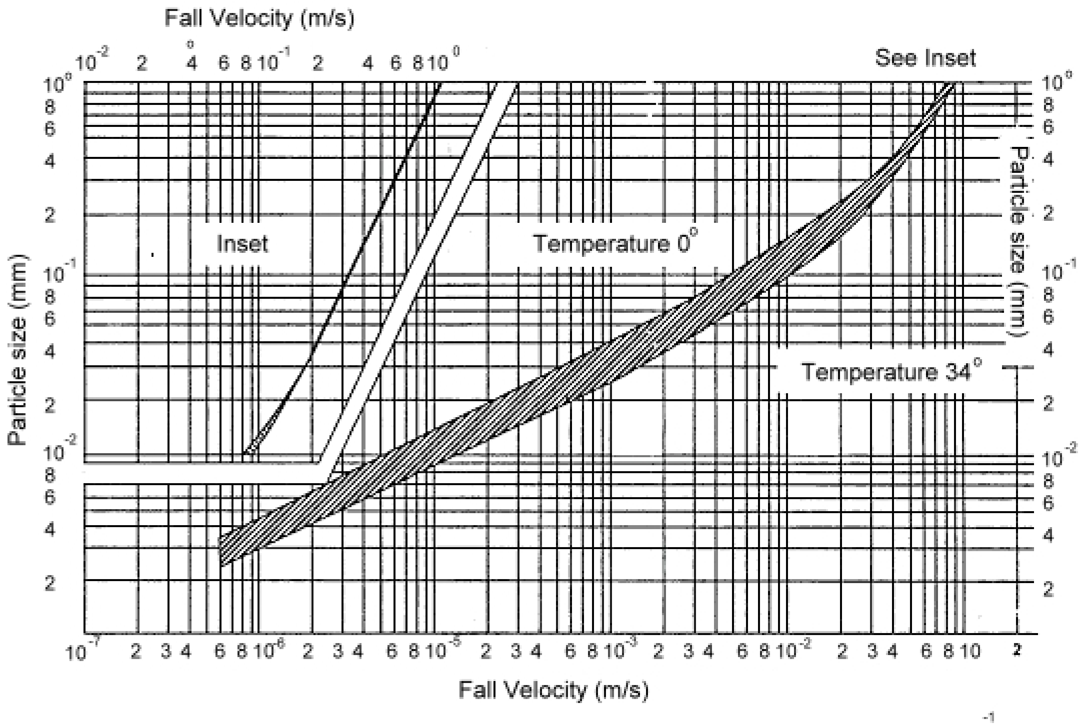


Figure 8.10: Particle velocity settling curves (after VicRoads, 1995)

- Where the allowable average event concentration has been specified, calculate the required basin efficiency using;

$$E = 100(C_i - C_o)/C_i \qquad \text{Equation (8.6)}$$

where

- E required efficiency (%)
- C_i concentration of suspended solids in the inflow (mg/l)
- C_o specified concentration in the outflow (mg/l)

- Using Figure 8.11 for the required efficiency E, read the basin size factor, S.
- Calculate the minimum basin surface area.

$$A_b = S.Q/V_s \qquad \text{Equation (8.7)}$$

where

A_b basin surface area (m^2)

S basin size factor from Figure 8.11

Q design flow into basin (m^3/s)

V_s average vertical settling velocity (m/s)

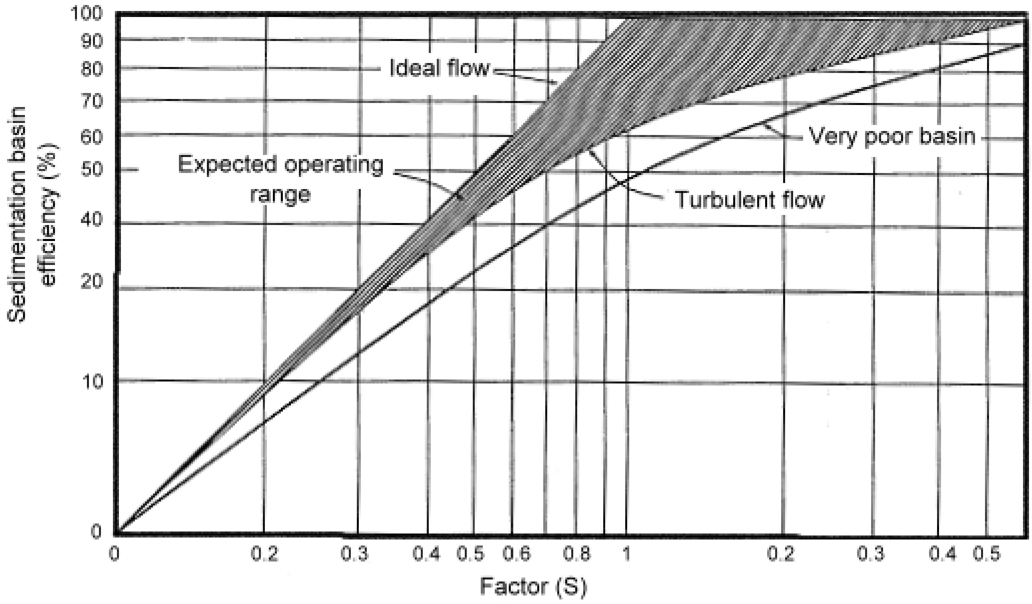


Figure 8.11: Sedimentation basin efficiency design curve (after VicRoads, 1995a)

- Estimate basin length and width, assuming L = 3W.

$$W = (A_b/3)^{0.5} \quad \text{Equation (8.8)}$$

where

W basin width (m)

A_b basin surface area (m^2)

The proportions of the basin may be varied to suit the available space. However, the length should be less than 200 times the settling depth.

- Estimate the basin depth.

Basin depth is the sum of the settling depth and the storage depth. The minimum settling depth shall be 0.6 metres. The storage depth is a function of the soil type and the required frequency of cleaning.

The sediment yield to the basin can be estimated by applying the universal soil loss equation to the catchment. However, it is preferable that the total depth should not exceed 2 metres.

- Estimate the dimensions of the spillway.

The minimum spillway width shall be 2 metres. The depth of flow can be calculated using the weir equation and the 100 year ARI discharge,

$$h = (Q_{100}/ C_w.L_w)^{0.6667} \quad \text{Equation (8.9)}$$

where

h depth of flow over spillway (m)

Q_{100} spillway design discharge (m^3/s)

C_w weir coefficient, usually about 1.6 (See Austroads, 1994)

L_w length of weir (m)

Spillway flow depth desirably should be less than 0.4 m.

- Apply freeboard to the spillway depth.

  The spillway depth should be greater than the depth of flow, to allow for waves and for the inherent inaccuracy in estimating a 100 year flow. The minimum freeboard is 0.2 metres.

- Review the design.

  Review the proposed basin layout to ensure that all specified requirements are met, and that efficiency is optimised.

8.3.9 Bioretention Systems

Background

Bioretention systems are appropriate for the treatment of fine particulates and '*Road Runoff and Drainage: Environmental Impacts and Management Options'* (Austroads 2001) provides some detailed background on this type of facility. A significant proportion of pollutants are closely associated with the finer fraction of suspended solids in stormwater runoff from highways. The use of such infiltration systems is appropriate to sites with moderately permeable soil types. Effective operation of the filter relies on the provision of pre-treatment in a swale to remove the coarse particles.

Vegetating the surface of the infiltration medium is beneficial. The root system in the soil promotes biological treatment processes that can result in a more effective retention of particulate-bound contaminants. A schematic diagram of a treatment system for a car park is shown in Figure 8.12. Typical operating hydraulic loading (flow per unit area per unit time) of these systems would range between 30 m/yr and 300 m/yr and as a consequence, they have high treatment area to catchment area ratio requirements.

The best approach is a combination of flow detention and infiltration. Stormwater is detained for a period between 24 to 48 hours, as it flows through the infiltration medium. Outflows from the filter are collected in perforated pipes for discharge into the receiving waters to prevent any interaction with the groundwater. A surcharge bypass should be provided as part of the system to enable flows from large storm events to be managed safely.

Runoff from roads and highways will enter the system laterally through gaps cut through the kerb at intervals or as sheet flow over a grass buffer into the grassed swale drain. The appropriate selection of grass and vegetation species is necessary to ensure a uniform cover of fine, hardy vegetation that can withstand the prevailing moist conditions. Wetland adapted species, such as *Juncus* and *Scirpus,* may be utilised if surface drainage is poor. A typical depth of the infiltration medium is of the order of one metre.

The capacity of a bioretention zone is largely influenced by the permeability of the filter medium. Strict quality control in selection of material and placement is critical to the operation. Over time, the filter medium will become clogged and will need to be replaced.

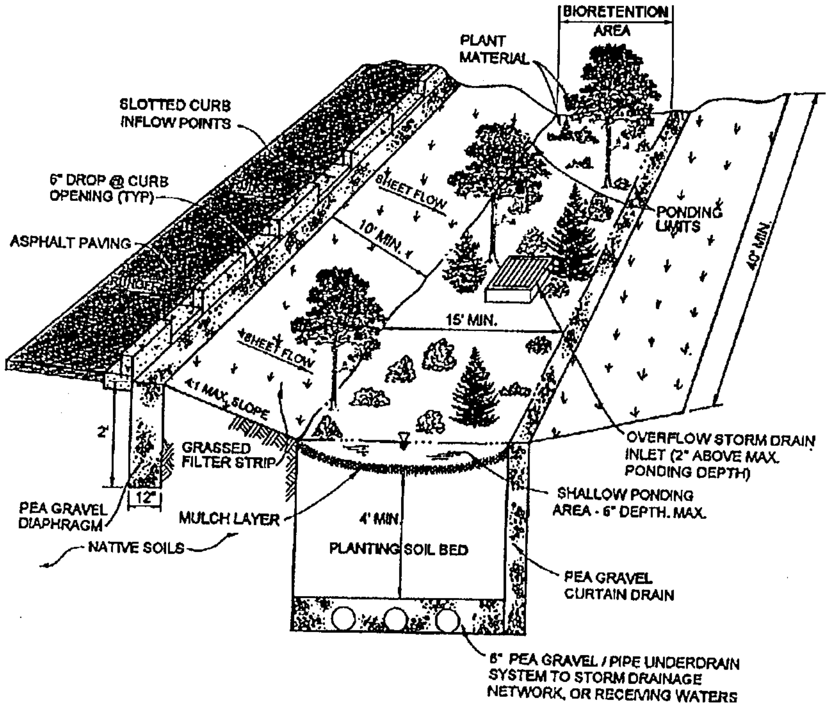


Figure 8.12: Typical infiltration system incorporating grassed buffer strips (after Schueler, 1987)

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 bioretention 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 bioretention 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.

These principles are shown in Figure 8.13 and discussed below.

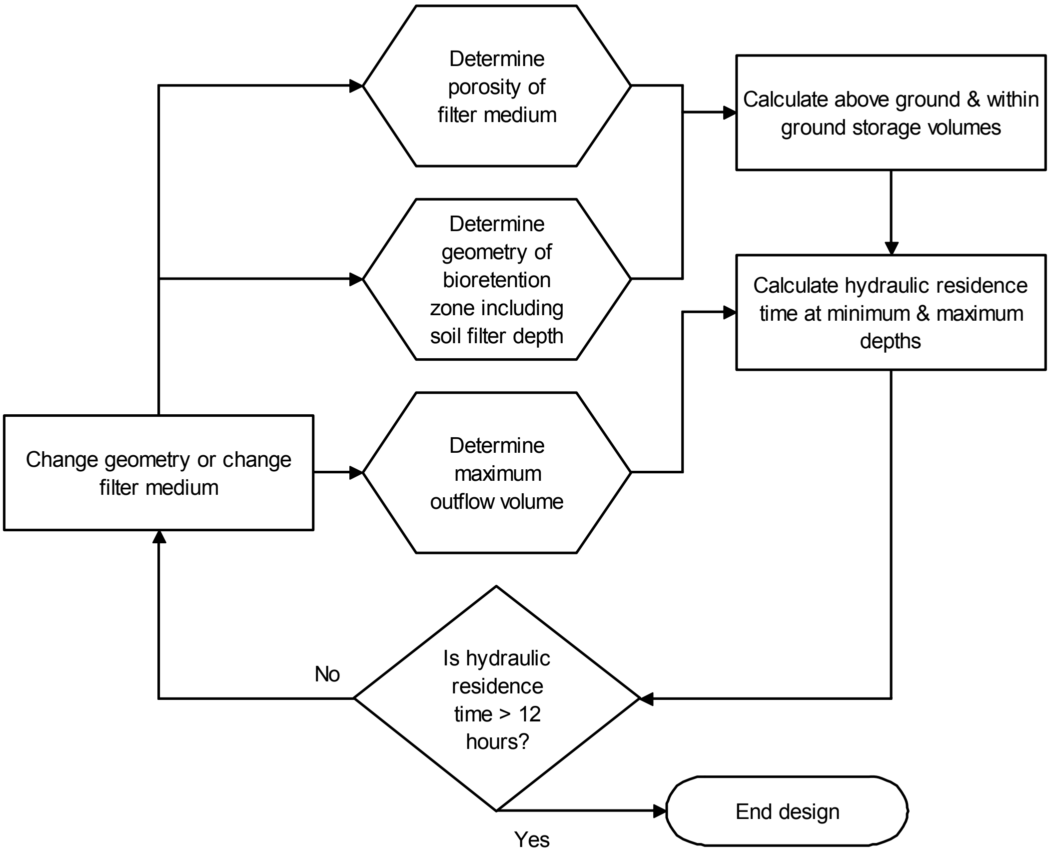


Figure 8.13: Flow chart for the design of a bioretention zone

A. For a bio-retention zone operating at maximum depth of inundation (i.e. h =h max) and at steady flow (Q max).

The hydraulic residence time (T_{HRT}) under steady flow conditions is the ratio of the storage volume and the discharge through the bio-retention system. The storage volume is made up of two components:

- The storage within the infiltration media, computed as the length of the bioretention system (L) multiplied by the base width of the bioretention system (W_{base}) multiplied by the depth of the infiltration media (d) and the porosity of the infiltration media (Φ).
- The above ground storage, computed as the length of the bioretention system (L), the average width of the bioretention system (W_{avg}) and the maximum depth of inundation (h_{max}).

$$T_{HRT} = L\ (\phi.W_{base}.d + W_{avg}.h_{max})/Q_{out} \qquad \text{Equation (8.10)}$$

where

L	length of the bioretention zone (m)
W_{base}	width of the infiltration area (m)
W_{avg}	average width of the ponded cross section above the filter medium (m)
d	depth of the infiltration medium (m)
h_{max}	maximum inundation depth above the soil filter (m)
ϕ	porosity of the infiltration media

Q_{out} maximum outflow from the bioretention zone for the design event (m^3/s)

B. For a bio-retention zone operating at minimum inundation depth (h ~0)

$$T_{HRT} = L(\square.W_{base}.d)/Q_{min} \qquad \text{Equation (8.11)}$$

The surface detention storage (V) is dependent on the maximum infiltration rate (Q_{max}) for the design event. It can be calculated by first assuming simplified triangular inflow and outflow hydrographs as shown in Figure 8.14.

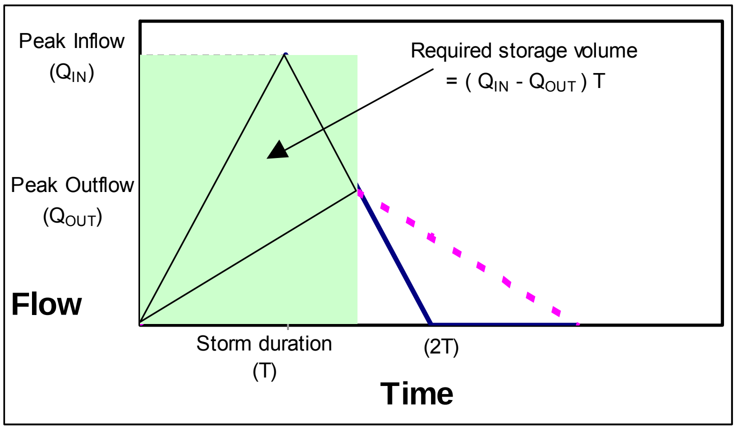


Figure 8.14: Idealised hydrographs (after Wong *et al*., 2001)

Referring to the above diagram, the above ground detention storage volume is expressed as follows:

$$V = L.W_{avg}.h_{max} = (Q_{in} - Q_{out})t_c \qquad \text{Equation (8.12)}$$

where

W_{avg} average width of the ponded cross section above the filter medium (m)

L length of the bioretention zone (m)

h_{max} depth of pondage above the sand filter (m)

Q_{in} maximum inflow to the bioretention zone for the design event (m^3/s)

t_c time of concentration of the catchment (s).

The maximum infiltration rate can be calculated using Darcy's equation:

$$Q_{out} = k \times L \times W_{base} \times (h_{max} + d)/d \qquad \text{Equation (8.13)}$$

where

k permeability of the filter medium (also referred to as the hydraulic conductivity, see Table 8.4 for typical values) (m/s).

Combining the above two equations gives the following relationship between the depth of pondage above ground and the dimensions of the bioretention zone:

$$Lh_{\max} = \frac{Q_{in}}{\left(W_{avg}/T_c + kW_{base}/d\right)}$$ Equation (8.14)

Table 8.4: Typical permeability values (see also Table 2.2)

Soil type	Permeability (m/s)	Soil type	Permeability (m/s)
Gravel	> 10^{-1}	Clean Sand	10^{-1} to 10^{-3}
Sand with clay or silt	10^{-3} to 10^{-5}	Silt	10^{-5} to 10^{-8}

Worked examples can be found in Austroads (2003b).

8.3.10 Detention Basins

Background

The need for detention may arise from an escalation in runoff resulting from catchment development. Detention storage can also be required where the local regulatory authority has specified that flows in receiving waters should not exceed those that existed prior to the development. This may require only a detention tank on an individual building site, or a much larger basin as described below. There are numerous proprietary on-site detention products on the market typically aimed at providing detention facilities for small catchments.

Detention basins, (see Figure 8.15) otherwise known as retarding basins, are designed to reduce the peak discharge from a catchment for events at and above a specified average recurrence interval. The basins are usually dry between storm events, and may be used for other purposes, such as sporting fields where the rise in water level is slow. Where a rapid rise of water level is expected, or batter slopes are steeper than 1 in 6, fencing may be required. (See Section 8.1.3).

Design principles - detention basins

Embankments higher than 3 m should be designed to Australian National Committee on Large Dams (ANCOLD) or the International Committee on Large Dams (ICOLD) guidelines for both the selection of the design event and the structural design.

A typical detention basin is shown on Figure 8.15 and consists of five elements:

- an inlet area consisting of one or more pipes and a trash rack
- a dam embankment that must be compacted and structurally sound and has an internal batter slope of less than 1:6 where the public can gain access
- a low flow drain that permits the basin to be completely emptied
- an outlet tower that controls flows at several levels
- a spillway to cater for excess flow from extreme flood events.

Three hydraulic design criteria apply:

- the basin should control frequently occurring flows
- the basin must achieve the required attenuation of flow in the design event (typically 1 year ARI which caters for over 90% of storm waters)
- the basin must be structurally and hydraulically safe in an extreme (typically 100 year ARI) event.

The features that may be relevant for each storm event include:

- peak outflows
- peak depths
- storage volumes reached in the basin
- duration of ponding
- the attenuation of hydrographs downstream
- the effect on flood levels upstream and downstream.

The site selected should be close to where control of flow is required. The foundations should be suitable for construction of a dam, and the soil should be impermeable in the vicinity of the levees.

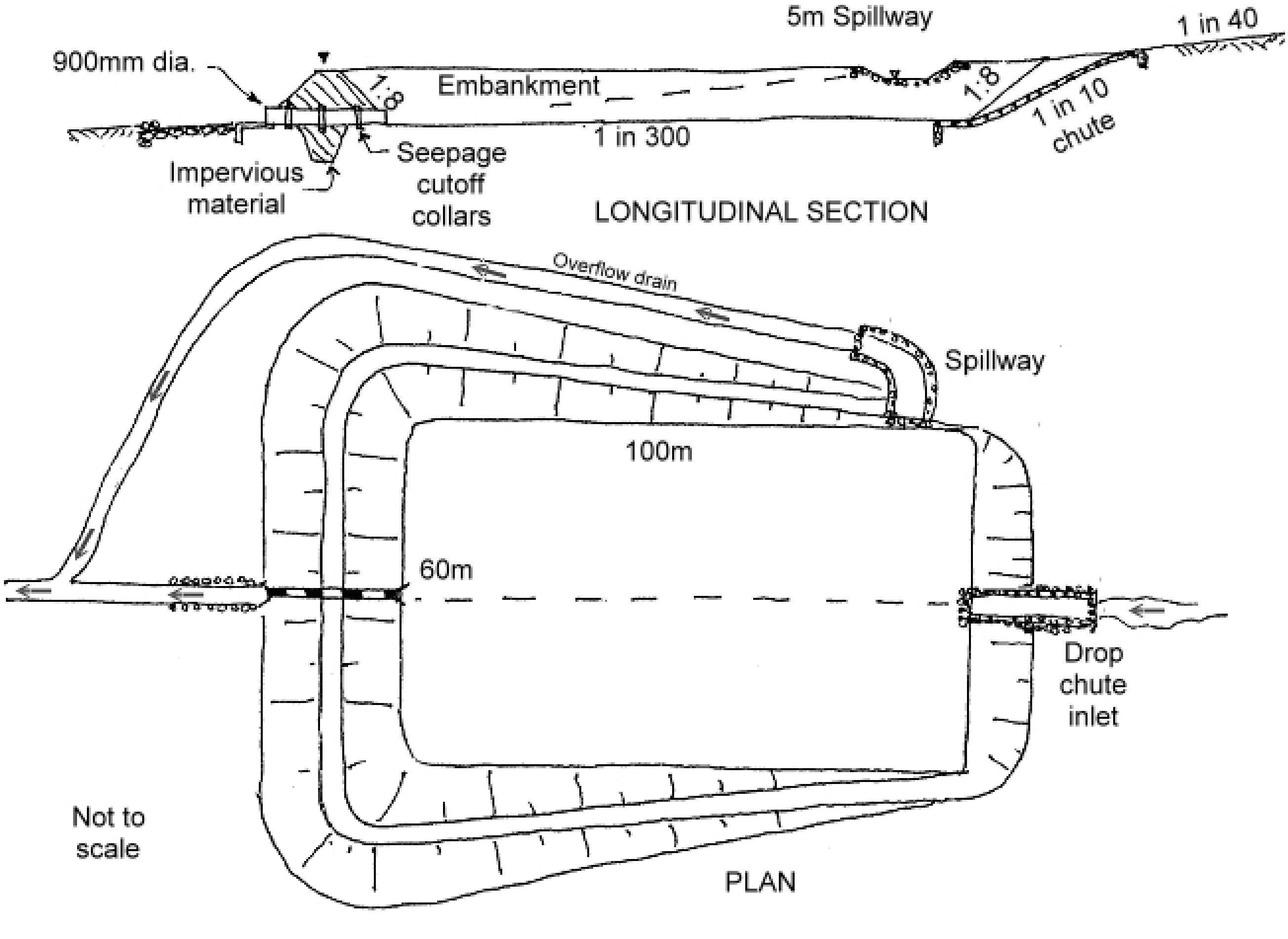


Figure 8.15: Typical detention basin

Design principles – on-site detention

On-site detention (OSD) can be provided as a combination of below ground and above ground storage. Typically, OSD is suitable for small catchments only. The combined above and below ground storage capacity is generally sufficient to cater for storms up to 10 year ARI. The split between above and below ground storage will depend upon where the above ground storage is located and how much inconvenience it will cause.

The capacity of the above ground and below ground storage can be designed to different ARI. The design ARI for the above ground storage component needs to be consistent with the inconvenience level associated with the area to be flooded and the depth of the flooding. For instance, it may required to use a paved pedestrian area as above ground detention storage. The flooding that occurs should be limited in depth for safety reasons, but it might be acceptable for the flooding to occur frequently (i.e. design ARI of less than one year), provided that the inundation period was limited.

Generally the permissible site discharge (PSD) from the OSD will be specified by the local regulatory authority procedure has been included in example F.2 in Appendix F. The PSD can be done at two levels:

Detain the water from a specific site to predevelopment levels

Detain the water from a specific site so that its contribution to the catchment discharge is maintained to pre-development levels.

Where frequent or prolonged inundation is likely in the above ground storage area, steps should be taken to ameliorate problems. Consideration should be given to:

- paving areas for the first 10-20% of the above ground storage capacity
- sub soil drains to reduce extended saturation of the soil
- do not use loose materials (mulch, etc.) which could be washed into the outlet leading to blockages
- designing retaining walls in the vicinity of the above ground storage to cope with hydrostatic loads.

Inundation of parking areas is likely to be very inconvenient to owners and where these areas are used as part of an OSD, the detention time should be limited. The frequency of ponding in parking areas should be limited to once a year. This may mean that about the first 15% of above ground capacity be sited in non-sensitive areas.

Underground storage, where fitted, needs to be maintained and provision at construction should be installed. They should be inspected after heavy rainfall and at three monthly intervals for residential OSDs and two monthly intervals for those installed in commercial areas. Clean out of accumulated trash and sediments will depend partly on the catchment characteristics, but typically in residential areas, this should be scheduled on a six monthly basis and on a four monthly basis in commercial areas.

All discharge outlets should be protected by a trash screen to protect downstream waterways from degradation and to provide stable conditions at the outlet to help achieve predictable discharge rates.

OSD can be installed either in-line or off-line and the underground storage component can consist of concrete chambers or a series of large diameter pipes joined at either end by an inlet and outlet chamber. There should also be provision to bypass flows from storm events greater than the design ARI.

8.3.11 Wetlands

A wetland is a permanent shallow water body with extensive emergent vegetation. 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.

Wetlands may be subdivided into two classes, those that are natural and those that are constructed. Design of constructed wetlands is beyond the scope of these guidelines. A comprehensive guide to layout, sizing and planting may be found in Melbourne Water (2001).

8.4 Treatment Combinations

In the following sections are a number of combinations of treatments to illustrate how various components can be used to improve water quality and/or reduce erosion. The combinations discussed do not represent the full range of options but provide a guide on what can be done. The actual combination of treatments selected will depend upon the site specific circumstances and the desired outcome.

8.4.1 Swale Drain and Buffer Strip

The slope of a buffer strip should not exceed 1 in 6, to avoid excessive formation of rills and should be heavily vegetated with native species. Drainage designers should ensure the uniform distribution of flow over the buffer strip. This can be achieved by provision of contour banks or a distribution channel aligned along the receiving waters, as shown in Figure 8.16. The discharge rate per unit width should be such that flow velocity over the buffer strip is kept below 0.3 m/s for the 100 year ARI event.

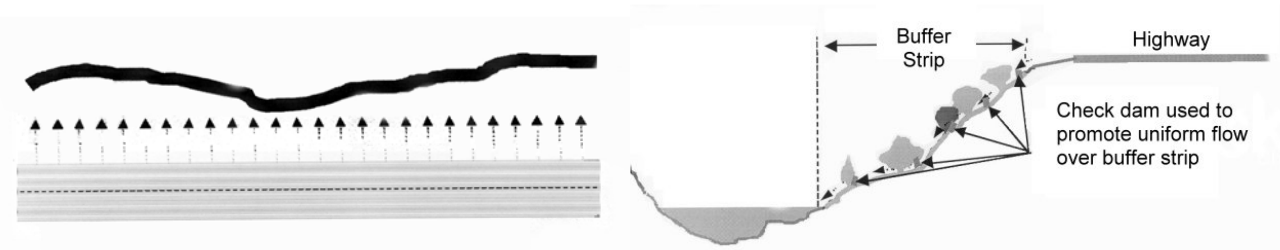


Figure 8.16: Plan and section of a buffer strip runoff management option (after Austroads, 2001)

8.4.2 Swale Drain and Discharge Pits

This option may be more suitable if the terrain between the highway and the receiving waters is too steep for a buffer strip. The road runoff discharges into a swale drain aligned along the highway as shown in Figure 8.17. To ensure low flow rates along the swale drain, pits are located at regular intervals to discharge to the receiving waters through a pipe outlet.

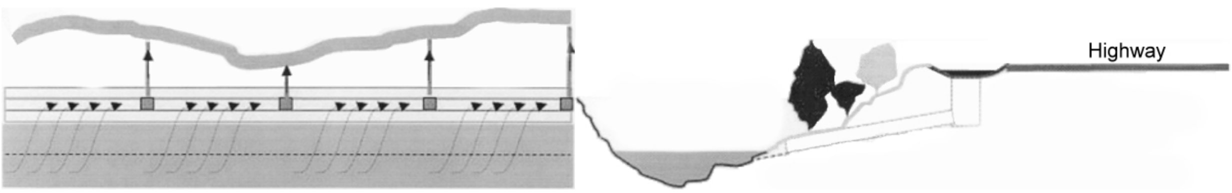


Figure 8.17: Plan and section of a swale drain runoff management option (after Austroads, 2001)

8.4.3 Swale Drain and Underground Pipe

This option would be most appropriate, where the vertical alignment of the road is relatively steep. The system collects the runoff from a section of road, and discharges this into a creek. It involves the use of swale drains to convey runoff from small sections of the road into a more formal drainage system consisting of inlet pits and an underground pipe as shown in Figure 8.18. The hydraulic loading of the swale drain can be maintained at a relatively low level to promote effective pollutant removal by the use of regularly spaced inlet pits. The expected total suspended solids removal efficiency of swale drains is of the order of 60%. Corresponding removal of heavy metals, polycyclic aromatic hydrocarbons and nutrients is expected to be lower, at approximately 20 to 30%.

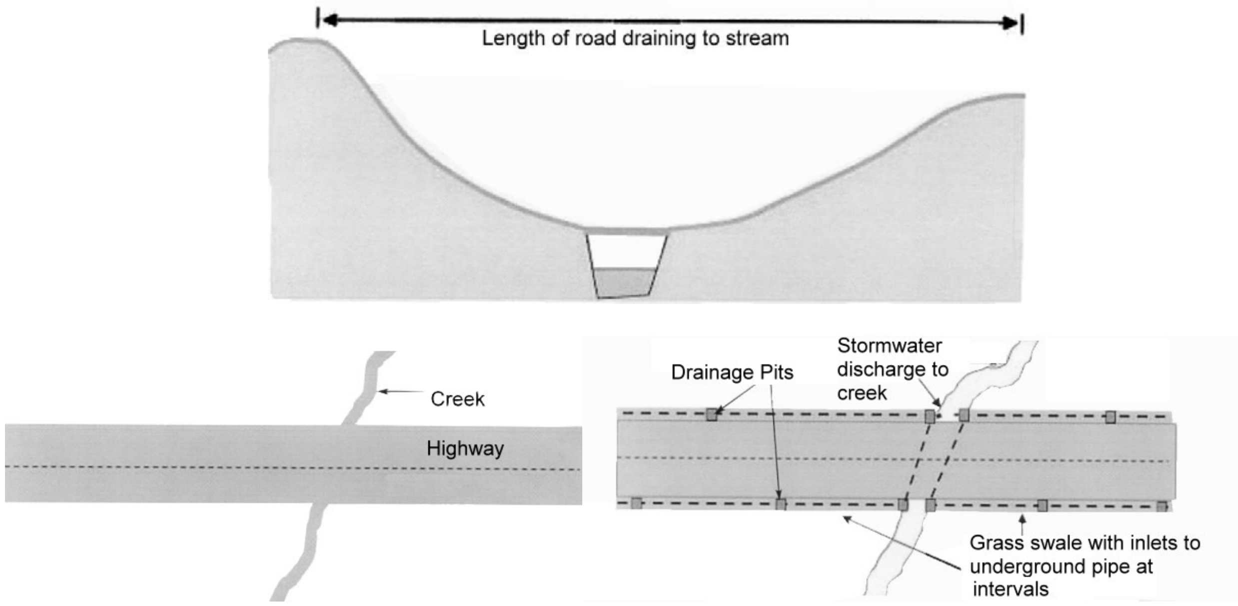


Figure 8.18: Plan and section of a swale drain runoff / drainage pit management option (adapted from Austroads, 2001)

8.4.4 Swale Drain and Bioretention Zone

A bioretention zone may be incorporated into a swale drain upstream of the discharge.

8.4.5 Kerb and Channel with Buffer Strip

Where there are steep slopes, it may be appropriate to collect road runoff by means of kerb and channels, and then discharge to buffer strips. This would avoid uncontrolled overland flow over steep terrain and reduce the risk of erosion.

A buffer strip may be protected from erosion by beaching and geotextile fabric prior to planting, where necessary.

8.4.6 Kerb and Channel & Treatment System

Where there are limited locations for stormwater treatment facilities along the highway, kerb and channel may be required to convey road runoff into an underground pipe system. Stormwater treatment measures such as a cylindrical tank or a continuous deflective flow system may have to be provided if space is insufficient for other measures.

8.4.7 Buffer Strips, Swale Drains and a Constructed Wetland

A complex treatment train may be utilised, depending on availability of a sufficiently large area, the target pollutant and hydraulic loading, and the desired outcomes.

8.5 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 8.5 shows advisable maximum culvert velocities (column 2), 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.

Table 8.5 Desirable maximum flow velocities in culverts or unprotected stream beds (after VicRoads, 1995)

Stream bed soil type	Maximum advisable culvert velocity (m/s)	Maximum allowable stream velocity (m/s)
Silt	1.0 to 1.5	Less than 0.3
Clay, soft	1.0 to 1.5	0.3 to 0.6
Clay, stiff	1.2 to 2.0	1.0 to 1.2
Clay, hard	1.2 to 2.0	1.5 to 2.0
Sand, fine	1.0 to 1.5	Less than 0.3
Sand, coarse	1.0 to 1.5	0.4 to 0.6
Gravel, 6mm	1.0 to 1.5	0.6 to 0.9
Gravel, 25mm	1.2 to 2.0	1.3 to 1.5
Gravel, 100mm	2.5	2.0 to 3.0
Rocks, 150mm	3.5	2.5 to 3.0
Rocks, 300mm	3.5	4.0 to 5.0

By examining Table 8.5, Columns 2 and 3, it can be seen that there will usually be some distance downstream of the culvert where the flow will need to be slowed to the maximum allowable stream velocity. This decelerating zone will require rock beaching to protect the streambed and banks.

Where for some reason, such as a steep culvert slope, the culvert velocity must exceed the value in Table 8.5, Column 2, an energy dissipator will be required (see Section 7.2, Energy Dissipators).

8.5.1 Rock Protection

Rock beaching, see Figure 8.19 (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 Austroads 1994. 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.

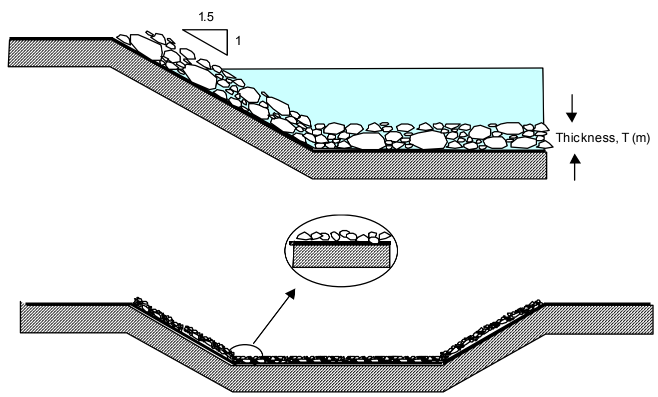


Figure 8.19: Detail of rip rap protection (major channels and culverts) and beaching (channels and culverts less than 6 m² in cross-sectional area)

The size and extent of rip rap for bridges and major culverts are set out in Austroads (1994). However, on culverts where the waterway cross-sectional area is less than 6 m^2, smaller sized stones known as beaching are used to protect the culvert entrance, and the stream bed and banks.

The average size of stone used in beaching may be estimated from:

$$D_{50} = \frac{8550\left(Q \times S_o^{2.16} \times R\right)^{0.4}}{P^{0.4}} \qquad \text{Equation (8.15)}$$

where

D_{50}	50^{th} percentile rock size	(mm)
Q	design discharge	(m^3/s)
S_o	channel bed slope	(m/m)
R	channel hydraulic radius	(m)
P	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 (Vicroads, 1995a) is:

$$X = N\,(Vo - Va) \qquad \text{Equation (8.16)}$$

where

X	length of beaching downstream (m)
N	3 for upstream, 5 for downstream protection
V_o	average outlet velocity (m/s)
V_a	maximum non-scouring velocity from Table 8.6, 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, Va.

8.5.2 Gabions

Gabions are rectangular wire cages used to retain rock fill to serve as retaining structures or surface protection against erosion and scour (see Figure 8.20). 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 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 rock fill 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.

Figure 8.20: Gabions

8.5.3 Rock Mattresses

Rock filled wire baskets with a thickness of 150 to 300mm 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.

8.5.4 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 Section 5.10.

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 taken during construction so that the geotextile is not punctured or torn.

8.5.5 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 (2001).

8.5.6 Energy Dissipators

Where the outflow velocity is greater than can be accommodated by the receiving waters, a form of energy dissipator will be required. As energy dissipators are most often associated with culverts, discussion of energy dissipators has been included in Section 7.2.

CHAPTER 9

9 CONSTRUCTION & MAINTENANCE ISSUES

9.1 Introduction

Achievement of optimum performance of the road and its drainage system depends on good communication between the drainage designers, the constructors, and maintenance personnel. Drainage designers need a sound knowledge of construction and maintenance procedures. On complex drainage projects involving multiple stages of works, the drainage designer should prepare construction drawings that detail construction sequences, construction costs, environmental matters and public safety.

Construction personnel should be encouraged to inform the drainage designer of any design-related difficulties or on-site changes and to provide suggestions to improve future designs.

Maintenance personnel need a working knowledge of the road authority's policies and design standards, and should implement these in their work practices.

It is important that the drainage designer consider costs over the design life of the facility. Construction costs are related to materials, machinery and labour. Maintenance methods and costs should be considered, since these can form a substantial part of the whole of life costs in a drainage scheme. Drainage devices should not be labourious to maintain nor difficult to replace. Drainage designers must achieve a proper balance between initial and future (recurrent) costs. Drainage facilities should be designed and constructed recognising that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users. Access tracks should be clearly identified and delineated.

The timing of drainage works during construction is significant. It is important that drainage works should be constructed no later than the formation earthworks. Should drainage construction lag behind the earthworks, inadequate compaction could result from working on undrained soils. Also, rain could damage the work where there is insufficient provision for permanent and temporary drainage of the formation.

The construction of pavements is discussed in this section because pavement failures can result from the ingress of stormwater and poor pavement drainage. Other drainage elements considered include culverts, underground drainage systems, subsurface drains and water quality control devices.

9.2 Pavement Construction

9.2.1 Pavement Failure Mechanisms

Premature failure of a road structure is generally caused by poor construction, incorrect material usage (including incorrect design assumptions), and moisture conditions not originally envisaged.

The prevention of premature failure is largely dependent upon the quality of construction management and subsurface drainage design. If precautions are not taken then one of the following types of failure may occur.

- Loss of subgrade strength and consequent rutting due to the actual moisture content being greater than the laboratory estimated moisture content.

- Overload of the subgrade due to hydrostatic transmission of loads through a saturated pavement. This will occur if the pavement has a low permeability resulting in retention of water that enters through the wearing course.
- Surface potholes may form by the retention of water in the vicinity of an existing defect such as a poor joint or a crack in the wearing course, even if the pavement has good strength characteristics.
- Deep potholes may form in saturated pavements. Surface wheel impact loads transmitted directly through incompressible water are transferred to the pavement material and often cause 'blowouts'. This type of failure usually occurs when permeable pavement materials are used.

9.2.2 Grading and Compaction

It is important that the subgrade and the pavement materials are compacted to the specified densities, to limit the void space available for storage of water that may cause failure. Further, surfaces should be carefully graded and trimmed so as to avoid ponding. The invert of subsurface drainage trenches should be smoothly graded, so that moisture may flow away from the pavement, even when its level is below the slotted drainage pipe.

Construction equipment that will alter the properties of the materials from those specified should not be used. For example, compaction equipment should not cause mechanical breakdown of the pavement materials or the subsurface drainage materials to such an extent that their permeability or capillary characteristics are altered.

9.2.3 Culverts

Culverts are constructed from a variety of materials. Corrugated steel, precast concrete are common as well as polyethylene. Labour intensive construction, such as brick culverts, is not common.

The selection of construction materials will largely be based on engineering judgement and cost, however fire damage should also be considered, particularly when considering polyethylene.

Location and alignment

The location of culverts to minimise stream erosion and sedimentation is covered in Chapter 7 – Culverts & energy dissipators. For convenience of construction and maintenance, culverts should be straight in both the horizontal and vertical planes. Significant differences in levels at inlet and outlet should be accommodated in a structure such as a drop inlet, a drop stilling basin, or some other form of energy dissipator (see Chapter 7 – Culverts & energy dissipators).

Culvert Failures

Common failures associated with culverts are:

- differential movement between adjacent culvert segments as a result of poorly constructed foundations
- blockage, by accumulation of debris or sedimentation
- rotation of precast headwalls away from the pipes if not properly installed
- erosion at culvert outlets resulting in undermining of the outlet structure.

Culverts are particularly susceptible to sedimentation when earthworks are exposed during construction. Soil loss on steep batters is a contributory factor. A sedimentation control program during construction is essential, and all culverts should be cleaned out before the road is opened to traffic.

All culvert aprons should include a cut-off wall below the stream bed level to inhibit any headward scour and undermining.

During construction, temporary measures such as stream bank protection by sandbags may be required until the permanent structure (such as an energy dissipator) or erosion protection is in place.

The extent of beaching, upstream and downstream, should be carefully determined on site (see Chapter 7 – Culverts & energy dissipators). Inadequately constructed foundations for beaching or insufficient length of beaching at culverts and floodways may result in the culvert and/or the adjacent road being washed away.

The minimum cover over a pipe during construction depends on the type of machinery traversing it. One option is to create a ramp over the culvert to achieve the cover required for the construction traffic. Another option when fall is not a constraint, is to design the pipe with more cover below subgrade level or increase the strength of the pipe.

Although box culverts are designed to carry traffic loading, they may not be adequate to carry heavy construction equipment. Construction traffic may have to be provided with a side track around the culvert site.

9.2.4 Backfilling and Compaction

The required strength of a pipe depends on the dead and live loads, and on the support provided to the pipe within the trench. Several bedding conditions are allowed under AS 3725, and the bedding and backfill materials should conform to the specified grading and be compacted to the required density. This is particularly important when side-support conditions are specified (HS1, HS2 or HS3). Not only should the materials in the different zones around the pipe be thoroughly compacted, but also the width of the support zone should be such that the natural material can support the shear force transmitted to it.

Flexible pipes, such as corrugated polyethylene or corrugated steel require firm support up to and above the obvert for their structural integrity. They depend upon the interactive transfer of loads into the surrounding materials. Otherwise, failure of the pipe by buckling and settlement of the filling above the pipe will occur. Proper placement and compaction of the bedding material during construction is vital.

9.2.5 Underground Drainage Systems

For convenience of construction and maintenance, pieplines should be straight in both the horizontal and vertical planes. Significant differences in levels at inlet and outlet should be accommodated in a structure, such as a drop pit. In addition, attention must be paid to proper filling and sealing of pipe joints. The joints should not allow water to leak from the pipe into the surrounding soil. This could cause a general weakening and subsidence of the pipe or flow of water outside the pipe, known as ‘piping’. Where trees or other vegetation are present, open joints may allow displacement of the pipe and blocking by roots. Joints should be finished smoothly inside the pipe to allow efficient hydraulic operation, especially where there are discontinuities due the use of steel reinforced concrete pipes of the same nominal diameter but of different strength classes.

Where punch-out pits are used, any exposed reinforcement should be trimmed and waterproofed and the space between the punched hole and the pipe should be carefully filled.

Where pipes regularly operate under pressure or are placed within deep fill where differential settlement could occur, rubber ring joints should be used.

Pipes on steep batters will require temporary anchorage until the pipeline is assembled and the backfilling is in place. On steep slopes, anchorage blocks may be required as part of the completed pipeline.

During construction, sediment traps should surround pit tops in order to prevent blockage of the drainage system by building materials, litter or sediment. Pits should be cleared and the pipe system flushed clear of sediment at the end of the construction contract. When installing deep pits or manholes, adequate provisions for future access need to be incorporated e.g. ladders, etc.

Where culverts are used for cattle, sheep, etc., allowance for vehicles such as tractors needs to be considered.

9.2.6 Subsurface Drains

Compaction

Construction equipment that will alter the properties of the materials from those specified should not be used. For example, a vibrating roller should compact the material to the specified density but should not cause mechanical breakdown of the pavement or the subsurface drainage materials to such an extent that their permeability or capillary characteristics are altered.

Clean Interfaces

If the interface between various materials is a critical factor influencing moisture movement, the constructor should ensure that no contamination of the interface occurs which could impede the flow of water into the subsurface drainage system.

Flushing

During construction, flooding of the filter material and the vibration of mechanical compaction may result in a large volume of fine material entering the pipe. This material should be flushed from the pipe when the filter backfill compaction is completed. This also provides a check that the pipe has not been crushed during compaction. The system should be flushed until it discharges clean water, then immediately should be made rodent proof.

Drain Markers

Where possible, markers should be placed near the start and finish of sub-surface drains to facilitate future location for maintenance purposes.

9.2.7 Temporary Works

Temporary Drainage

Temporary traffic lanes and detours may be required during construction. The design of these should include provision for temporary drainage. It is preferable for the temporary road drainage system to be separate from any natural drainage systems as an environmental protection condition. This may require construction of a diversion drain around the site as well as temporary drains around the works themselves. The diversion drain should be protected from erosion. The runoff from the construction site should pass through some water quality treatment device before discharge to the natural system.

As earthworks progress, all surfaces should be shaped to be free draining in order to prevent soft spots due to ponding of stormwater.

Where seepage occurs, or earthworks will pass through swampy ground or there is daily rainfall, the construction of most drains should be carried out prior to the earthworks. Temporary construction drains may need to be excavated several months prior to the main earthworks to allow stable, acceptable ground moisture conditions to be achieved.

Erosion and Sediment Control

Considerable temporary work is required to control stormwater on site and the resultant erosion and sedimentation. In many States, the constructor is required to have a written Environmental Management Plan as part of the contract. There are many useful manuals on this topic, such as DMR NSW (1991), EPA Victoria (1991). DLWC NSW (2000) has produced a conveniently sized illustrated booklet for field personnel.

The quantity of erosion depends among other things on the area cleared of vegetation, the steepness of the slope, the duration of exposure and the intensity of rainfall. Contractors should plan the works such that the exposed area is kept to a practical minimum. As soon as a section of cut to fill earthworks is completed, the surface should be protected. If the final topsoil cannot be placed and seeded, methods such as mulching or hydroseeding should be employed to provide temporary cover.

Flow across broad exposed areas of earthworks should be controlled by contour banks or silt fences. In recent times, silt fences commonly consist of geotextile barriers rather than straw bales. Similarly, flow in open unlined drains should be interrupted by silt fence, taking care that the water cannot bypass the edges of the fence. In steeper drains, temporary drop structures should be provided as required, using materials such as sandbags or gabions.

Before discharge into a natural stream or gully, stormwater from the construction site should be cleaned. The minimum treatment would be a gabion-type sediment trap, but larger quantities of water may require a sediment basin. More extensive treatment may be necessary, depending upon the water quality standards set by the relevant authority. In some cases, sheet flow through a heavily vegetated strip or a filter trench may be acceptable, but in other cases polluted flows may have to go through wetlands and aeration ponds. Regulatory authorities or roading organisations have produced useful manuals on this topic such as [refer to some of the relevant publications listed in the references section]. The design of permanent water quality treatment devices is covered in Chapter 8 – Water quality & erosion.

9.3 Maintenance

9.3.1 Introduction

The primary objective of maintenance is to preserve or to repair works in order to prevent the deterioration of quality or efficiency below the levels achieved immediately after construction.

In order to realise this objective, maintenance tasks can be divided into three types:

Routine maintenance, carried out as part of regular inspections along the route.

Specific or periodic maintenance, such as cleaning of sediment basins.

Restoration of facilities damaged by events such as flood or fire.

Clear documentation of maintenance objectives, procedures and actions allow potential and actual events to be managed and may also assist the road authority to demonstrate due diligence if exposed to litigation.

A secondary objective is to gain the best return from limited resources. Managers decide priorities and work systematically toward these, determined generally on the basis of greatest need and other clearly defined policies and objectives of the road authority.

Common Problems

Common problems encountered during drainage maintenance include the accumulation of debris, sedimentation, erosion, scour, piping, carriageway and embankment settlement, and structural damage.

Open channels should be designed so that control of grass and weed growth can be achieved without recourse to specialised equipment. Drains with deeper cross-sections should be maintained using backhoes, front-end loaders and trucks. Where access is restricted, maintenance of open drains can be minimised by lining them with rock beaching or an equivalent.

Marker System

Drainage outlets and inlets may become overgrown by weeds or obscured by significant siltation. This can make them difficult to locate for maintenance. Guide posts should identify culverts and subsurface drain outlets or some similar markers placed so as to minimise interference with other maintenance activities such as shoulder grading or mowing. Guide posts need to conform to AS 1742. Marker posts should be designed and erected in such a way as to ensure that the only maintenance they will require is repainting.

9.3.2 Inventory

Most road authorities now use Pavement Management Systems. This concept is more frequently being extended to include the management of the road drainage system and subsurface drains. Highway condition survey data should be systematically recorded on forms or by means of hand-held computers. The information should be collated and maintenance needs identified in priority order for scheduled treatment.

9.3.3 Schedules

Schedules for drainage maintenance will usually be part of the overall plan for road and roadside management. Indicative maintenance frequencies of a sub-surface system are in Table 9.1.

Energy dissipaters should be inspected at least every 6 months and after each significant storm event. Debris, litter and sediment should be removed to ensure that the device operates as designed.

Sedimentation basins, whether stand-alone or part of a water quality treatment train, will require the regular removal of accumulated materials. The frequency of cleaning depends, among other things, on the catchment area and the amount of construction or development taking place within it. After a period of monitoring, it will be possible to predict the cleaning frequency for each basin and place it on a regular maintenance schedule.

In addition to short term activities such as collection of litter, and clearing of small diameter outlets, wetlands may require pruning or removal of vegetation every 5 to 10 years in order to retain the ability to adsorb nutrients from stormwater.

Table 9.1: Frequency of subsurface drain maintenance

Activity		Typical preceding event (i.e. the event that initiates the need for maintenance.)	Approx. frequency of activity
Routine	Corrective		
Check Rodent Protection		Scheduled task	as needed
Paint markers		Scheduled task	as needed
Flushout		12 months after construction	as needed
Inspect outlets		Storm, prolonged rainfall	as needed
Observe discharge		Prolonged rainfall	as needed
	Flushout	Discoloured discharge, reduced performance	If corrective action is required more often than 3-6 months, then the cause of the problem should be investigated
	Clean outlets	Inspection discovered blocked outlet	If corrective action is required more often than 3-6 months, then the cause of the problem should be investigated
Monitor roadway performance			Efficiency of drainage system in doubt if performance is poor

9.3.4 Records

For records to be of any practical value they should be simple to compile and use. Each drainage component should be uniquely identified. The record for each component should include at least a locality plan and the inventory report. For larger structures, such as major culverts and energy dissipators, a copy of the structural general arrangement plan should be kept in order to assist assessment of the effects if there is storm damage.

In addition to the inspection reports and site notes, a drainage component file should include any monitoring results, a record of non-conformances and corrective action, meeting minutes, and correspondence. The costs of maintenance also should be kept. If a component is very costly to maintain, then possible improvements to the system should be investigated.

The keeping of accurate records establishes a data bank of drainage performance throughout the life of the road and is an essential element of a successful maintenance management system.

9.3.5 Improving Existing Systems

If a drainage system is inefficient, costly to maintain, lacks capacity or does not meet expectations for managing environmental effects, then possible improvements to the system should be investigated. Minor works such as relocation of outlets or installation of additional inlets may be considered as maintenance activities.

Even complete reconstruction of a subsurface drainage system could be considered as a road maintenance activity, if it is anticipated that by so doing the major asset (the road structure) will maintain a satisfactory level of performance.

Improvements may include:

- raising short lengths of roadway above flood level
- increasing pavement crossfall
- installing additional culverts
- installing additional pits
- installing additional subsurface drains
- installing mitre drains
- installing additional catch drains
- widening table drains
- flattening batter slopes
- extending rock beaching
- provision of top soil and planting unstable areas and / or
- installing detention / retention / sedimentation / infiltration facilities.

9.3.6 Safety Measures

Drainage facilities should be designed and constructed recognising that periodic inspection and repair will be required and provide for the safety of maintenance personnel as well as for road users.

Culvert headwalls (see Section 7.1.3) may be obscured by long grass or reeds and present a potential hazard to mower drivers. Where possible culvert headwalls should be flush with the adjacent batter to reduce the likelihood of causing errant vehicles to become airborne.

Other safety measures include:

- access tracks should be clearly identified and delineated
- at each permanent sedimentation basin, the access track should ideally be paved to ensure safe all year round access
- investigation of potentially cracked or failed underground pipes should be carried out using a remote television camera to reduce the risk to inspection personnel.

CHAPTER 10

10 RISK ASSESSMENT

10.1 Introduction

This chapter gives a brief overview of risk assessment and how it may be applied to stormwater management. Readers are advised to seek out more detailed information for specific circumstances.

The values given in Tables 4.2 and 6.3 for the ARI design standards should be treated with caution. They represent design standards that may not take into account the individual circumstances. Increased flows in natural watercourses from stormwater runoff can have devastating effects on downstream flora, fauna and local amenity. Increasing water levels upstream as a result of a drainage scheme (afflux) can have a severe effect on adjacent properties. Where necessary the potential problems of downstream alterations to flow behaviour and afflux should be considered during the design of drainage schemes.

For example, consider a simple culvert planned for a minor road crossing. Normally this would be a minimal design task, but immediately upstream of this particular site is a large nursing home. The impact of the proposal on upstream water levels therefore becomes a matter of concern and the effort expended on hydraulic modelling must be commensurate with the risk associated with the consequences of upstream flooding.

Risk assessment goes beyond this simple example and must consider the risks and associated costs or benefits involved in providing a level of service to the community. Not every drainage scheme will require a risk management study as the resources and expertise required can be high.

Two methods are offered and comprise:

- least cost analysis; and
- risk factor analysis.

In addition, there are some concluding remarks on adopting a regional approach to stormwater management.

10.2 Least Cost Analysis

One approach is to undertake a least cost analysis. In this approach, a range of floods is examined and the predicted outcomes are related to costs to the community.

Increasing the level of flood immunity increases initial construction costs, and often requires higher ongoing maintenance. A high level of flood immunity reduces the costs to the community due to surcharging of the facility (i.e. flood volume exceeds design capacity) due to low frequency of exceedence and lower volumes exceeding capacity. Designs to cater for storms of lower ARI (more frequent flooding) decreases the construction costs but increases the cost to the community due to capacity being exceeded more often and by being exceeded by greater volumes. The total cost is then correlated to the various storm ARIs and the least total cost provides the design ARI for the facility. The idealised concept is shown in Figure 10.1.

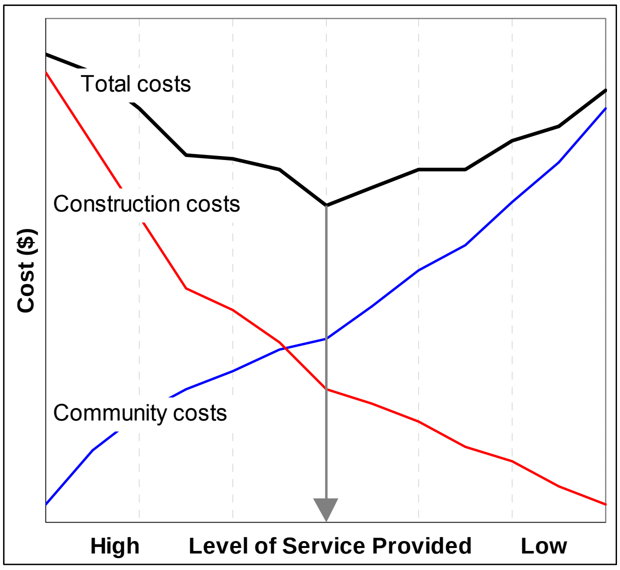


Figure 10.1: Idealised least cost analysis (based on RTA, 1999)

One of the main problems in use of the least cost analysis is quantifying the costs to the community of flooding. The community costs should reflect the social, commercial and environmental costs incurred.

10.3 Risk Factor Analysis

Risk factor analysis involves setting appropriate assessment criteria, definition of the key elements and the risks associated with each of the elements. A number of key elements are identified in Table 10.1. However, due to the widely varying circumstances involved in provision of drainage schemes, it is not possible to provide a comprehensive list for all situations and eventualities. The elements in Table 10.1 should be viewed as typical of those that may be applied, and not considered an exhaustive list.

The risk assessment assigns factors associated with the likelihood of an event taking place and the consequences of that event for each of the key elements. The likelihood factor (LF) is often adopted as 1/ARI, i.e., for a design using an ARI of 20 years the likelihood factor would be 1/20 = 0.05. The consequences for each element would then be considered and assigned a factor. A typical range of consequence factors is shown in Table 10.2.

A risk factor can then be calculated for each key element using Equation (10.1). The risk factors for all key elements are then summed and plotted against the costs in providing the infrastructure to each level of ARI (likelihood level). The costs and the risk factors for each ARI scenario are summed and expressed as the total for all scenarios examined. The lowest point of the 'total costs' line is when the construction costs are balanced against the risk. The plot is similar to a least cost analysis except the community cost line is replaced with a risk factor line (see Figure 10.2).

$$\mathbf{RF = LF + CF - (LF \times CF)} \qquad \textbf{Equation (10.1)}$$

where

CF consequence factor

LF likelihood factor

RF Risk factor.

Table 10.1: Typical risk management considerations

Assessment Criteria	Key Elements	Risks
Duration of road closure Frequency of road closure Etc.	Safety	Injury to users, infrastructure elements damaged to unsafe levels
	Economic costs due to disruption	Increased user costs, damage to vehicles
	Social costs due to disruption	Community concern
	Restoration costs to drainage infrastructure	Repair or replacement costs
	Flood damage costs to others	Inundation of adjacent land
	Cost to environment	Loss of amenity, flora or fauna. Increased pollutant load or turbidity
	Impact on downstream and upstream water ways	Alteration to stream flows, siltation, scour
	Construction difficulties	Delays to work programs
	Maintenance	Higher maintenance costs
	Etc.	Etc.

Table 10.2: Consequence factors (CF)

Consequence Factor	Critical	Major	Serious	Moderate	Minor	Negligible
	0.9	0.7	0.5	0.3	0.1	0.0

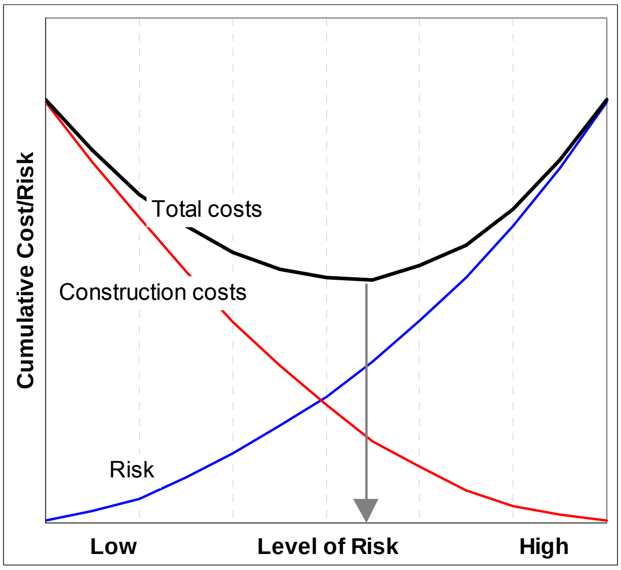


Figure 10.2: Idealised risk factor concept

10.4 Regional Approach

It can be advantageous to consider individual drainage schemes in relation to a regional drainage plan. A thorough analysis of the regional flood estimates can be calculated on a one-off basis and reviewed periodically to ensure that the estimates have not varied. This will reduce the need to fully examine the flood estimates on a project by project basis. The regional approach permits:

- efficient use of resources in that only a single major hydrologic analysis is required per region
- identifying areas of major risk; and provide a framework in which minor drainage schemes contribute towards an overall objective.

CHAPTER 11

11 TERMINOLOGY, SYMBOLS & ABBREVIATIONS

Introduction

The terms, symbols and abbreviations used throughout this document are those in common usage in Australia and New Zealand. The terms are consistent with the Australian Standard AS1348-2002 'Road and traffic engineering – glossary of terms' and the Institution of Engineers, Australia 'Australian rainfall and runoff – A guide to flood estimation'.

Terminology and Abbreviations

Term	Definition
Annual average daily traffic (AADT)	The total yearly two-way traffic volume divided by 365, expressed as vehicles per day.
Annual exceedance probability (AEP)	AEP is the probability of an event exceeding a given discharge within a period of one year. It is expressed as '1 in Y', and is most applicable to major flood studies.
Average area slope	This is determined by fitting a linear regression equation to the paired data representing the height above the outlet (in metres) and the distance from the outlet (in kilometres). The regression analysis should be forced to go through zero. Also referred to equal area slope.
Average recurrence interval (ARI)	ARI is the average or expected value of the period between exceedance of a given discharge value. The period is a random variate.
Biological Oxygen Demand (BOD)	A measure of the oxygen required by micro-organisms whilst breaking down organic matter.
Consequence factor (CF)	A weighting value describing the severity of an event.
Continuous deflective separator (CDS)	A gross pollutant trap designed for larger flows that successfully traps vegetative matter and coarse sediment, while the system ensures that the separating screen remains unblocked.
Design flood	A probabilistic or statistical estimate of flood data used in design.
Drainage system	A system of pipes and drains/channels, either natural or artificial, used to intercept and remove surface or subsurface water.
Dual-channelled	Roads with normal two way crossfall, i.e. flows accumulate in both gutters.
EOS	Equivalent size opening (of fabric).
Equal area slope	See average area slope.
Gauging station	A monitoring site that records stream levels against time.
Gross pollutant trap (GPT)	A permanent structure designed to collect litter by means of a trash rack, and to settle coarse sediment in a sediment basin.
Hydraulic grade line (hydraulic gradient) (HGL)	A line representing the pressure head along a pipeline, corresponding to the effective water surface elevation in the piped portions of a stormwater drainage system.
Hydraulic loading	The ratio of the flow rate (m^3/yr) to the surface area (m^2) expressed in metres per year.
Hydraulic residence time (T_{HRT})	The time that storm flows are processed through a bioretention zone.
Hydrograph	A graph of stream height or volume flow rate past a specific point against time.
IFD	See intensity-frequency-duration.
Intensity-frequency-duration (IFD) design rainfall curves	A plot of annual maximum rainfall data for a duration of 5 minutes to 72 hours with ARIs ranging from 1 to 100 years.
Likelihood factor (LF)	A weighting factor describing the likelihood of an event taking place, usually taken as 1/ARI for hydrological analysis.
Major drainage system	An arrangement of pavements, roadway reserves, open space floodway channels, detention basins, lagoons, etc. planned to convey to a disposal outlet, a design rare flood of specified frequency (Australian practice, generally, recognises floods of average recurrence interval ARI = 50 years or 100 years or more as 'rare' flood events).

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Term	Definition
Master drainage plan	Overall drainage management plan for an entire catchment area.
Minor drainage system	An arrangement of soakage wells, kerbs, gutters, roadside channels, swales, sumps, inlets and underground pipes and junction pits designed to fully contain and convey to disposal a design minor stormwater flow of specified frequency.
POA	Percent open area (of geofabrics).
Risk factor (RF)	An estimate of risk assigned to an individual site or activity.
Return period	See average recurrence interval.
Run-off (1)	A general term for water (normally from rainfall) flowing across the surface of the ground.
Run-off (2)	That part of the water precipitated onto a catchment area, which flows as surface discharge from the catchments area past a specified point.
Single-channelled	Roads with a one-way crossfall.
Surcharging	In hydrological terms, it applies to exceedance of the capacity of a hydraulic system.
Time of concentration (t_c)	The shortest time necessary for all points on a catchment area to contribute simultaneously to run-off past a specified point.
Texture depth (TXD)	The macro texture evident on the surface of the road.
Total Phosphorus (TP)	A measure of the total phosphate present (in water). An indicator of the nutrient level.
Total Nitrogen (TN)	A measure of the total nitrogen present (in water). An indicator of the nutrient level.

Symbols

Term	Description	Units
0m	Diameter of holes in a fabric.	mm
090m	Diameter of hole that is greater than 90% of the holes in the fabric.	mm
A	Cross-sectional area of flow.	m^2
A	Catchment area.	ha, km^2
A_b	Basin surface area.	m^2
A_i	Impervious catchment area.	ha
A_p	Pervious catchment area.	ha
A_s	Predicted average annual soil loss per acre unit area.	t/ha
C	An empirical constant that depends on the shape of the grains and varies from 0.1 to 0.5 cm^2 (For perfect spheres = 0.1 cm^2).	cm^2
C	Runoff coefficient.	Dimensionless
C	Cover management factor allows for erosion protection practices such as provision of vegetation or other erosion protective measures (e.g. mulching).	Dimensionless
C_i	Runoff coefficient for impervious area.	Dimensionless
C_i	Concentration of suspended solids in the inflow.	mg/l
Ch	Chezy's coefficient.	Dimensionless
C_o	Specified concentration of suspended solids in the outflow.	mg/l
C_p	Runoff coefficient for pervious area.	Dimensionless
C_W	Weighted runoff coefficient.	Dimensionless
C_w	Weir coefficient, usually about 1.6.	Dimensionless

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Term	Description	Units
CY	Runoff coefficient for a storm with an ARI of Y years.	Dimensionless
d	Depth.	m
d_1	Depth prior to hydraulic jump.	m
d_2	Depth after hydraulic jump.	m
d_c	Critical depth.	m
d_e	Equivalent depth.	m
d_g	Depth of flow at the kerb.	m
d_p	Partial depth.	m
D	Pipe diameter or culvert depth.	mm
D_u	Diameter of upstream (inlet) pipe.	mm
Ds	Diameter of particles in a soil adjacent to the fabric.	mm
D_{10}	Allen Hazen's effective grain size based on the sieve opening in cm that 10% of the material passes.	cm
D_{50}	Mid size of rock beaching.	mm
D85s	Soil particle diameter that is greater than 85% of the diameters of the soil.	mm
Do		
Dt		
e	Void ratio.	Dimensionless
e	Relative pipe roughness (for use in determining Reynolds's Number).	Dimensionless
E	Required efficiency (removal of pollutant load).	%
f	Friction factor for flow in pipes (usually determined from Moody diagrams).	Dimensionless
f	Fraction of impervious area in the catchment.	%
F_r	Froude number.	Dimensionless
F_Y	Frequency factor for converting runoff coefficients for use in Rational Method.	Dimensionless
g	Gravitational acceleration (9.81 m/s^2).	m/s^2
h	Depth of flow over spillway.	m
h_{max}	Maximum inundation depth above the soil filter.	m
H	Head of water.	m
Hp	Head loss through pit.	m
HW	Head water depth upstream of feature.	m
${}^{Y}I_{t_c}$	Average rainfall intensity for a storm of t_c hours duration and an ARI of Y years.	mm/h
I_{max}	Maximum inflow to a bioretention zone for a design event.	m^3/s
j	Geometrical factor (used in Schilfgaarde's method).	Dimensionless
k	Saturated permeability.	m/s
k	Permeability of the filter medium (also referred to as the hydraulic conductivity.	m/s
k	Roughness value of pipes.	Dimensionless
K	Soil erodibility factor relating particle size, soil structure, organic content and permeability.	Dimensionless
K_p	Coefficient dependent on the inlet and outlet diameters, the inlet and outlet discharges, and the pit geometry.	Dimensionless

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Term	Description	Units
L	Length (between points, of beaching, pipe or culvert).	m
LS	Slope length factor slope multiplied by the steepness factor.	Dimensionless
Lw	Length of weir.	m
n	Manning's roughness coefficient.	Dimensionless
n	ARI recurrence period for minor drainage scheme	years
N	ARI recurrence period for major drainage scheme	years
q	Discharge per unit width.	L/s.m or m^3/s.m
P	Wetted perimeter.	m
P	Support practice factor compensates for surface treatment practices.	Dimensionless
Q	Maximum discharge.	L/s or m^3/s
Q_c	Capacity flow.	L/s or m^3/s
Q_f	Full area discharge by the Rational Method.	L/s or m^3/s
Q_g	Flow from above the pit water level.	L/s or m^3/s
Q_{gap}	Gap flow.	L/s or m^3/s
Q_L	Flow from lateral pipes.	L/s or m^3/s
Q_o	Flow out of pit.	L/s or m^3/s
Q_p	Part area discharge by the Rational Method.	L/s or m^3/s
Q_{sc}	Corrected storage capacity flow.	L/s or m^3/s
Qu	Flow from upstream pipe.	L/s or m^3/s
Q_Y	Peak discharge for a storm with an ARI of Y years.	L/s or m^3/s
Q_{100}	Design discharge for a storm with an ARI of 100 years.	m^3/s
R	Hydraulic radius	m
R	Rainfall runoff erosivity factor.	
S	Longitudinal slope.	m/m
S	Basin size factor used in Figure 8.8 and equation 8.8.	Dimensionless
S_e	Average area slope.	m/km, %
S_c	Slope of culvert.	m/m
t	Critical storm duration	time
t_c	Time of concentration .	time
t_i	Time of concentration of directly connected impervious area of catchment.	time
TA_i	Tributary area.	ha
TW	Tail water depth downstream of feature.	m
V	Velocity.	m/s
V	Total volume (often taken as 1 when V_v is expressed as a percentage).	
V_{ave}	Mean flow velocity.	m/s
V_a	Maximum non-scouring velocity.	m/s
V_o	Average outlet velocity.	m/s

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Term	Description	Units
V_s	Average vertical settling velocity.	m/s
Vv	Total volume of voids.	%
W	Basin width.	m
W_{avg}	Average width of the ponded cross section above a filter medium.	m
W_{base}	Width of an infiltration area.	m
X	Length of beaching downstream.	m
Z	Reciprocal of the channel crossfall.	m/m
Zd	Elevation of the dissipator floor level.	m
Zc	Elevation of the culvert floor level (culvert invert).	m
Zt	Elevation of the tailwater (typically about 10 m downstream from feature).	m
λ	Energy loss factor at base of overfall.	Dimensionless
η	Kinematic viscosity of water, approximately 1.0×10^{-6} m^2/s at 20°C.	m^2/s
[illegible]	Porosity of the infiltration media.	m/s

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APPENDICES

APPENDIX A

A.1 WORKED EXAMPLE OF THE DERIVATION OF IFD CURVES

The derivation of IFD curves is described in ARR Vol. 1, Book 2 (2001) and consists of eight steps. Presented below is the algebraic method suitable for input into a computer spreadsheet package. A numerical example is included at the end of the procedure to validate calculations.

Step 1

The basic storm intensities for storms with an ARI of two years and 50 years and durations of one hour, 12 hour and 72 hour are on six sets of maps covering all of Australia. In addition to these there are two further sets of maps that show geographical factors for the two year and 50 year ARI. Map 7 details the skewness coefficient for the rainfall data.

- determine the rainfall intensities from Maps 1 to 6 ARR Vol. 2
- determine the skewness, G, from Map 7 ARR Vol. 2
- determine the geographical short duration factors, F2 and F50, from Maps 8 and 9 ARR Vol. 2, respectively (required for durations less than one hr).

Step 2

A short duration intensity of six minutes is determined in this step. Later in the procedure this data will be used to interpolate intensity values for storm durations between six and 60 minutes. If durations greater than one hour are required, this step may be skipped.

Calculate the intensity for a six minute storm with a two year ARI:

$$^{2}i_{6m} = F2 \times \left(^{2}i_{1}\right)^{0.9} \qquad \text{Equation (A.1)}$$

Calculate the intensity for a 6 minute storm with a 50 year ARI:

$$^{50}i_{6m} = F50 \times \left(^{50}i_{1}\right)^{0.6} \qquad \text{Equation (A.2)}$$

Where $^{2}i_{1}$ and $^{50}i_{1}$ are the 2 year ARI one hour storm intensity and the 50 year ARI one hour storm intensity (from Maps 1 and 4), respectively.

Step 3

The regional skewness is applied into the data for the four durations (6 min, 1 h, 12 h and 72 h).

Calculate the mean for each duration:

$$X_D = \log_{10}\left(^{2}i_D / 1.13\right) \qquad \text{Equation (A.3)}$$

Calculate the standard deviation for each duration:

$$S_D = 0.4869 \times \log_{10}\left(^{50}i_D \times 1.13 / {}^{2}i_D\right) \qquad \text{Equation (A.4)}$$

Where D is the rainfall duration (6 min, 1 h, 12 h and 72 h) and thus four means and standard deviations are calculated.

The skewed rainfall intensities are then calculated:

$$^{Y}I_D = {}^{Y}P\left(10^{\left(X_D + {}^{Y}K \times S_D\right)}\right)$$ Equation (A.5)

where ^{2}P = 1.13, ^{5}P = 1.05 and ^{Y}P = 1.00 for Y ≧ 10 years.

$$^{Y}K = 2 \times \left(\left\{\left[{}^{Y}K_N - G/6\right]G/6 + 1\right\}^3 - 1\right)/G$$ Equation (A.6)

where G is the skewness factor from Map 7, ARR Vol. 2.

^{Y}K is the log Pearson Type III standard deviate for an ARI of Y years and is given in the Table A.1.

Table A.1: Log Pearson Type III standard deviate where the skewness coefficient, N, is equal to zero

$^{2}K_N$	$^{5}K_N$	$^{10}K_N$	$^{20}K_N$	$^{50}K_N$	$^{100}K_N$	$^{500}K_N$
0.0000	0.8416	1.2816	1.6449	2.0537	2.3263	2.878

Note at this step only the two ARI intensities (two and 50 years) are calculated but at Step 5 the additional ARI intensities will be required to be calculated.

Step 4

This step is optional but is used to check that the calculations undertaken so far are correct. Using the interpolation diagram (Diagram 2.1 ARR Vol. 2) plot the intensities for the four durations and the two ARI.

Step 5

The rainfall intensities for the four durations (6 min, 1 h, 12 h and 72 h) are now calculated for ARI of 5, 10, 20 and 100 years using Equation (A.5) and Equation (A.6). The equations can be applied to any ARI above two years.

Step 6

The one year ARI intensities are calculated for the same four durations (6 min, 1 h, 12 h and 72 h).

$$^{1}I_D = 0.885 \times {}^{2}I_D / \left[1 + 0.4046 \log_{10}\left(1.13 \times {}^{50}I_D / {}^{2}I_D\right)\right]$$ Equation (A.7)

Where D is the duration for which the rainfall intensity is sought.

Step 7

From the data calculated above, it is possible to derive the rainfall intensity of a storm with any duration between 6 minutes and 72 hours duration. If a non-standard ARI were required then the Equation (A.5) and Equation (A.6) would need to be used for that particular ARI before proceeding to interpolate for the storm duration-intensity values.

$$^{Y}I_D = {}^{Y}I_L\left({}^{Y}I_U / {}^{Y}I_L\right)^N$$ Equation (A.8)

where

$$N = \left(P_D - P_L\right)/\left(P_U - P_L\right)$$ Equation (A.9)

and

$$P_D = \log_{10}(D) + 0.103\left(\log_{10}(D)\right)^2 - 0.0710\left(\log_{10}(D)\right)^3 + 0.0108\left(\log_{10}(D)\right)^5$$ Equation (A.10)

where

L = the basic duration below the required duration, D.

U = the basic duration above the required duration, D.

Using Equation (A.10) PL, PU and PD are calculated. These are then used to determine N from Equation (A.9) which is input into Equation (A.8) to determine the rainfall intensity for a storm of duration D and an ARI of Y. A number of values of N have been calculated and these are listed in Table A.1 at the conclusion of Chapter 2 of ARR Vol. 1, Book 2.

Step 8

This step is optional and involves fitting a 6th degree polynomial to each ARI design curve. Where a computer solution has been implemented this step is redundant unless charts are to be compiled for use or for records.

Example of IFD Calculations

Data is presented for Dunk Island off the Queensland coast between Cairns and Townsville. The values for Step 1 are read from the maps in ARR Vol. 2 are shown inTable A.2.

Table A.2: Design rainfall isopleths, geographical and skewness factors for Dunk Island (ARR Vol. 2, 1987)

Map 1 ${}^{2}I_{1}$	Map 2 ${}^{2}I_{12}$	Map 3 ${}^{2}I7_{2}$	Map 4 ${}^{50}I_{1}$	Map 5 ${}^{50}I_{12}$	Map 6 ${}^{50}I7_{2}$	F2	F50	G
65.0	14.8	5.8	120.0	34.0	13.0	3.97	17.2	0.06

Step 2 ${}^{2}i_{1}$ and ${}^{50}i_{1}$ were 170 mm/h and 304 mm/h, respectively

Step 3 X_{6m}, S_{6m}, X_{72} and S_{72} were 2.18, 0.15, 0.71 and 0.20, respectively (intermediate values not shown)

${}^{2}I_{6m}$, ${}^{100}I_{6m}$, ${}^{2}I_{72}$ and ${}^{100}I_{72}$ were 170 mm/h, 6 mm/h, 339 mm/h 15 mm/h, respectively (intermediate values not shown).

${}^{2}K_{0.06}$ and ${}^{100}K_{0.06}$ were 0 and 2.37 (intermediate values not shown).

Step 4 Optional

Step 5 ${}^{10}I_{6m}$and ${}^{10}I_{1}$ were 234 mm/h and 91 mm/h respectively

Step 6 ${}^{1}I_{6m}$ and ${}^{1}I_{72}$ were 149 mm/h, 5 mm/h respectively

Step 7 L = 6 minutes and U = 1 hour

P_D, P_L and P_U were–0.17, –0.8368 and 0 respectively

N was 0.794

${}^{10}I_{6m}$ and ${}^{10}I_{1}$ were calculated in step 5.

Based on the data in Table A.1, the storm intensity with 40 minutes duration with an ARI of ten years was calculated to be 132 mm/h.

Step 8 Optional

The IFD curves developed above cover durations from six minutes up to 72 hours and with ARIs ranging up to 100 years. The curves can be extrapolated up to 500 years where required and the ${}^{500}K_N$ value for use in Equation (A.6) has been included in Table A.1.

A.2 WORKED EXAMPLE OF THE RATIONAL METHOD

A.2.1 Example of single land-use catchments

Consider runoff from an impervious catchment (runoff 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 runoff is conveyed directly to, and discharged from, 'O' [see Figure A.1]. 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 runoff hydrograph shown in Figure A.2. Note that the 'time of rise' of the hydrograph is t_c minutes, the catchment's time of concentration.

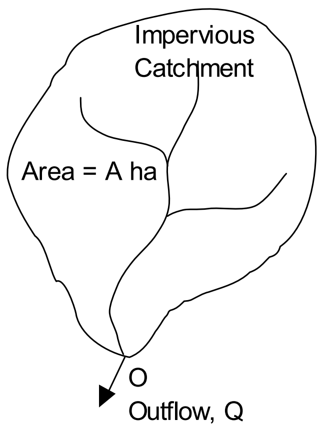


Figure A.1: Impervious catchment 'I' of area A ha draining to O (after ARRB Special Report 34, 1986)

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 Equation (3.9) with the runoff coefficient equal to one.

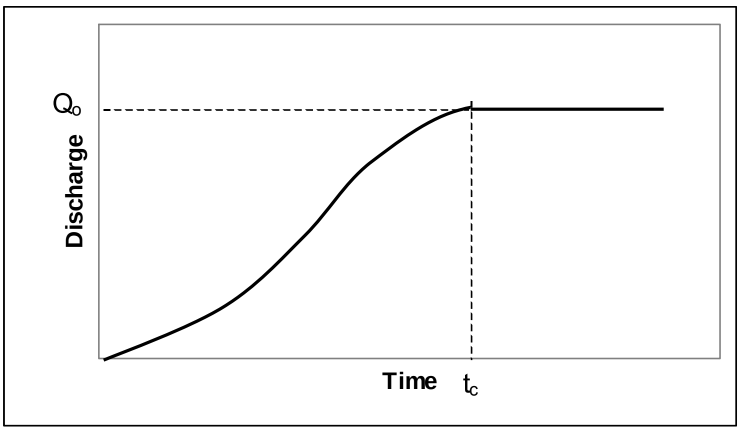


Figure A.2: An idealised runoff hydrograph for a constant intensity storm for catchment 'I' (after ARRB Special Report 34, 1986)

Consider now runoff from Catchment B, 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 (see Figure A.3). Catchment B is located in the climatic region whose rainfall IFD relationship for frequency ARI = 10 years is presented in Figure A.4.

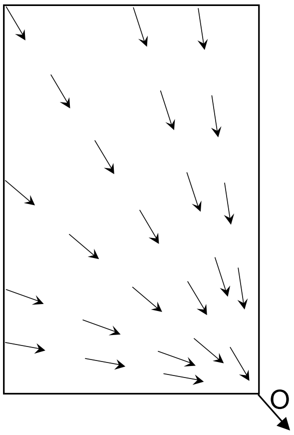


Figure A.3: Catchment B (after ARRB Special Report 34, 1986)

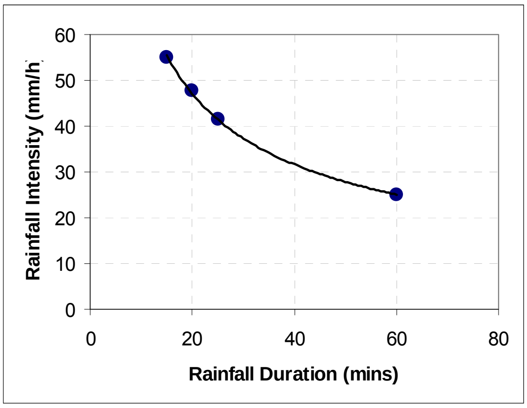


Figure A.4: Rainfall intensity-duration relationship for ARI = 10 years (after ARRB Special Report 34, 1986)

If the storm events (intensity constant), from the Figure A.4 curve, are applied to Catchment B, the resulting runoff hydrographs at ‘O’ take the forms shown in Figure A.5. It has been assumed that the speed at which the runoff travels to the discharge point at ‘O’, is the same for the whole catchment, hence the straight lines in the hydrographs in Figure A.5.

- intensity ${}^{10}I_{15m}$ = 55 mm/h, for a duration of 15 minutes (dash-dot line)
- intensity ${}^{10}I_{20\,m}$ = 48 mm/h, for a duration of 20 minutes (solid line)
- intensity ${}^{10}I_{25m}$ = 42 mm/h, for a duration of 25 minutes (dotted line)
- intensity ${}^{10}I_{1}$ = 25 mm/h, for a duration of 60 minutes (dashed line).

For hydrographs listed above, the peak flow rate at ‘O’, Q_O, is given by application of Equation (3.9) and is:

$$Q_{peak} = \frac{(C \times A) \times {}^{10}I_D}{0.36}$$

where

Q_{peak}	peak flow rate in L/s
C	runoff coefficient (= 1.00 in each case)
A	catchment area contributing the discharge at time of concentration (ha)
${}^{10}I_D$	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_O = (1.0 x 0.10 x (15/20)) 55/0.36 = 11.5 x 10^{-3} m^3/s
- Q_O = (1.0 x 0.10) 48/0.36 = 13.3 x 10^{-3} m^3/s
- Q_O = (1.0 x 0.10) 42/0.36 = 11.7 x 10^{-3} m^3/s
- Q_O = (1.0 x 0.10) 25/0.36 = 6.9 x 10^{-3} m^3/s.

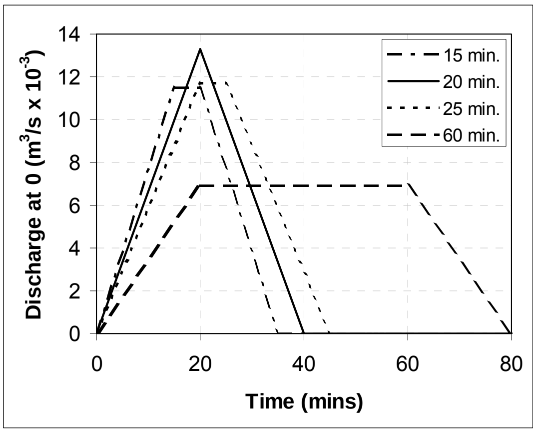


Figure A.5: Hydrograph for rainfall bursts on Catchment B (adapted from ARRB Special Report 34, 1986)

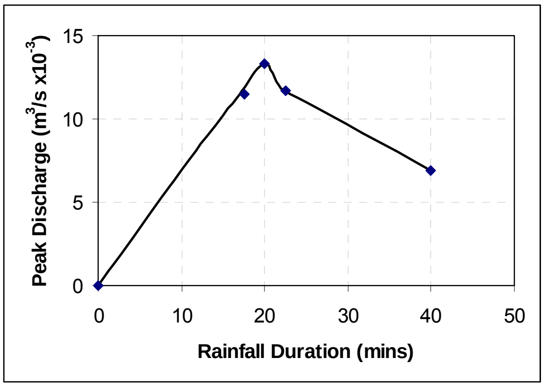


Figure A.6: Plot of peak discharge against rainfall duration for Catchment B

The peak discharges can then be plotted against storm duration to determine the design discharge for the catchment as shown in Figure A.6. The maximum discharge (13.3 $\times 10^{-3}$ m^3/s) occurs when the rainfall duration is 20 minutes, i.e. when the rainfall duration is equal to the time of concentration for catchment B.

The trend displayed in Figure A.5 and Figure A.6 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 runoff 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.

A.2.2 Example of multi land-use catchments

When discharge estimates are required in a multi land-use catchment of the type illustrated in Figure A.7 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 runoff 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 runoff 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 runoff coefficients C_1 and C_2, the weighted runoff coefficient, C_W, is:

$$C_W = \frac{C_1 A_1 + C_2 A_2}{A_1 + A_2} \qquad \text{Equation (A.11)}$$

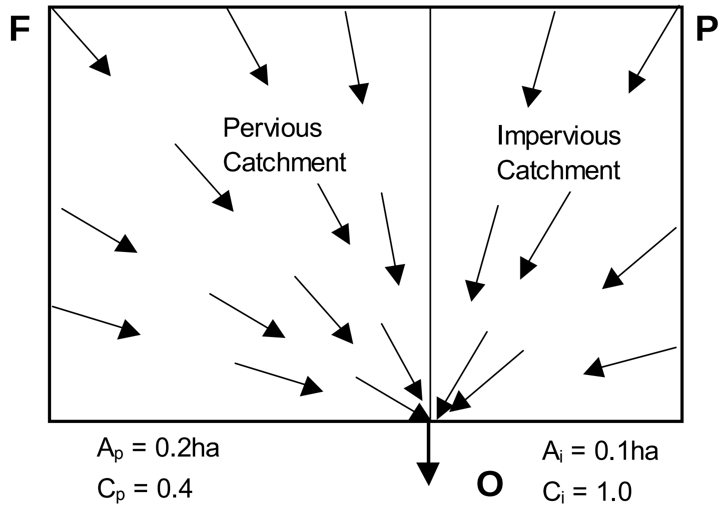


Figure A.7: Catchment E — composite impervious / pervious catchment (after ARRB Special Report 34, 1986)

The following example illustrates conventional application of the Rational Method to a multi land-use catchment, in this case Catchment E:

impervious area,	A_i	=	0.10ha 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 weighted runoff coefficient is determined using Equation (A.11):

$$C_W = \frac{(1.00 \times 0.10) + (0.40 \times 0.20)}{(0.10 + 0.20)} = 0.60$$

Time of concentration must be the greater of t_i and t_p 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 Equation (3.9):

$$Q_0 = \frac{(C_w A)^{10} I_1}{0.36} = \frac{(0.60 \times 0.30) \times 25}{0.36} = 12.5\text{m}^3/\text{s} \times 10^{-3}$$

But this is less than the peak discharge estimated at ‘O’ for that part of Catchment E previously considered in Section A.2.1 as Catchment B.

A.2.3 Example of time / area representation

The Rational Method assumption that *’...the speed with which runoff elements travel to discharge point, ‘O’, is steady...*’ 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 A.8 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 runoff 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 runoff at 'O' at time $t_c/4$. Similarly for the area / time pairs A/2 and $t_c/2$, 3A/4 and $3t_c/4$, A and t_c. These pairs, plotted in Figure A.8, yield the time-area relationship for the catchment.

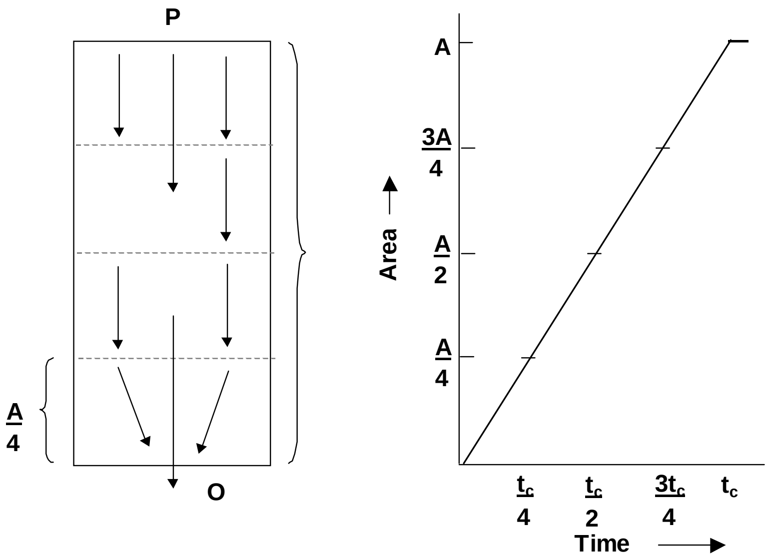


Figure A.8: Time-area graph for simple catchment of area, A and time of concentration, t_c (after ARRB Special Report 34, 1986)

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 runoff coefficient and component area, (CA). The time-area graph of Catchment E, presented in Figure A.9, illustrates this process.

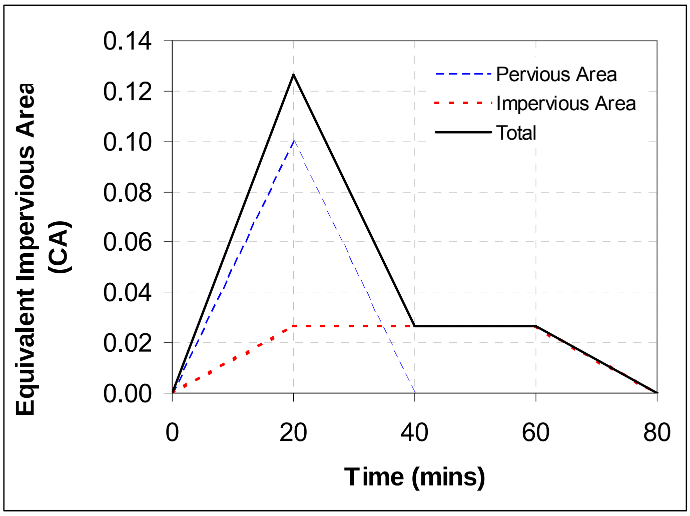


Figure A.9: Time-area graph for Catchment E

Catchment E:

- At 20 min. rain fall stops. All of impervious area contributing, one third of 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, lowest third of pervious area ceases to contribute but mid third area of pervious area contributes to discharge. Therefore at this time one third of pervious area contributes.
- At 60 min. the furthest point in the pervious area begins to contribute. Mid third area ceases to contribute to discharge. Furthermost third of pervious area contributes.
- At 80 min. all flow ceases.

A.2.4 The Partial Area Rational 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 unit. The two design outflows arise from 'full-area' and 'part-area' considerations.

Full-area flow estimate

Peak runoff flows are determined for single or multiple land-use drainage units in the manner described in Section A.2.3 using Rational Method procedures for multiple land use:

- critical design storm duration = t_c, travel time from the outer extremity of the most remote pervious area
- full equivalent impervious area, $(CA)_{full}$ = ($C_W A$), where C_W is determined by application of Equation (A.11)
- fainfall intensity, ${}^{Y}I_{tc}$ = average intensity, duration to be obtained from catchment rainfall intensity-duration chart for selected ARI of Y years; and
- discharge is calculated using Equation (A.11).

The flows calculated by this approach are referred to as 'full-area' flow estimates.

Directly connected impervious (or paved) areas are those that contribute runoff directly to the drainage collection network. Such runoff may be conveyed by pipe, channel or informally across the impervious surface before reaching the formal collection system (e.g. roadside channels). Runoff, 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 runoff flows are also determined for the same catchment units using:

- critical design storm duration = t_i, travel time from the outer extremity of the most remote, directly connected impervious area
- 'part' equivalent impervious area is calculated as below:

$$(CA)_{part} = C_i A_i + \left(\frac{t_i}{t_c} \times C_p A_p \right)$$ **Equation (A.12)**

- rainfall intensity ${}^{Y}I_{tc}$ = average intensity, duration to be obtained from catchment rainfall intensity-duration chart for selected ARI of Y years.

The flows calculated by Equation (A.12) are referred to as 'part-area' flow estimates.

The theoretical basis for Equation (A.12) follows from time-area representation (see Section A.2.3) 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 presented in Figure A.7. 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 Part-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 A.7.

Full-area estimate by Equation (3.9):

equivalent impervious area = ($C_W A$) = 1.0x0.1 + 0.4x0.2 = 0.18 ha

storm duration (equal to t_c) = 60 minutes, hence ${}^{10}I_{60\,m}$ = 25 mm/h

Hence, full-area peak flow estimate,

$$Q_f = \frac{(0.18)\times 25}{0.36} = 12.5 L/s$$

Part-area estimate by Equation (A.12):

(equivalent impervious area) $= C_i A_i + \left(\frac{t_i}{t_c} \times C_p A_p \right)$

= 1.0 x 0.10 + (20/60 x 0.4 x 0.2)

= 0.127 ha

storm duration (equal to t_i) = 20 minutes, hence ${}^{10}I_{20\,m}$ = 48 mm/h

Part-area peak flow estimate:

$$Q_p = \frac{0.127 \times 48}{0.36} = 16.9 \ L/s$$

Recommendation: outlet works for point 'O' in Catchment E should carry a design flow (ARI = 10 years) of 16.9 L/s.

APPENDIX B

B.1 WORKED EXAMPLE OF ESTIMATION OF GAP FLOW

For a location in the northern (tropical) part of Australia the rainfall intensities in Table B.1 apply for a storm with a duration of 45 minutes, previously determined as the critical storm duration. It is desired to determine what ARI should be used to calculate Q_{gap} for the design of the major system elements.

Table B.1: Rainfall Intensity (mm/h) for a 45 minute duration storm in tropical Australia

ARI	Frequency factor (Table 3.1)	Intensity (mm/h)	Intensity x frequency factor
1	0.80	65	52
2	0.85	75	64
5	0.95	94	89
10	1.00	105	105
20	1.05	120	126
50	1.15	139	160
100	1.20	154	185

For the particular location and situation it has been decided that the major event occurs with an ARI of 100 years (taken from Table 4.2). The data in Table B.1 is plotted in Figure B.1. The Q_{gap} can be determine for each of the varying ARIs applicable to the minor drainage system and these are shown in Table B.2 (divided by [CA_{10}/0.36]) for an assumption of, zero blockage and with 50% blockage. These values can then be used to estimate the design ARI for Q_{gap} using Figure B.1. The final two columns of Table B.2 are simply rounding up of the design ARI which provides some conservatism in the procedure.

Table B.2: Estimation of Q_{gap}

N (years)	$Q_{gap} = Q_{100} - Q_n$ (need to multiply by $CA_{10}/0.36$)		Estimate ARI for Q_{gap} (from Figure B.1)		ARI rounded up	
	0% Blockage	50% Blockage	0% Blockage	50% Blockage	0% Blockage	50% Blockage
1	133	159	21	50	25	50
2	121	153	14	41	15	45
5	96	140	6	26	10	30
10	80	132	3	20	5	20
20	59	122	2	14	2	15
50	25	105	0.5	8	1	10

Using the data in Table B.2: it is possible to design the major drainage system to cater for Q_{gap} based on a minor drainage system designed to cope with storms with varying ARI and blockage levels.

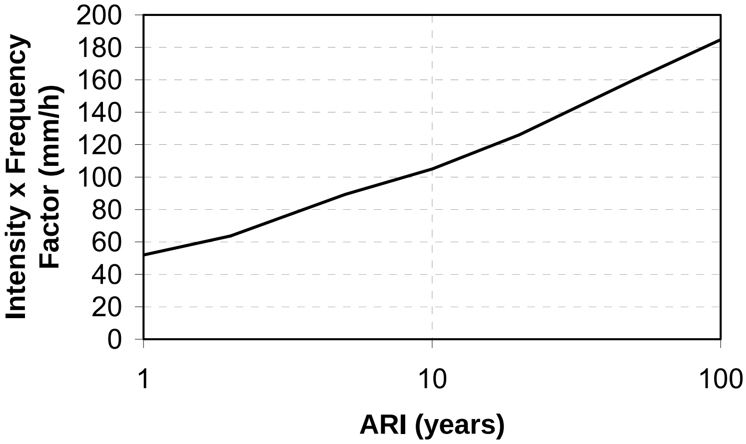


Figure B.1: Intensity x Frequency factor for various ARI plotted on a logarithmic scale

B.2 WORKED EXAMPLE OF MAJOR DRAINAGE SYSTEM DESIGN

B.2.1 Introduction

A step-by-step procedure for planning a major stormwater drainage system for an urban catchment, is applied to a 38 ha hypothetical residential sub-division in the Adelaide foothills, South Australia.

The catchment area is bordered on three sides (north, east and west) by main roads (see Figure B.2). A green belt reserve forms its southern boundary. The catchment topography slopes generally to the south making this the natural drainage direction.

Housing density in the sub-division will be, initially, 16 residences per ha of dedicated area i.e. excluding roadway reserves, etc. Ultimate development during the life of the drainage system is estimated to be equivalent to 20 residences per ha. Some open space park areas are located within the sub-division.

The road layout uses a road pattern of the conventional grid type adapted to the site. All roads will have sealed pavement carriageways with concrete kerb-and-gutter borders. It is anticipated that underground pipes will be used in the catchment minor drainage system. Partial blockage (50 per cent blockage) of these pipes in major runoff events is considered likely.

The sub-division is illustrated in Figure B.2. It is assumed, for purposes of illustration, that runoff from individual residences in the ultimately developed catchment will be directly-channelled to the surface stormwater drainage system. Drainage from positive grade allotments (see Figure B.2) will be conveyed directly to fronting roadways and drainage from adverse grade allotments to rear-of-allotment channels or rear access lanes.

It may be further assumed that High Street will have the status of local traffic distributor. All other residential roads and streets will have 'access road' status. No roundabouts or street closures are anticipated during the life of the sub-division.

The worked example is presented as an eight step process as discussed in Section 4.2.2 and data and information relating to the example are presented below.

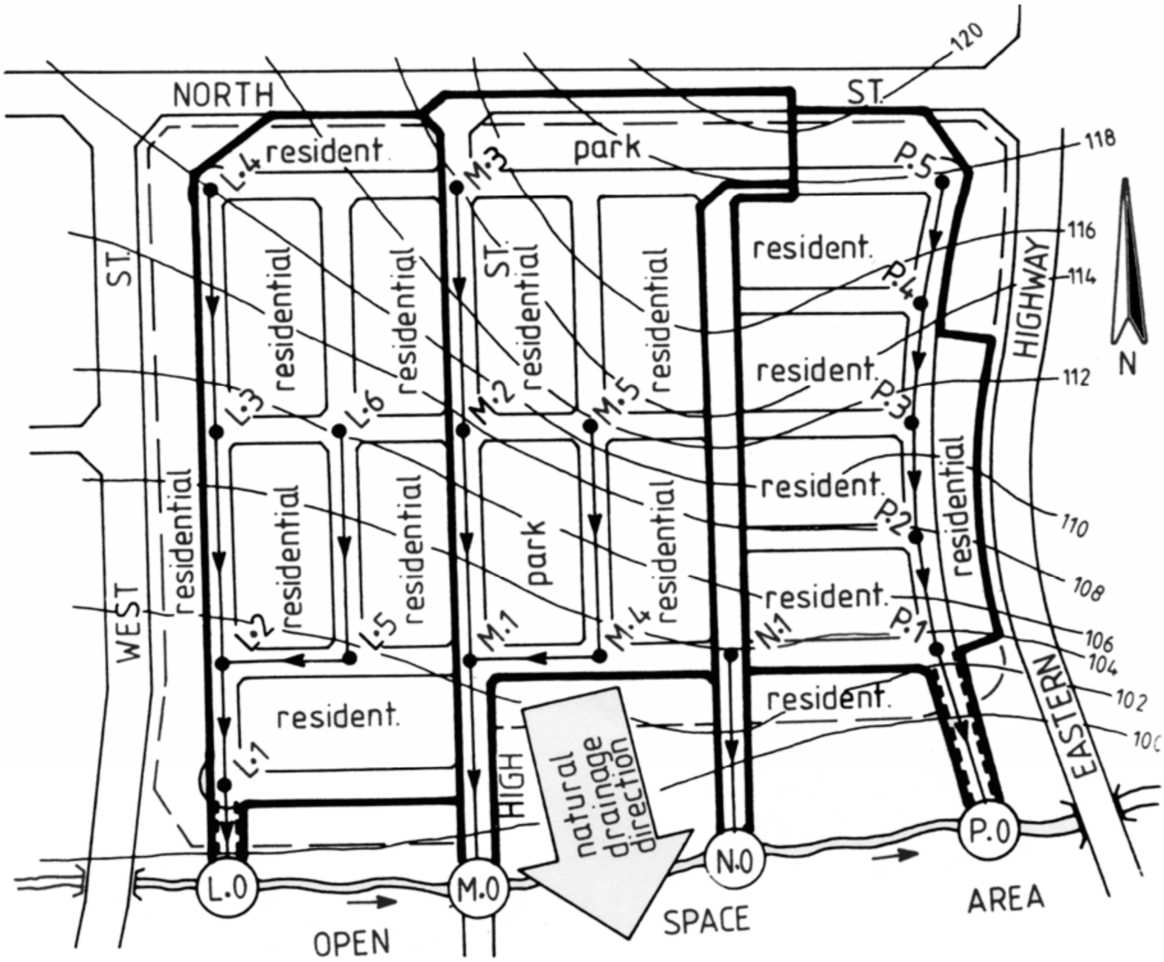


Figure B.2: Theoretical residential subdivision in the Adelaide foothills (after ARRB Special Report 34, 1986)

B.2.2 Step 1: Catchment definition

The following data and information relating to catchment definition are available. Most items are included in Figure B.2.

- Catchment location: Adelaide foothills
- contour map (see Figure B.2)
- sub-division catchment boundary (see Figure B.2)
- pattern of internal roads and streets (see Figure B.2)
- dual-channel and potential single-channel streets (see Figure B.2)
- major (common) land use areas (see Figure B.2)
- an underground network will be incorporated, design ARI = 2 years: 50 per cent blockage to be assumed
- natural drainage direction (see Figure B.2)
- flood disposal points (see Figure B.2)
- internal sub-areas within catchment boundaries and node points (see Figure B.3)
- flood escape networks, node sections and sub-areas (see Figure B.3).

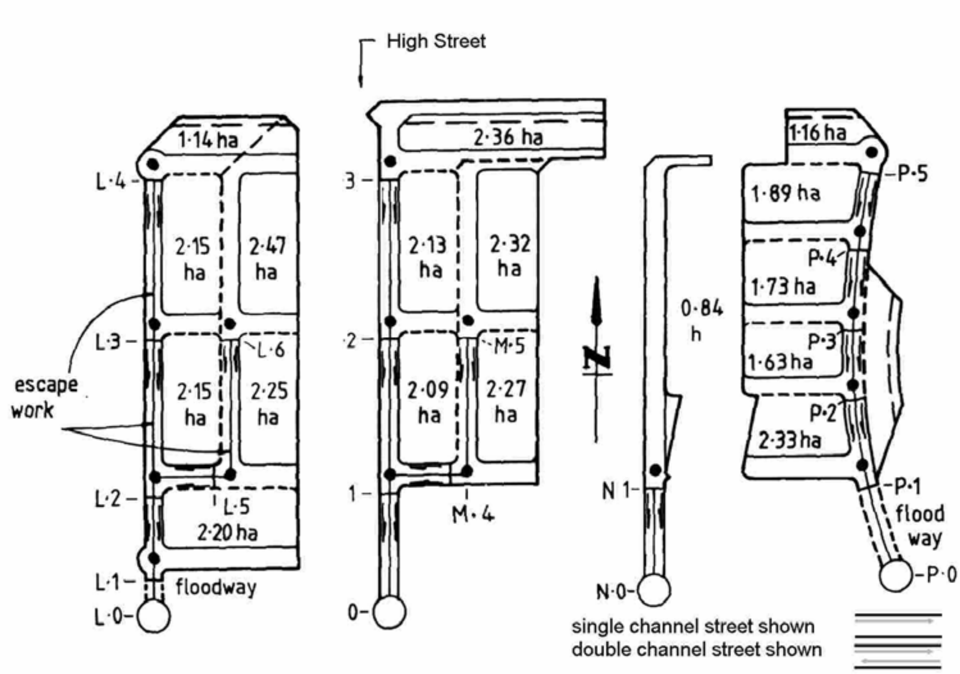


Figure B.3: Sub-areas within catchment (after ARRB Special Report 34, 1986)

Concerning ‘catchment boundary and boundary constraints’, there is no inflow to the roads and streets of the subdivision from outside its boundary. The passing of storm runoff to side boundary flood paths (Eastern Highway and West Street) should be minimised. It is assumed that these flood paths convey sizeable flows from remote catchments and that additional floodwater input in the vicinity of the receiving domain should only occur if unavoidable.

The preliminary road hierarchy information available for the sub-division is sufficient to force a change in one item of catchment definition data relating to High Street. Its status as a local traffic distributor requires its cross section to be changed from potential single channel (northern segment) to dual channel throughout.

B.2.3 Step 2: Roadway Reserve Capacity Flows

The particular roadway forms to be used in the development consist of:

- 7.5 m and 10.0 m carriageways within 16.0 m and 20.0 m road reserves, respectively
- kerb-and-gutter profile; 0.375 m wide with 0.150 m depth
- pavement cross-slopes, Z_b = 30 and 40 for the 7.5 m and 10.0 m carriageways, respectively.

The 7.5 m carriageways are used for access roads, the 10.0 m carriageway for local distributors.

Flow capacities of these carriageways (or half-carriageway flows) are presented in Table B.3. The capacities of these carriageways have been increased by 10 per cent to model the storage capacity and these storage-corrected capacities are shown in brackets in Table B.3.

Table B.3: Capacity flows, Q_c, and storage-corrected capacity flows (in brackets) for 7.5 m and 10.0 m carriageways

Longitudinal Slope S	FLOW CAPACITIES, Q_c, IN L/s			
	7.5 m carriageways: Z_b = 30		10.0 m carriageways: Z_b = 40	
	Single channel	Dual channel	Single channel	Dual channel
0.005	390 (430)	780 (860)	500 (550)	1000 (1100)
0.010	560 (615)	1120 (1230)	700 (770)	1400 (1540)
0.020	780 (860)	1560 (1720)	990 (1090)	1980 (2180)
0.030	700 (770)	1400 (1540)	915 (1005)	1830 (2010)
0.040	635 (700)	1270 (1400)	830 (915)	1660 (1830)
0.050	580 (640)	1160 (1280)	760 (835)	1520 (1670)
0.060	535 (590)	1070 (1180)	700 (770)	1400 (1540)

Values in brackets indicate the storage corrected capacities

B.2.4 Step 3: Design storm selection

The major drainage systems are required to be designed for an ARI of 100 years (see Table 4.2) and partial blockage of 50 per cent of the associated underground network is assumed. Since the minor drainage system will be designed subsequently for ARI of two years, it follows that the gap flow, Q_{gap}, is equal to the total discharge from the catchment minus the runoff carried by the minor drainage system. For this example the Q_{gap} will be determined based on an ARI of 50 years. Appendix B.1 outlines the procedure for determining an ARI for the Q_{gap}.

Design storm duration for the particular conditions which are anticipated for the ultimately developed catchment may be found from Table 4.1. For this example a storm duration equal to 10 minutes is adopted.

Taking design ARI 50 years for the surface system only, and storm duration of 10 minutes, design storm average intensity (${}^{50}I_{10}$) is estimated to be 90 mm/h.

B.2.5 Step 4: System planning table

By bringing together data from Step 2 and Step 3, it is possible to determine the tributary (impervious) area, TA_i, which will yield, under design storm conditions, a peak runoff flow matching the storage-corrected capacity of any roadway flood escape path likely to be used in the Figure B.2 developments. The TA_i value required in each roadway case can be found from substitution into the Rational Method formula.

$$Q_{sc} = \frac{(TA_i)\ {}^{50}I_{10}}{0.36} \quad \text{L/s} \qquad \text{Equation (B.1)}$$

where

Q_{sc} = storage-corrected capacity flow in flood escape path (from Table B.3)

${}^{50}I_{10}$ = average rainfall intensity for storm with a design ARI of 50 years and a duration of 10 min

TAi = tributary (impervious) area in ha

$$TAi = \frac{0.36\ Q_{sc}}{90} = \frac{Q_{sc}}{250}\ ha \qquad \text{Equation (B.2)}$$

Applying Equation (B.2) to the storage-corrected capacity data in Table B.3 gives a set of tributary (impervious) areas which can, for design storm conditions, be serviced by the 7.5 m and 10.0 m roads and streets. At this time the flows in the major system elements need to comply with the two criteria in Section 4.2.2 — Fixing of roadway reserve capacity.

A major system planning table can now be assembled and this is shown in Table B.4.

Table B.4: Major Drainage System Planning Table

Longitudinal Slope S (m/m)	Tributary (impervious) area TA_i (hectares) which can be serviced by roadway flood escape path in design storm			
	7.5 m carriageways: Z_b = 30		10.0 m carriageways: Z_b = 40	
	Single channel	Dual channel	Single channel	Dual channel
0.005	1.72	3.44	2.20	4.40
0.010	2.46	4.92	3.08	6.16
0.020	3.44	6.88	4.36	8.72
0.030	3.08	6.16	4.02	8.04
0.040	2.80	5.60	3.66	7.32
0.050	2.56	5.12	3.34	6.68
0.060	2.36	4.72	3.08	6.16

B.2.6 Step 5: Network Review

One final item of information is needed before the network review can be commenced. This is the set of runoff coefficients that must be applied to convert catchment component areas into equivalent impervious areas, (C x A). Appropriate factors are to be found by the application of Equations (3.4) to (3.6).

Three land uses only are included in the Table A.2 sub-division:

- residential, 20 residences per (excluding roads. etc.)
- park areas
- roadway reserves of 16 m and 20 widths.

Data required:

${}^{10}I_d$ rainfall intensity for a storm with an ARI of 10 years and a duration of d (10 minutes for this example) = 26 mm/h

f fraction impervious

In the case of residential segments = allow 43%

In the case of park lands = allow 0%

F_y frequency conversion factor = 1.15 (from Table 3.1).

Hence, for residential areas,

$$C^1_{10} = 0.1 + 0.0133\ ({}^{10}I_1 - 25) = 0.1 + 0.0133\ (26 - 25) = 0.1133$$

$$C_{10} = 0.9 \times f + C^1_{10} \times (1\text{-}f)$$

$$= 0.9 \times 0.43 + 0.1133\ (1 - 0.43) = 0.45$$

$$C_{50} = Fy \times C_{10} = 1.15 \times 0.45 = 0.52$$

In the case of park areas,

$$C^1_{10} \quad \text{as previous} = 0.1133$$

$$C_{10} = 0.9 \times f + C^1_{10} \times (1\text{-}f)$$

$$= 0.9 \times 0 + 0.1133\ (1 - 0) = 0.11$$

$$C_{50} = Fy \times C_{10} = 1.15 \times 0.11 = 0.12$$

Roadway reserves comprise about 50 per cent carriageway area, 15-30 per cent footpath and driveways and the remainder nature strips. Use of a lumped, arbitrary fraction impervious of 0.85 to convert roadway reserve area to equivalent paved area is recommended. The runoff coefficient that should be applied to the converted area is:

$$C^1_{10} \quad \text{as previous} = 0.1133$$

$$C_{10} = 0.9 \times f + C^1_{10} \times (1\text{-}f)$$

$$= 0.9 \times 0.7 + 0.1133\ (1 - 0.7) = 0.78$$

$$C_{50} = Fy \times C_{10} = 1.15 \times 0.66 = 0.90$$

Using these coefficients it is possible to determine weighted C_{50} values which may be applied to each sub-area of the development to determine their individual $(C \times A)_{50}$ values (see worked example in Appendix A, Section A.2).

Great accuracy is not warranted in performing this task and the following values, based on the above, are considered satisfactory:

- Residential sub-areas (including surrounding roads)

  Weighted runoff coefficient, allow 15% of subdivision to be road reserve and 85% to be residential development

  Weighted C_{50} = $(0.85 \times 0.52 + 0.15 \times 0.9) / (0.85 + 0.15) = 0.58$

- Park sub-areas (including surrounding roads), allow 20% of subdivision to be road reserve and 80% to be park lands

  Weighted C_{50} = $(0.80 \times 0.12 + 0.2 \times 0.9) / (0.80 + 0.20) = 0.28$

These coefficients are incorporated into the Network Review Tables (Table B.5). The areas for each sub-area within the catchment are summarised in Table B.5 which is used in the calculation of the discharges.

Table B.5: Network review for catchments L, M, N and P

Sub-Area	Flood escape path details				Adjacent contributing sub-area			Cumulative upstream CA_{50} (ha)	Capacity Check	Remarks
	Node Section	Escape path		Tributary (impervious area, TA_i) (ha)	Area (ha)	Weighted runoff coefficient	$(C \times A)_{50}$ (ha)			
		Path description	Longitudinal slope, S (m/m)							
1	2	3	4	5	6	7	8	9	10	11
L	L.4	Dual channel, 7.5m	0.03	6.16	1.14	0.28	0.32	0.32	0.32 < 6.16	Okay
	L.3	Dual channel, 7.5m	0.02	6.88	2.15	0.58	1.25	1.57	1.57 < 6.88	Okay
	L.6	Dual channel, 7.5m	0.03	6.16	2.47	0.58	1.43	1.43	1.43 < 6.16	Okay
	L.5	Dual channel, 7.5m	0.005	3.44	2.25	0.58	1.31	2.74	2.74 < 3.44	Okay
	L.2	Dual channel, 7.5m	0.01	4.92	2.15	0.58	1.25	5.56	5.56 > 4.92	Reconsider
	L.2	Dual channel, 10m		6.16					5.56 < 6.16	Okay
	L.1	Floodway, 10m	0.005	4.40	2.20	0.58	1.28	6.84		
	Design flow at L.1 = 6.84/4.40 x 1000 = 1554 L/s									
M	M.3	Dual channel, 7.5m	0.01	4.92	2.36	0.28	0.66	0.66	0.66 < 4.92	Okay
	M.2	Dual channel, 7.5m	0.04	5.60	2.13	0.58	1.24	1.90	1.90 < 5.60	Okay
	M.5	Dual channel, 7.5m	0.03	6.16	2.32	0.58	1.35	1.35	1.35 < 6.16	Okay
	M.4	Dual channel, 7.5m	0.005	3.44	2.27	0.58	1.32	2.67	2.67 < 3.44	Okay
	M.1	Dual channel, 7.5m	0.02	6.88	2.09	0.28	0.59	5.16	5.16 < 6.88	Okay
	Note: These calculations indicate that a 7.5m carriageway for High Street is satisfactory, however, a 10m carriageway is required for traffic management.									
N	N.1	Dual channel, 7.5m	0.03	6.16	0.63	1.00	0.60	0.60	0.60 < 6.16	Okay
P	P.5	Dual channel, 7.5m	0.03	6.16	1.16	0.58	0.67	0.67	0.67 < 6.16	Okay
	P.4	Dual channel, 7.5m	0.03	6.16	1.89	0.58	1.09	1.79	1.76 < 6.16	Okay
	P.3	Dual channel, 7.5m	0.04	5.60	1.73	0.58	1.00	2.76	2.76 < 5.60	Okay
	P.2	Dual channel, 7.5m	0.03	6.16	1.63	0.58	0.95	3.71	3.71 < 6.16	Okay
	P.1	Floodway, 7.5m	0.04	5.60	2.33	0.58	1.35	5.06		
	Design flow at P.1 = 5.06/5.60 x 1270 = 1148 L/s									

For example, consider Catchment M (see Figure B.3 and Table B.). At node section M.3 the High Street carriageway is of dual channel type and although likely to take the 10.0 m form in the final urban plan is investigated here as a 7.5 m trial roadway.

The longitudinal slope, S, of High Street at node section M.3 is approximately 0.01 m/m.

Reference to Table B.4 (carriageway 7.5 m, S = 0.01m/m) reveals that it can convey a capacity flow matching the peak generated in 4.92 ha of tributary (impervious) area, TA_i. This information is summarised in Table B.5 under the heading 'flood escape path detail' (columns 2-5).

The sub-area which contributes runoff to node section M.3 is, mainly, a park (total sub-area of 2.36 ha) for which C_{50} = 0.28. This sub-area is therefore equivalent to 2.36 x 0.28 = 0.66 ha of impervious area, (C x A). These data are listed in columns 6-8 of Table B. under the heading 'adjacent contributing sub-area'. The 'cumulative upstream $(C \times A)_{50}$' for the catchment draining to M.3 is, of course, 0.66 presented in column 9 in Table B.5.

The flood escape path capacity check (column 10) compares the total $(C \times A)_{50}$ contributing to node section M.3 with the tributary (impervious) area, TA_i which could be serviced, under design storm conditions, by the flood escape path at that section. A successful outcome, 'Okay' (column 11), indicates that the trial road carriageway is satisfactory.

The same trial road form is then tested at node section M.2 where the cumulative upstream $(C \times A)_{50}$, column 9, is the sum of the areas draining to M.3 and M.2.

Catchment M has a flood escape path that divides at node M.I where runoffs from the northern and eastern arms join.

The trial road form at node section M.4 is a 7.5m dual-channel carriageway (see Figure B.3) the test at this section also succeeds.

At node M.I the flood escape path required by storm flows from the two arms of the network is checked against the TA_i which can be serviced by a trial 7.5 m dual-channel carriageway. In this case, the cumulative upstream $(C \times A)_{50}$ is the sum of the equivalent impervious areas at M.2 and M.4 and the sub-area adjacent to M.I. The test at node section M.I shows the 7.5 m carriageway to be adequate here also.

It is concluded from this analysis that the total runoff generated in Catchment M in a design storm of ARI = 100 years approximately can be contained within the 7.5 m carriageways of its flood escape network acting conjunctively with its associated underground network, assumed to be 50 per cent blocked. The likely decision to assign a 10.0 m carriageway to High Street for reasons of good traffic management practice will not conflict with this conclusion.

Consider now Catchment L (see Figure B.3 and Table B.5).

The analysis and tabulation proceed in much the same manner as for Catchment M until node section L.2 is investigated. Here, the test for a 7.5 m dual-channel flood escape path fails ('Reconsider' in Remarks column 11) and a 10.0m dual-channel carriageway is considered. This proves to be hydraulically satisfactory. However, it is unlikely that an access road will be built with 10.0 m carriageway for no reason other than its inability, as a 7.5 m residential street, to convey rare storm flows.

The designer is forced to explore a range of alternatives:

- remove roadway L.I-L.2 from the urban plan and extend the floodway north to L.2

- change gutter / pavement profile in roadway L.1-L.2 to one giving greater capacity within the limits of Criteria 1 and 2 (see Section 4.2.2)
- retain roadway L.1-L.2 as a 7.5 m dual-channel carriageway carrying flows which exceed the 'kerb-plus-50 mm' capacity criterion and apply local flood-proofing measures, e.g. raised footpaths, raised floor levels, etc.
- adopt design ARI = 10 years for minor system pipeline L.O-L.I-L.2, hence 'gap flow' would be based on a design ARI of 30 years.

Detailed design relating to these options need not, of course, be undertaken until Step 7.

The procedures outlined above have been followed for each of the four catchments and the results tabulated in Table B.5. The four sub-area catchments cover, between them, many of the common major network situations met in contemporary urban drainage practice.

B.2.7 Step 6: System evaluation

Completion of the above network review provides the designer with what should be regarded as a 'first approximation' of a major drainage system. This follows from the fact that important design information contained in the System Planning Table (Table B.5) is based on catchment travel time adopted from the guidelines of Table 4.1.

Before the design can progress to Step 7, a check on the suitability of the adopted catchment travel time must be made in each catchment of the derived major system. Note that major system catchment travel time for an urban development is taken as catchment impervious area travel time (see Table 4.1).

Allowing for roof-to-gutter or roof-to-easement travel and adding flow time along road and street gutters it will be observed that travel time of 10 minutes adopted in Steps 4 and 5 is, in some catchments, short. A travel time of 15 minutes for some catchments might be more appropriate.

The network review (Step 5) has shown that the flood escape paths of Catchments M and P can accommodate the runoff generated in a 10 minute design storm. Further review of these using a catchment travel time and, hence, storm duration of 15 minutes (lower rainfall intensity) will reach the same conclusion making it a redundant exercise.

The situation in Catchment L at node section L.2, warrants further attention.

Fifteen-minute ARI = 50 years storm bursts in the Adelaide foothills show an average intensity of 78 mm/h. Applying this value into Equation (B.2) yields a new relationship for tributary (impervious) area, namely:

$$TA_i = \frac{0.36\,Q_{sc}}{78} = \frac{Q_{sc}}{217}\ ha$$

There is no need to completely recalculate Table B.5 using the above revised expression for TA_i in order to further investigate one 'trouble spot' i.e. node section L.2 when in fact, only the TA_i value for a 7.5 m dual channel carriageway with S = 0.01 is required and leads to:

$$TA_i = \frac{1230}{217} = 5.67$$

For the particular surface channel of interest, comparison of this value with the cumulative $(C \times A)_{50}$ at node section L.2 ($[C \times A]_{50} = 5.56$) leads to the conclusion that the 7.5 m carriageway is satisfactory and that no further consideration need be given to alternatives (i) - (iv) listed in Step 5.

The above review concerns situations where the originally adopted (Step 3) catchment travel time of 10 minutes was less than that which closer inspection subsequently revealed to be more appropriate.

Of greater concern to the designer is the reverse situation where the adopted travel time proves to be the longer of the two. Catchment N (Figure B.2) falls within this category. Recalculation of TA_i for the node section N.I, however, is unnecessary considering the small cumulative $(C \times A)_{50}$ which the catchment draining to this section presents.

If, however, the difference is significant and widespread, e.g. affecting all main catchments in a development, then the designers have no alternative but to return the ‘revised’ shorter travel time into Step 3, recalculate Table B.5 and repeat Steps 5 and 6.

It is clearly advantageous to adopt travel times which prove to be short rather than long as this builds some conservatism into the design. Table 4.1 reflects this philosophy.

The above review, while fortuitous in its outcome for development of Figure B.2, highlights the need for close liaison to be maintained between the drainage system designer and those responsible for traffic management, building approvals (floor levels) and city planning generally.

It should be restated here that the major drainage networks identified in Figure B.3 has been shown capable of conveying design rare storm runoff flows without surcharge of their flood escape paths. This does not preclude the possibility of overflow from flow paths within component sub-areas. This matter is addressed in Step 7.

B.2.8 Step 7: Sub-area detailing

With a satisfactory major flood escape network in place in each catchment, the designer's next task is to check flow conditions in all roads and streets which convey flow to the main network. Principal among these are lateral streets, particularly those which are, potentially, of the single-channel type. The check which needs to be carried out is a reduced version of the ‘test and modify’ procedure of Table B.5.

Consider design extreme storm runoff joining the major system network from the lateral street leading to node point M.5 in catchment M (see Figure B.2). The terrain cross slope in the vicinity is about 5 per cent, it is potentially a single-channel carriageway street:

- flood path description: single-channel , 7.5 m longitudinal slope, $S_o = 0.02$
- tributary (impervious) area: 3.44 ha (from Table B.5)
- upstream contributing area* 1.40 ha
- upstream contributing $(C \times A)_{50} = 1.40 \times 0.58 = 0.81$ ha
- Capacity check: 0.81 < 3.44 therefore okay.

* Upstream contributing area is that sub-area portion which yields runoff in a design storm to the channel discharge section under review.

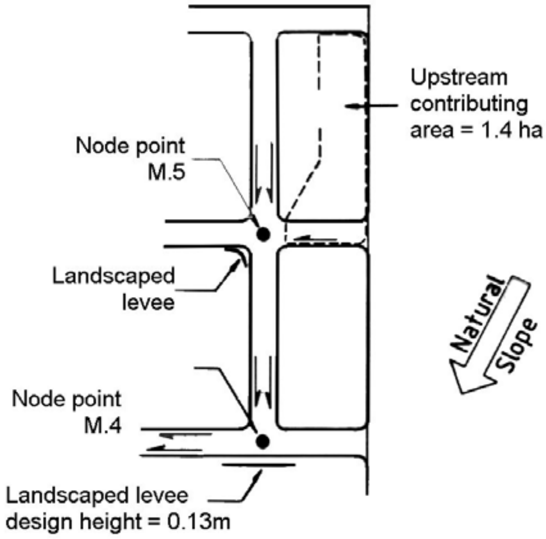


Figure B.4: Drainage detail at node points M.4 and M.5 (after ARRB Special Report 34, 1986)

Investigations carried out on streets which deliver storm runoff laterally to major system networks will almost invariably yield similar 'Okay' results. Experienced drainage designers are able to recognise the few problem situations by inspection.

Of greater concern are the problems that arise at roadway intersections.

The need to employ dual-channel carriageways, generally in flood escape networks, has been stressed. This precaution does not preclude, however, the possibility of surcharge at intersections such as at node point M.5 in Catchment M. Flood proofing in the form of a low landscaped levee bank within the nature strip at the south-western corner of the intersection may be warranted.

The channelling of floodwater, making its way to node point M.4 (Figure B.2), also needs careful attention. Again, the presence of a low, landscaped nature strip levee bank along the southern boundary of the intersection would be a valuable safeguard. Calculations carried out to fix the height of this embankment are part of the necessary hydraulic computations.

Some indication of the height required may be gained from the following:

If flow moving down slope towards node point M.4 were to be abruptly stopped by a pond of water in the region of the T-intersection, the maximum height of levee (approximate) which would be required to contain the pond is given by:

$$h = V^2_{avg}/2g$$

where

h levee height above top of kerb (m)

V_{avg} average velocity of down-slope moving flow (m/s)

g acceleration due to gravity (9.81 m/s^2)

In the vicinity of M.4, S, or the down-slope roadway is approximately 0.03 and average velocity approximately 1.6 m/s in a design flood. Required levee height is, therefore, about 0.13 m. Some freeboard would need to be added to this.

It will be necessary in some of the hydraulic computations of Step 7 to find the design flood flow rate at various locations in developed catchments.

Alternatively, an approximation for these flows may be calculated by simple proportion using information from Table B.3 and Table B.4. Flows at all node sections (Figure B.3) can be computed in the same way catchment floodway discharges (approximate) are calculated in Table B.5.

Where flow at a location other than a node section is required, for example, the flow moving west towards node point M.5 in Figure B.2, then the following procedure may be used:

- flood path description: single-channel , 7.5 carriageway longitudinal slope, S.= 02
- upstream contributing area: 1.40 ha
- upstream contributing (C x A) : 0.81 ha
- capacity flow, Q_c for flood path: 780 L/s (from Table B.5)
- tributary (impervious) area for flood path: 3.44 ha
- required design discharge (approximate): $\frac{0.81}{3.44} x 780 = 184 \ L/s$

Values computed by this method tend to underestimate required f1ows by about 10 per cent.

Further discussion of these matters cannot be justified in the present publication. Suffice to conclude that with careful detailing, thoughtful planning and the use of modest flood-proofing measures, it is possible to handle major flood flows generated in small, urban catchments without damage indoors and without interrupting the supply of essential community services or the functioning of strategic installations.

B.2.9 Step 8: Final design detailing

The outcome of Steps 6 and 7 is a series of design and planning decisions and component detail computations which will, when realised in the field, enable the goals of a major drainage system to be achieved. Step 8 involves the process of committing those decisions and detailing to paper in the form of drawings for a works program.

B.2.10 Contributions from and to other catchments

Where runoff from an adjacent catchment, or runoff is channelled to an adjacent catchment, runoff management planning and design problems are significantly more complex than those presented by the above example. Typically problems consist of:

- How can peaks flow be reduced?
- How can peaks flow be delayed?

Solutions available to the designer fall into three broad categories:

- flow retention measures
- flow detention measures
- flow retardation measures.

A discussion of these topics and a review of their role in urban storm runoff management are included in Chapter 7 – Culverts and energy dissipaters and Chapter 8 – Water quality and erosion.

B.2.11 Concluding Comments and Summary

Validity of the procedure

The design procedure used in the example above incorporates some simplifications that may attract criticism on the grounds of their violating recognised hydrological principles. Most significant among these is the use of a ‘blanket’ design storm duration to arrive at a first approximation major drainage system. The same simplification is also used in revision calculations, where these are needed, in catchments (Step 6) and in sub-areas (Step 7).

Strict adherence to the conventional Rational Method requires that a series of design storms be applied at successive node sections down the main drainage path commencing at the highest. At each node a design storm of duration equal to contributing catchment travel time should be used to fix average storm intensity and to compute design flood magnitude at that location.

Such an analysis leads to stormwater node section flows higher than those given by the ‘blanket’ design storm procedure. Computed disparity between the values is greatest in the uppermost sub-areas of each catchment tapering to insignificant near its flood disposal point.

Reference to uppermost sub-areas in the catchments (Table B.5) reveals flood escape path capacities, represented by tributary (impervious) areas, which are well in excess of demand (see Column 10 in Table B.5). Strict hydrological analysis requiring significantly increased manual or computing effort would undoubtedly reduce this gap. Nevertheless, and as a general rule, major system surcharge in an upper catchment sub-area is unlikely where the tributary (impervious) area of its flood path is greater than twice the equivalent impervious area being served.

Summary

A 'broad brush' procedure for designing major stormwater drainage systems for small urban catchments has been described. The resulting system uses only the roadway reserves and floodway easements of the urban landscape to contain runoff resulting from major storms up to and including that having a design average recurrence interval of 100 years (approximately). The procedure takes account of flow conveyed in underground pipes of minor system networks and includes an arbitrary, optional allowance for part malfunction of the minor system network (50 per cent blockage). Design for zero blockage follows the same procedure but employs different values for gap flow, Q_{gap}, and design ARI.

The procedure enables trial flood escape networks to be rapidly assessed and focuses on those segments of the flood path which require particular attention, e.g. special hydraulic design, building restrictions, road layout modifications, etc.

The procedure assumes that, in general, kerb side flow depths up to 0.20 m i.e. 0.050 m above kerb, can be tolerated and incorporates a flood path maximum depth / velocity constraint which ensures the safety of pedestrian and wheeled traffic during major flood events.

APPENDIX C

C.1 Worked Example of Estimating Ground Water Cross Pavement Subsurface Drains

The problem is to estimate the optimum drain spacing and depth necessary to lower a water table by 0.7 m. The soil is a silty clay. The drains are situated 4 m above bedrock and are 1 m below the water table. It is desired to lower the water table 0.7 m between the drains within a week. Using the definitions outlined in Figure C.1 and Equation (5.1) determine the drain spacing.

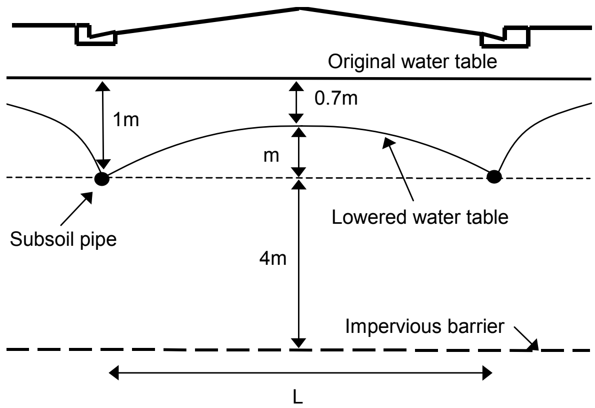


Figure C.1: Diagram of problem

saturated permeability (m/s) k	= 1 x10^{-6} m/s (typical value for silty clays)
depth of pipe below original water table (m), m_0	= 1 m
depth of pipe below lowered water-table (m), m	= 1 – 0.7 = 0.3 m
distance water-table is lowered (m), m_0- m	= 0.7 m
height of pipe above impervious barrier (m), d	= 4 m
drainable pore space, f	= 0.03 (typical value for silty clays)
time to lower water table (sec), t	= 7 days = 604,800 seconds
equivalent depth, d_e	= 0.9 m
$d_e/(d_e+m_0)$ = 0.9/(0.9+1)	= 0.47
geometrical factor ‘j’ determined from Figure 5.10	= 0.82

Distance between pipes, L (m) (as a first guess, assume a spacing of 10 m.)

From Figure 5.9 with L = 10 m and d = 4 m, then the equivalent depth can be estimated.

L $= 3 \times 0.82\left(\frac{(1(0.9+1)(0.9+0.3)604{,}800)}{2 \times 0.03(1-0.3)}\right)^{0.5} = 14.1\text{m}$

Second iteration, try L = 19 m

d_e = 1.4 m (from Figure 5.9)

$d_e/(d_e+m_0)$ = 0.58

j = 0.77 (j determined from Figure 5.10**)**

L = 17.71 m which is close enough to estimate.

Solution: Design sub-surface drainage system with pipes spaced no more than 19 m apart to lower water table by 0.7 m.

C.2 Worked Example of Draining an Inclined Aquifer

A trench 300 mm wide is cut to intercept an inclined aquifer that is 1m deep. The aquifer has a permeability of 10μm/s and is inclined at a slope of 10%. What is the discharge from the aquifer? What is the minimum permeability of the filter material required ensuring all infiltrated water is intercepted?

k_a = 10μm/s (=10 x10^{-6} m/s)

S = 0.1% (=0.1)

T = 1 m

W = 0.3 m

Using Equation (5.3), discharge is estimated at:

Q = 10 x10^{-6} x 1 x 0.1 = 1 x 10^{-6} m^3/s = 1 mL/s per m length

The minimum permeability of the filter material in trench can be estimated using Equation (5.4) where {tan(B)/tan(A)} can be approximated by W/T. Rearranging equation (5.4) to solve for the permeability of the filter material (k_f), we find:

$k_f = k_a$ x T / (W x S) = 10 x10^{-6} x 1 / (0.3 x 0.1) = 3.33 x10^{-4} m/s (say 330 μm/s)

Therefore the trench backfill should be sand or a gravel (see Table 8.4 for typical permeabilities).

C.3 Worked example of capillary rise

A road is to be built across an area of high water table. The road is be built on a raised formation 1 metre above the water table. The embankment material is a relatively coarse and free draining material.

D_{10} from the grading curve is about 0. 17 cm (i.e. only 10% of material is finer than 0. 17 cm)

e is estimated to be about 0.4 at best and 0.25 at worst

C the shape factor can vary from 0.1 (spheres, best case scenario) to 0.5 (angular, worst case scenario).

Capillary rise = $10C/eD_{10}$

▪ Best case scenario	=	10 x 0.1/(0.4 x 0.017)	=	147 mm
▪ Worst case scenario	=	10 x 0.5/(0.25 x 0.017)	=	1176 mm

Solution: Provided the embankment consists of a layer of the free draining material over 1200 mm thick, then the capillary rise should not be an issue.

APPENDIX D

D.1 Worked Example of Flow Width in Kerb and Channel

Determine the maximum spread of flow for a kerb and channel with the following properties:

Kerb height	**0.15 m**	
Channel width	**0.3 m**	
Channel cross slope	**1 in 8**	**Using Equation (**
Pavement cross slope	**1 in 30**	
Maximum spread to top of kerb	**= 0.3 + (0.15 - 0.3/8) x 30 = 3.675 m**	

D.2 Worked Example of Surface Flow Depths

For a superelevation development on a freeway, the length of flow has been found to be 45 m and the average slope is 0.008 m/m. The surface will be a dense graded asphalt with a texture depth (TXD) of 0.9 mm. The one year ARI, 5 minute intensity is 47 mm/h. Calculate the maximum depth of flow.

The 1 year ARI is less then 50 mm/hr therefore adopt 50 mm/hr.

$$\mathbf{d_{max}} = \frac{0.0149 \times (0.9)^{0.11} \times 45^{0.43} \times 50^{0.59}}{0.006^{0.42}} - (0.9) \text{ mm}$$

Using Equation (6.2)

d_{max} = 4.9 mm

d_{max} < allowable maximum of 5 mm (Table 6.8)

Conclusion: This depth is less than the allowable depth and is acceptable.

D.3 Worked Example of Flow in an Open Channel

A grassed catch drain has a bottom width of 2.0 m and side slopes of 2 to 1. The longitudinal gradient is 0.70 per cent. Manning's 'n' for the grass covering is 0.030. Calculate the capacity of the drain if the depth of flow is 0.5 metres.

Cross Sectional Area, A	= (0.5 x 2.0) + 2 (0.5 x 1.0 x 0.5)	= 1.5 m^2
Wetted Perimeter, P	= 2.0 + 2 √(1.02 + 0.52)	= 4.48 m
Hydraulic Radius, R	= A/P = 1.5/4.24	= 0.335 m

Velocity, V

$$= \frac{(0.335)^{0.667}(0.007)^{0.5}}{0.03} = \mathbf{1.34\ m/s}$$

Using Equation (6.3)

Estimated Discharge, Q = VA

= 1.34 x 1.5 = 2.02 m^3/s **Using Equation (6.4)**

D.4 Worked Example of Flow in an Open V Shaped Channel

A V shaped grassed catch drain has side slopes of two to one. The longitudinal gradient is 0.7 per cent. Manning's 'n' for the grass covering is 0.030. Calculate the capacity of the drain if the depth of flow is 0.5 metres. Izzard's formula (Equation 6.5) is applicable to V shaped channels.

Reciprocal of Side Slope, Z = 1/(0.500 m/m) = 2.0

Manning's n = 0.030

Longitudinal Gradient, S = 0.007 m/m

Maximum Water Depth, d = 0.5 m

Shape factor, β = 2.0

Estimated Discharge, Q $= \dfrac{375 \times 2.0 \times (0.007)^{0.5} (0.5)^{2.667}}{0.030} \times \dfrac{1}{1000}$ m^3/s

= 25.0 x 0.0837 x 0.157 = 0.33 m^3/s

For calculations of flow in streams of irregular cross section, refer to Austroads (1994).

D.5 Worked Example of Kerb and Channel Capacity

The roadway forms to be used have 7.5 m carriageway within 16.0 m roadway reserve. Roadway reserve data:

- Manning's n for concrete kerb and channel = 0.012 (n_a)
- Manning's n for asphalt laneway = 0.014 (n_b)
- kerb and channel, 0.375 m width with 0.1 m vertical depth
- cross slope of channel is 1 to 8 (Z_a)
- pavement cross fall of 1 to 30 (Z_b)
- longitudinal slope of 1 to 50 (S_o = 0.02 m/m), and
- shape factor is 0.8 (β).

Therefore, for a fixed kerb and channel type, we can write the modified Izzard's formula (see equation 6.6) for composite kerb and channels as:

$$Q_t = KS_o^{0.5}$$

where K is a constant related to a particular kerb and channel configuration. The maximum depth allowable is 50 mm above top of kerb, therefore, d_a is equal to 150 mm.

For d_a = 0.15 m, d_b = 0.15 to 0.375/8 = 0.153

$$375\beta\left(\frac{Z_a}{n_a}\left\{d_a^{8/3} - d_b^{8/3}\right\} + \frac{Z_b}{n_b} d_b^{8/3}\right)$$ See Equation (6.6)

K = 375 x 0.8(8/0.012[$0.15^{2.67}$ to $0.153^{2.67}$] + 30/0.014 x $0.153^{2.67}$)

= 5708

Kerb and channel capacity = 5708 x $0.02^{0.5}$ = 807 L/s

Similarly, using the K factor, it is possible to tabulate the capacities of the kerb and channel for a range of longitudinal slopes. However, flows must meet the two criteria (see Section 6.4.1.5):

- spread no wider than 2.5 m
- depth times average velocity to be less than 0.4 m^2/s

These criteria effectively limit the flows that can be carried particularly as longitudinal slopes increase. The two criteria would be calculated and where necessary the capacity flow would then be back calculated to comply. To compensate for temporary storage in the channels and pipes, the capacity flows can be increased by 10% (see Section 4.2.2 — Fixing of roadway reserve capacity 'flows') to give the storage correct capacity.

D.6 Worked Example of Side Entry Pit Spacing

Compare the channel capacity to the side entry pit capacity (SEP, 1m in width) on a divided road, each carriageway 10.5 metre between kerbs, paved median 6 m wide, footway 3.0 m wide. All lanes are through lanes. Longitudinal gradient 1.5%, crossfall 3%, Trial pit spacing 100 m. Design ARI is 10 years, and 5 minute intensity is 103 mm/h.

Contributing Area = (10.5 + 3 + 3) x 100/10000 = 0.165 ha

10 year ARI discharge = 0.9 x 103 x 0.165/360 = 0.042 m^3/s

Enter Figure 6.13 at channel grade 1.5%, proceed vertically:

Channel capacity for maximum width of 1.5 metres is about 0.026 m^3/s

SEP capacity for 95% capture is about 0.042 m^3/s with a corresponding width in channel of between 1.5 m and 2 m.

Conclusion: The SEP has a capacity to cater for the expected runoff. At 100 m pit spacing, the encroachment into the lanes would not meet the requirement to limit flow width to 1.5 metres. Try 60 metre spacing.

D.7 Worked Example of the Design of Minor Network System

D.7.1 Introduction

The following discussion and diagrams present, in schematic form, an ordered account of the main computational and graphical components of the minor drainage system design procedure. The procedure is applied to Catchment M, a 12 ha portion of the 38 ha residential sub-division featured in the major design example (see Figure B.2). The minor system network for this catchment includes a branching main drain pipeline and four lateral pipelines.

Although design of the minor system network using an ARI of two years would complement the major system design already prepared, the present illustration adopts an ARI of five years. This level of design ARI is employed solely to generate a wider range of design problems. This better demonstrates the scope of the procedure that would not be revealed by a design based on an ARI of two years level.

The catchment area is bordered on three sides (north, east and west) by main roads. A green belt reserve forms its southern boundary. The catchment topography slopes generally to the south making this the natural drainage direction.

Housing density in the sub-division will be, initially, 16 residences per ha of dedicated area i.e. excluding roadway reserves etc. Ultimate development during the life of the drainage system is estimated to be equivalent to 20 residences per ha. Some open space park areas are located within the subdivision.

It is assumed for purposes of illustration, that runoff from individual residences in the ultimately developed catchment will be channelled directly to the surface stormwater drainage system. Drainage from positive grade allotments will be conveyed directly to fronting roadways and drainage from adverse grade allotments will be channelled to the rear of the allotments.

All roads will have sealed pavement carriageways with concrete kerb-and-gutter borders. It is anticipated that underground pipes will be used in the minor drainage system for the catchment.

The Partial Rational Method is used to calculate runoff volumes. Throughout the example in the tabulated data, each column is provided with a unique number to assist in identifying accompanying notes.

D.7.2 General Overview Of Minor System Design Procedure

Scope

There are important procedural differences which distinguish the approach used in major system planning and that presented here in connection with the design of minor drainage systems. The planning procedure described for the major system was essentially a 'broad brush' operation concerned with the maintenance of order in the face of potentially devastating flood flows. Great precision in predicting flow behaviour in such circumstances is neither possible nor warranted. The aim of the major system planner is to provide a scheme which, given the great unpredictability of rare storm events, should operate at least 'satisfactorily'.

The minor system, on the other hand, is expected to contain and, indeed, control a prescribed level of flooding. Its design therefore calls for a higher level of predictability than is required of the major system. The narrower spectrum of events with which the drainage designer must contend, however, makes this task achievable for nuisance or frequent storms.

The procedure by which a minor system is designed may be divided into three broad phases:

Phase I Catchment definition and design guidelines;

Phase II Flow estimation and distribution; and

Phase III Surface system and underground network design.

The aim of this section is to state the principles and briefly describe the various steps that must be followed in the design of a minor stormwater drainage system. The system is to cope with a storm event with an average recurrence interval of N years for a commercial, industrial, residential or mixed development sub-division. Following this outline of procedure, guidelines and hydrological / hydraulic information is presented to design a minor stormwater drainage system.

Design Procedure - Phase I

The primary data used in the design of a minor stormwater drainage system involve three aspects:

- catchment definition
- design guidelines for surface moving flow management
- design guidelines for underground moving flow management.

The first task requires information on the physical nature and properties of the catchment, including, in particular, the location of existing underground services. It is similar to, though more detailed than, that required of the major system planning / design process.

Surface moving flows are the most obvious manifestation of storm runoff in a developed catchment. Criteria to control their impact on members of the general public both as pedestrians and as users of vehicles must be adopted.

All aspects of flow estimation and design that follow rest on the guideline information presented in Phase I.

Design Procedure - Phase II

With the catchment defined and design guidelines identified, the procedure addresses the task of preparing a valid hydrological model which satisfactorily represents the hydrological response of the catchment to runoff.

The catchment hydrological model forms the basis for all later stages in the design of a minor drainage system. It must describe its components (ultimately developed) and yield a satisfactory representation of its runoff response to rainfall input in all design storms of ARI = N years and smaller.

Design storm rainfall applied to this model by way of an adopted rainfall / runoff mathematical procedure yields the required flow estimate(s) at each catchment reference point. The rainfall / runoff mathematical model used with the catchment hydrological model must be a valid or acceptable procedure which yields the greatest runoff estimate for each component drainage unit, considered individually, for design storms of ARI = N years and smaller.

The designer needs to estimate the design flows resulting from ARI = N years storms in, firstly, all primary drainage lines. With this information and the guidelines on surface-moving flows, it is possible to select and locate gutter inlet and concentrated flow entry structures (i.e. pits) needed to transfer surface moving stormwater to the underground network or surface channels.

Drainage designers must base their selection and placement of components so that any primary drainage line including its inlet(s) allowing bypass, terminal inlet, concentrated flow entries, branch and lateral underground pipelines, etc., convey and discharge the peak flow generated in its catchment critical design storm having ARI = N years.

The same flow estimation concept is also applied in the case of the catchment or sub-catchment main drain pipeline or 'mainline'. The on-line components of any main drain pipeline, including its junction pits, node pits, pipes, etc., should convey and discharge peak flows generated at successive node pits by design storms that are critical in the catchments contributing to those stations.

The completion of Phase II of the procedure provides the designer with a clear picture of how and where the stormwater runoff flows generated in a catchment are disbursed in either the surface drainage system or its underground counterpart.

Operations in the procedure outlined above are carried out for each developed landscape drainage unit. These may be small catchments, which have no runoff into or from adjacent catchments, or sub-catchments of complex urban landscapes. In the latter case, Phase II must include a final step in which the interaction of flows between sub-catchment disposal points towards their (catchment) central disposal point must also be considered.

Design Procedure - Phase III

Phase II encompasses the flow estimation / distribution segment of the total procedure. It also includes some elements (surface-to-under-ground component selection) which, strictly, fall into the category of 'design'. Identification of these components forms the starting point for Phase III. Phase III involves the application of guidelines relating to the management of surface and underground moving stormwater flows to design an 'approximate' drainage network. Approximate head loss values estimates are employed in the development of this design which concludes with a catchment Hydraulic Grade Line (pit water levels) being declared, satisfying N years design ARI requirements.

This network is combined, in each case, with the selection of surface drainage component made in Phase II to provide an approximate design for the minor stormwater drainage system.

The final part of the procedure involves detailed modification of the approximate system design, as required, in the light of site constraints forced on the design by service locations, cover requirements, need for additional inspection pits, etc. In certain cases, final design detailing may call for a re-working of the Hydraulic Grade Line computations using accurate pipe and pit head loss values, some significant changes to the approximate network may ensue in these circumstances.

D.7.3 Minor System Design Procedure

Introduction

The procedure outline that follows has been prepared for use in the design of stormwater systems for small urban catchments e.g. the L, M, N and P catchments of Figure B.2, and for interconnected sub-catchments of large catchments e.g. the drainage units of catchments N and P in Figure B.3. The latter category represents, of course, the more general case. Underground pipe systems are used throughout the catchments with no surface channels in the minor system.

The basic drainage unit used in the following outline is therefore referred to as a 'sub-catchment' in the interests of generality and interaction between it and its fellows reviewed at the appropriate stage.

Where the outline is to be interpreted for isolated urban catchment cases, 'sub-catchment' should be replaced by 'catchment' and all references to sub-catchment interaction disregarded.

The three phases into which the minor system design procedure is divided may be sub-divided into a set of eleven main steps (see Table D.1). These include Step I0A, an optional cost / frequency step, which need only be employed by designers seeking comparative cost data for networks resulting from design ARIs covering a range of values of N (see Section 2.6).

Table D.1: Outline of eleven step design procedure

Step	Activity	Phase
1	Sub-catchment definition	I
2	Design guidelines and data (Surface-moving flows)	
3	Design guidelines and data (underground-moving data)	
4	Hydrological model to stage 1	II
5	Hydrological model to stage 2	
6	Design flow distribution in primary and main drainage lines	
7	Design flow compilation to all components of sub-catchment	
8	Pit water levels and ‘first-round’ pipe sizes	III
9	H.G.L. and pipes of approximate network design	
10	Approximate system design to ARI = N years	
10a	Cost / frequency insert)	
11	Final design detailing	

Step 1: Sub-catchment definition

This involves listing information required to plan a major drainage system for the sub-catchment, plus the following:

- location of sub-catchment stormwater disposal point and terminal node pit, if different
- identification of design flood level at sub-catchment stormwater disposal point
- adoption of a network of main drain pipeline(s) and associated node pits taking account of the presence of major services and other underground installations
- recording of gutter invert levels at roadway sections containing node pits
- definition of sub-areas draining to identified node pits
- identification of primary drainage line contributing areas and associated terminal gutter inlets
- identification of the locations of all significant pedestrian crossings.

Identification of the following as matters of policy:

- value for N years design ARI
- types of 'preferred inlets' and hydraulic characteristics of these
- acceptable or non-acceptable cross flows at minor street intersections
- pipes: concrete or FRC, normal condition or poor condition.

Measure, extract or classify and record the following for all components contributing runoff to primary drainage lines:

- area
- land use
- 'distributed' or 'concentrated' flow entry
- drainage line characteristics (type, length, slope, etc.)

- allowable or capacity flow at terminal inlets.

Almost all of items (a) to (f) inclusive relate to drainage network structure. The following definitions and explanations are needed in order to proceed with Step 1.

Component identification code: Node pits in the minor drainage system network use the same system of identifiers that are applied in a major system. A supplementary coding system is required to enable components other than node pits to be identified. The system set out in Figure D.1 is used throughout this example.

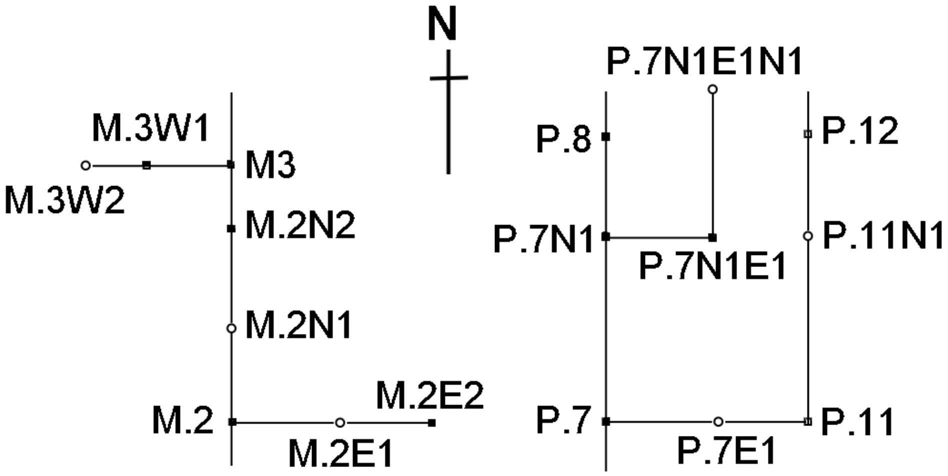


Figure D. 1: Identification system for minor drainage system (after ARRB Special Report 34, 1986)

Commencing from any node pit on the main drain pipeline, subsidiary components, inlets, inspection pits, etc. are numbered in the cardinal directions N, S, E and W along all main and lateral pipelines. Simple sub-catchments draw on only one or two of these directions, but the code is capable of extension to provide systematic identification in networks of great complexity.

'Distributed' and 'concentrated' flow entries: Distributed flow originates on sub-catchment areas such as roadways, unchannelled open space areas and residential or commercial subdivisions in which numerous small units discharge stormwater individually. Flows classified as concentrated come from subsidiary or allotment drains that collect runoff from (usually) large area components, e.g. residential sub-division, drive-in cinema, car park, etc. Stormwater from such areas enters the system as concentrated input to the main surface drainage line or is passed to the underground network via a sump and / or junction pit.

Allowable or capacity flow at a terminal gutter inlet can only be found after surface-moving flow management guidelines have been adopted in Step 2. This item relates to the hydraulic characteristics of the catchment roads and streets [Item (e), Step 1] and preferred inlets [Item (h) above].

Data for example

- Location of sub-catchment stormwater disposal point and terminal node pit is at M.0 (see Figure D.2).
- Design flood level at sub-catchment stormwater disposal point is RL (reduced level) 100.0 m.
- A network of main drain pipeline(s) and associated node pits accounting for the presence of major services and other underground installations (see Figure D.2).
- Gutter invert levels at roadway sections containing node pits are listed in Table D.2.

- Sub-areas draining to identified node pits, (see Figure D.3).
- identify primary drainage line contributing areas and associated terminal gutter inlets, (see Figure D.3).
- The locations of all significant pedestrian crossings (none in this example).

The following apply to this example:

- the design ARI is five years (typically a subdivisions of this type would normally only require an ARI of two years (see Tables 6.3 & 6.5)
- side entry inlet pits of 1 m or 2 m width without deflectors are preferred
- cross flow is not acceptable at minor street intersections; and
- FRC pipes in poor condition are assumed.

The following information for all components contributing runoff to primary drainage lines is recorded in Table D.2:

- area
- land use
- 'distributed' or 'concentrated' flow entry
- drainage line characteristics (type, length, slope, etc.)
- allowable or capacity flow at terminal inlets.

Almost all of items (a) to (f) inclusive relate to drainage network structure. The following definitions and explanations are needed in order to proceed with Step 1.

The drainage network for the example sub-catchment is represented by the schematic layout shown in Figure D.3. Included in Figure D.3 are:

- sub-areas catchments sizes
- road reserve information
- slope
- main node pits.

Figure D.4 shows greater detail of the proposed drainage network and includes:

- component identification codes; and
- Distances between pits.

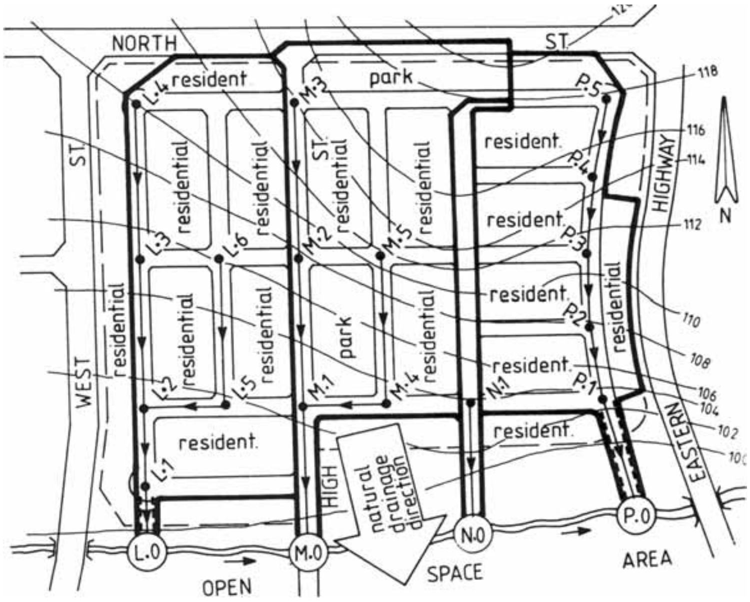


Figure D.2: Overall layout of a small subdivision located in the Adelaide Hills (after ARRB Special Report 34, 1986)

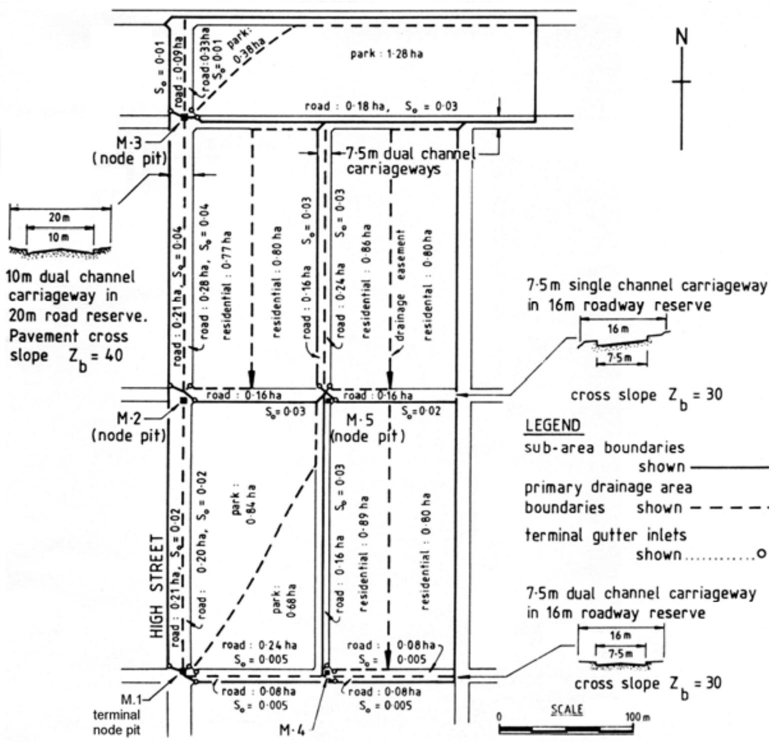


Figure D.3: The sub-catchment 'M' used in the example (after ARRB Special Report 34, 1986)

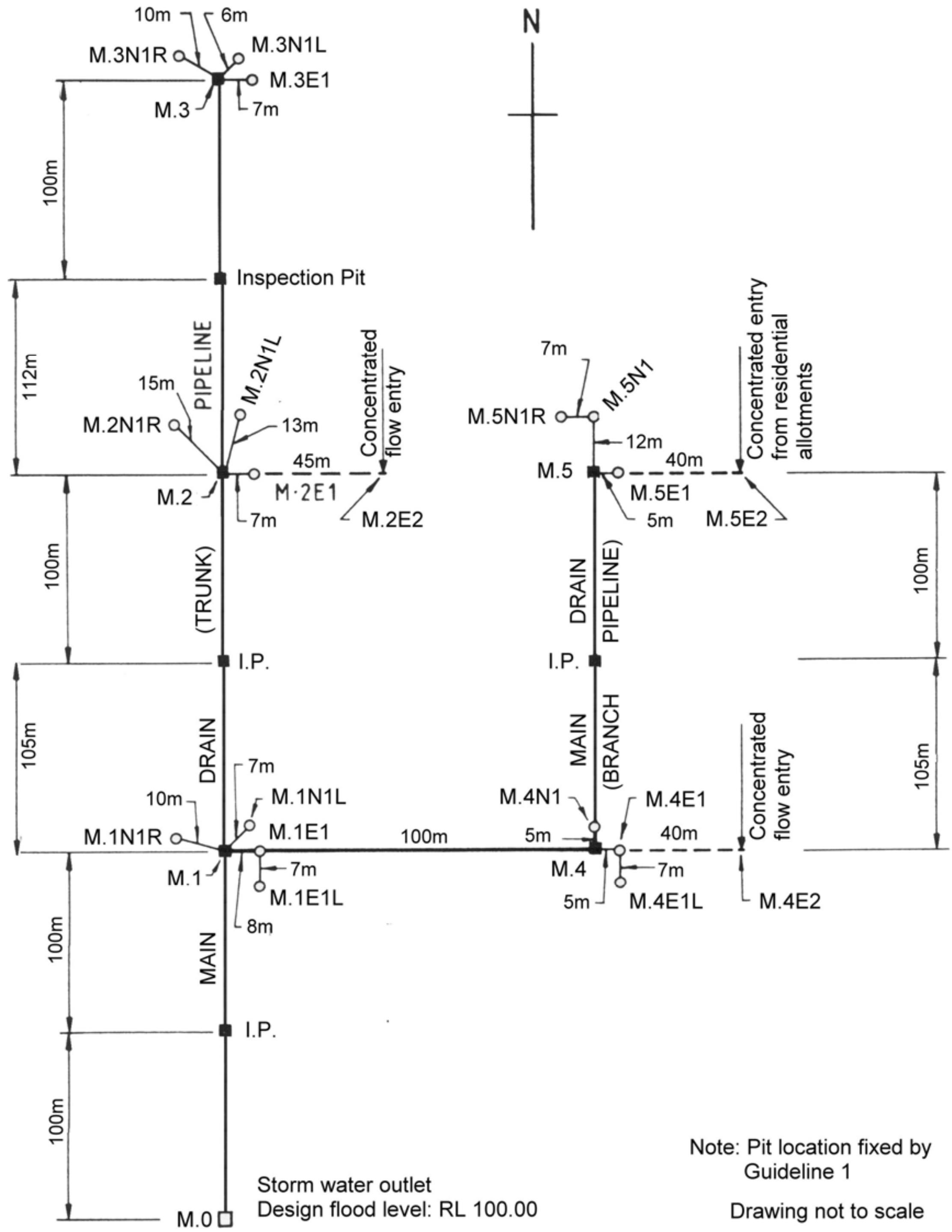


Figure D.4: Schematic layout of drainage network (after ARRB Special Report 34, 1986)

The data from Figure D.3 and Figure D.4 are tabulated in Table D.2. The following describes the values in each column:

Columns 1 to 7 Data related to the catchment.

Column 8 The contributing areas to each pit. No adjustments are made at this point for impervious or pervious areas.

Column 9 Identifies if the flow at the inlet is distributed or concentrated.

Columns 10 and 11 Data related to the catchment.

Column 12 Gives the length over which the bulk of contribution enters the drainage line.

Columns 13 and 14 Data related to the catchment.

Column 15 Estimates the gutter capacity that meets the spread criteria adopted (i.e. limit spread across road such that at least one lane remains open which implies that the spread in either gutter is equal to one quarter of the road width = 7.5/4 = 1.79 m). Using Izzard's equation, the gutter capacity can be determined for each slope and road / gutter configuration.

Column 16 provides comments such as, the calculated flow times are less than recommended minima in Table 4.1.

(See Table D.2 over page)

Table D.2: Catchment definition data

Catchment	Sub-catchment	Sub-area node pit	Entry point	Gutter invert level	Component description	Hydrological surface classification	Area or equivalent area	Type of contribution (conc. / distr.)	Drainage line conveying flow to terminal inlet: Total length	Fall	Effective length	Slope near terminal inlet	Carriagway cross slope	Capacity flow guideline	Remarks
							ha		m	m	m	m/m	%	L/s	
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
M		.5	node	110-30											
			NIR	110-65	7.5m road	paved 1st grade	0.12	D	200	4.5	200	0.03	30	65	
			N1	110-65	7.5m road	paved 1st grade	0.19	D	290	5.0	180	0.03	30	65	
					resid.	20res/ha	0.86	D	180	4.0					
			E1	110-40	7.5m road	paved 1st grade	0.12	D	90	2.0	90	0.02	30	76	see min. travel time (table)
					resid.	20res/ha	0.80	C	40	1.0					
M		.4	node	103-00											
			N1	103-15	7.5m road	paved 1st grade	0.13	D	200	7.0	185	0.03	30	65	
					resid.	20res/ha	0.89	D	185	6.0					
			E1	103-03	7.5m road	paved 1st grade	0.06	D	90	0.5	90	0.005	30	75	Min. travel time
					resid.	20res/ha	0.80	C	40	0.2					
			E1L	103-03	7.5m road	paved 1st grade	0.06	D	90	0.5	90	0.005	30	75	Min. travel time
M		.3	node	114-20											
			NIR	114-25	10m road	paved 1st grade	0.07	D	70	0.7	70	0.01	40	70	Min. travel time
			N1L	114-25	10m road	paved 1st grade	0.25	D	310	6.0	60	0.01	40	70	
					park	pervious	0.38	D	0						
			E1	110-40	7.5m road	paved 1st grade	0.14	D	250	4.0	250	0.03	30	65	
					park	pervious	1.28	D	0						
M		.2	node	107-30											
			NIR	107-80	10m road	paved 1st grade	0.15	D	200	5.5	200	0.04	40	65	
			N1L	107-80	10m road	paved 1st grade	0.21	D	280	8.0	180	0.04	40	65	
					resid.	20res/ha	0.77	D	180	5.5					
			E1	107-50	7.5m road	paved 1st grade	0.12	D	90	2.0	90	0.03	30	65	Min. travel time
					resid.	20res/ha	0.80	C	45	1.0					
M		.1	node	102-20											
			NIR	102-30	10m road	paved 1st grade	0.15	D	200	4.5	200	0.02	40	76	
			N1L	102-30	10m road	paved 1st grade	0.02	D	200	4.5	200	0.02	40	76	
					park	pervious	0.84	D	0						
			E1	102-25	7.5m road	paved 1st grade	0.15	D	290	7.0	90	0.005	30	75	
					park	pervious	0.68	D	0						
			E1L	102-25	7.5m road	paved 1st grade	0.06	D	90	0.5	90	0.005	30	75	Min. travel time
M		.0	Flood disposal Point Design water level RL 100.00												

Table D.3: Outcomes of Step 1

Item	Description	Output
1.	Contour map of sub-catchment, boundaries, node pits, terminal gutter inlets, etc	Figure D.2
2.	Sub-catchment underground network (Stage 1) gutter inlets, dimensions, etc	Figure D.4
3.	Table of sub-catchment properties	Table D.2

Step 2: Design guidelines and data (surface-moving flows)

A set of guidelines and associated hydraulic information and data must be adopted for the management of runoff moving in the surface channels of minor stormwater drainage systems. The guidelines are discussed under Appendix D.7.3 and flows must conform to the criteria set out in Table 6.5.

Data for example

In this example, the criteria in Table 6.5 were adopted to limit the spread to runoff so that at least one lane width on residential streets should be trafficable during a five year ARI storm (normally for this type of development a two year ARI would be applicable, see Appendix D.7.1). No bus stops or pedestrian crossing zones were identified in sub-catchment 'M'.

Table D.4: Outcomes of Step 2

Description	Output
Adopt guidelines in Table 6.5	Table 6.5

Step 3: Design guidelines and data (underground flows)

The task of managing underground moving flows within an urban minor stormwater drainage system involves the design of a network where pipe sizes and locations are governed by various practical and geometrical constraints. The junction pit water levels are fixed by associated gutter and roadway finished levels. The drainage designer must formulate, or be provided with, a set of guidelines similar to those listed in Tables 6.1, 6.11 and Table D.5. These guidelines should not be regarded as mandatory for use throughout Australian practice.

Table D.5: Guidelines for surface drains and underground pipe networks (see Sections 6.3.2 & 6.3.6)

Guideline	Description
1	Main drain or lateral pipelines should be aligned as follows: **Minor roads** (carriageways less than 10 m): They should connect succeeding gutter inlet pits (both side-entry and grated inlet types) located along the drainage path. The alignment should favour the carriageway 'high side' in dual-channel roads and streets. 'Low side' inlets on dual-channel minor roads should be connected to the pipeline by either inter-inlet cross-connections or by deviating the alignment if necessary. The alignment should be just within the carriageway where inlets are grated and just outside where side-entry inlets are used. The alignment in single-channel minor roads should be on the 'low side' of the street. **Major roads** (carriageways 10 m or greater): Main drain or lateral pipelines should be located within carriageways between 1.5 m or 2.5 m from the 'high side' kerb. Gutter inlet pits should be cross connected to on-line junctions. The alignment in single-channel major roads should be on the 'low side' of the street.
2	Every effort should be made to space on-line junction pits as far apart as possible. Avoid the use of angled cross connections where possible.

Table D.5: Guidelines for surface drains and underground pipe networks *(Continued)*

3	In order to facilitate inspection and cleaning, the maximum recommended pit spacing are: — 200 m for pipe diameters > 1800 mm — 120 m for pipe diameters ≤ 1800 mm
4	In all gutter and 'sag' inlet pits and junction pits, design water levels assigned to pits should not be higher than the gutter invert level minus 0.15 m (where 'gutter invert level' means the undepressed gutter invert level at the roadway section containing the pit, or where invert levels differ, the lower of the two).
5	A 3-point priority sequence should be followed in assigning pit design water levels in accordance with the above: Priority 1: junction pits along lateral pipelines including the pits where these pipelines join with main drain pipelines. Priority 2: junction pits along main pipeline branches, where these are present. Priority 3 junction pits along the main pipeline trunk. In each priority, assignment of pit design water levels should commence at the upstream extremity in priorities 1 and 2 and at the downstream extremity priority 3.
6	A minimum pipe diameter of 300 mm should be used in the design.
7	As a practical design rule, pipe diameters should not decrease in the direction of flow.
8	In order to reduce the likelihood of blockage due to sedimentation, flow velocities in pipes operating under design conditions should be greater than 0.5 m/s. Flows which lead to violations of this limit should normally be excluded from the underground network.
9	The diameter of a cross connection may be selected from either Table D.6 or may be set equal to the diameter of the pipe conveying flow from the connected mainline or lateral pipeline junction pit, D_o, whichever is smaller. In cases where the tabulated diameter, $D_{t,}$ exceeds D_o by more than one increment, diameter D_o may be used in the cross connection provided pit head loss and water level conditions at either end are investigated for satisfactory performance.
10	Except of cases where D_t exceeds D_o by more than one pipe increment (Guideline 9), pit floor levels in cross connected inlet / junction pits should be set at or below (gutter invert level to 0.45 m to D_o). Pit floor level should coincide with the invert.
11	Pit floor levels in mainline or lateral pipeline junction pits receiving flow from cross connected inlets must allow for slope or not less than 0.01 m/m in cross-connection pipes. Pit floor level should coincide with invert of the pipe carrying discharge from a junction.
12	Underground stormwater network components should be specified by class in accordance with the technical information available from manufacturers and the provisions of AS 3725.

Table D.6: Recommended pipe diameters for cross connections (copy of Table 6.11)
(maximum length assumed to be less than 10 m, minimum slope to be not less than 0.01 m/m)

Flow in cross-connection L/s	Nominal pipe diameter, D_t mm	Flow in cross-connection L/s	Nominal pipe diameter, D_t mm
< 55	300	180 to 220	675
55 to 80	375	220 to 270	750
80 to 110	450	270 to 320	825
110 to 140	525	320 to 370	900
140 to 180	600	370 to 500	1050

Data for Example

This step defines criteria that are to be adopted in governing the movement of runoff throughout the sub-catchment in the drainage network. The guidelines in Table 6.5, Table D.5 and Table 6.11 were adopted which among other things suggest that no underground pipe should be no more than 120 m between pits (for maintenance purposes).

Table D.7: Outcome of Step 3

Description	Output
Adopt guidelines Table 6.5 or similar	Tables 6.4, 6.5; Tables D.7 and Table D.8

Adoption of these guidelines, taken with the database compiled in Step 1, completes Phase I and provides the foundation for all subsequent stages of the design procedure.

Steps 4 & 5: Hydrological models

By means of the procedure and guidelines reviewed in Phase I, it is possible to convert real-world sub-catchments into mathematical representations or 'hydrological models'. These models can be used to yield design flow estimates and flow distributions in all drainage path components.

It is assumed in the initial sub-catchment hydrological model (Figure D.5) that design flows generated in all primary (surface) drainage lines fall below the limits set by the guidelines in Table 6.5.

Primary drainage lines that are shown to conform to this assumed behaviour are called ***determinate*** drainage lines. Those that fail one or more of the guidelines, or include significant pedestrian crossings are termed ***indeterminate*** lines. Testing of the Stage 1 model (Figure D.5) to determine the classification of each primary drainage line is carried out in Step 4 and presented in Table D.9. The tasks involved are, for each primary drainage line and concentrated flow entry:

- compute design flow (N years design ARI), and,
- Compare design flow with guideline limit(s).

Where a drainage line includes one or more concentrated entries, the component flow from each is determined and compared with the Table D.8 Guideline 4 limit. Where the flows do not meet the criteria then the Stage 1 network layout (Figure D.4) needs to be adjusted until compliance is achieved.

Sizes and locations of gutter inlets of ***determinate*** drainage lines can be declared at this point.

Those drainage lines found to be ***indeterminate*** are modified by the addition of in-path gutter inlets or other means to enable them to satisfactorily convey and discharge their design flows without violating Table Guidelines 1 - 4 inclusive. The process by which the sizes and locations of additional inlets etc. are fixed is graphical, and carried out in Figure D.4. Such additions also affect change in the Stage 1 network layout.

Table D.8: Guidelines for the management of surface flows in developed urban catchments (minor system elements only)

Guideline	Description
1	Flow at 'sag' or terminal inlets and in roadside channels near intersections where bypass will cause cross flow which is unacceptable, must be: not greater than that giving a flow spread (from kerb line) of 2.50 m, and, Not greater than the 95 per cent capture approach flow of the favoured gutter / inlet.
2	Flow width at pedestrian zones to conform to those given in Table 6.5.
3	Roadside channel flow along surface drainage lines generally and at non-terminal inlets must be not greater than the 80 per cent capture approach flow of the favoured gutter / inlet and conform with Table 6.5.
4	A concentrated flow may be accepted into the roadside surface drainage line provided that: it is not greater than 20 L/s and, the accumulated channel flow at the concentrated flow outlet meets Guideline 3 above.
5	Where a concentrated flow does not meet Guideline 4 above, it must be passed to a junction pit.

Data Example

The hydrological model leads to the derivation of flows throughout the network. The sub-catchment in Figure D.4 and the data in Table D.2 are combined to prepare the hydrological model shown in Figure D.5. Estimation of the flow in the primary drainage lines is then carried out. The required data and results are summarised inTable D.9.

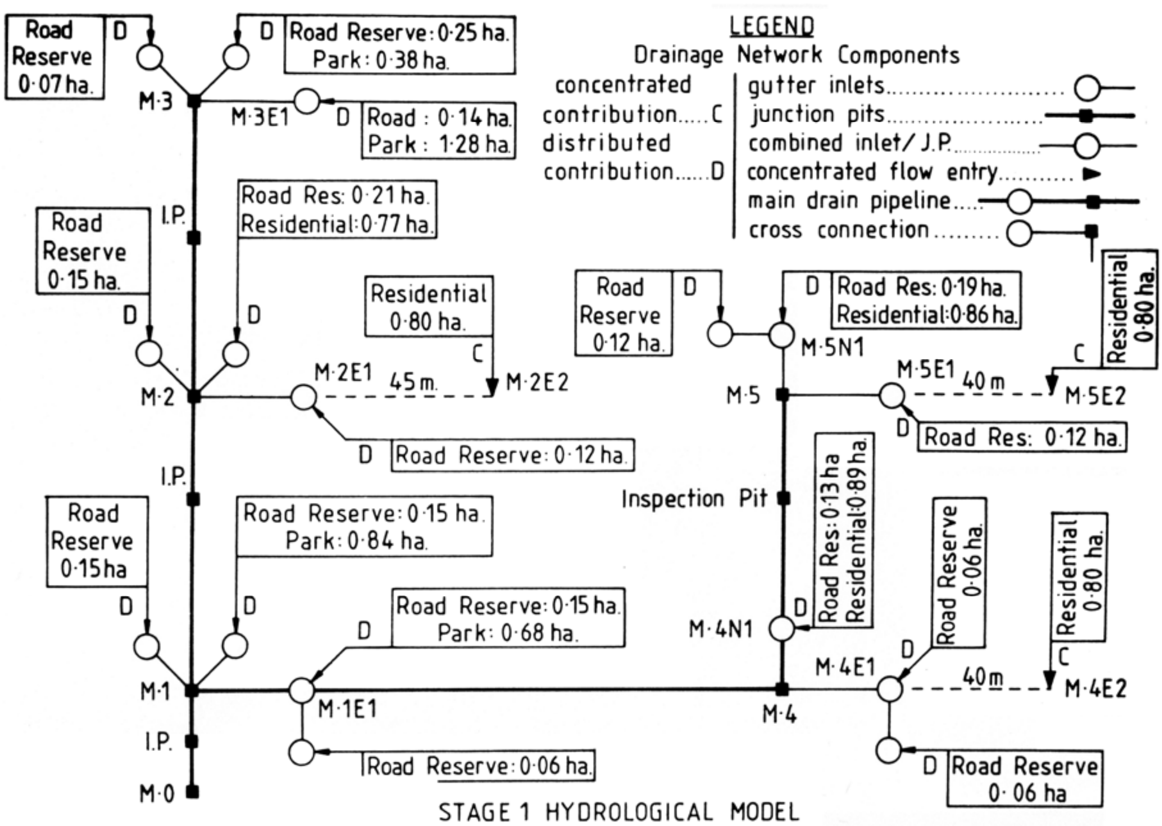


Figure D.5: Stage 1 hydrological representation of drainage scheme (after ARRB Special Report 34, 1986)

Columns 1 to 7 record data related to the catchment (similar toTable D.2).

Column 17 records the runoff travel time from roof to gutter or to allotment drain. In this example, the recommended time of five minutes has been adopted (see Section 4.4.2).

Columns 18 to 20 record data related to overland flow. The travel time from crown to gutter in a narrow carriageway is taken as two minutes which is the minimum overland flow time for any catchment element as specified in Section 4.4.2. For the wider (10 m) carriageways, the travel time is three minutes. For overland travel time in the residential catchment component, a standard 15 minutes is adopted.

Information regarding the overland flow distance and slope of the park can be derived from a topographical map of the area such as the one shown in Figure D.2. In the example, for the overland flow from the park into the terminal gutter inlets M.3N1L and M.3E1, the maximum flow distance and slope are given as 110 m and 0.025 m/m respectively. The flow travel time over the park is calculated using the kinematic wave formula (Equation 3.8).

Manning's roughness coefficient, n, for the park grassland can vary from 0.17 to 0.48 (see Appendix B). In this example, a median value of 0.32 is assumed. Correct use of the kinematic wave formula requires several iterations as the rainfall intensity is dependent on t_c and vice versa. Taking a conservative approach, the rainfall intensity for a storm duration of 5 minutes with a design ARI of five years is assumed. This yields a rainfall intensity of 76 mm/hr and overland travel time of approximately 32 minutes.

Columns 21 to 29 record data related to flow in channels. The approximate flow time may be obtained directly from Figure 5.4 — Flow travel time in channels. The time of entry for all carriageway components up to 200 m in length, regardless of slope, are taken as five minutes as specified under Section 4.4.2 for minimum (total) flow travel time.

Columns 30 and 31 record data related to full area and part area travel time. It is determined from using the appropriate travel times determined from columns 17, 20, 23, 26 and 29. The critical storm duration is then determined for each sub-area.

Columns 32 and 33 assign rainfall intensities for critical storm durations determined in columns 30 and 31.

Column 34 records the contributing areas flow to each pit. Information is identical to column 8 of Table D.2.

Column 35 assigns runoff coefficient to the different catchment surface types for the design ARI of five years. The runoff coefficient is calculated using the procedures outlined in Section 3.5.1.

Table D.9: Flow estimation in primary drainage lines

Catchment	Sub-catchment	Sub-area node pit	Primary drainage contributing area: Entry point	Component description	Hydrological surface classification	Time of Entry – Component travel time to minor element: Roof to gutter or to allotment drain	Overland Flow: Length	Overland Flow: Slope	Overland Flow: Time	Allotment Drain: Length	Allotment Drain: Fall	Allotment Drain: Time	Pipe or channel travel to entry point – Gutter or channel: Length	Gutter or channel: Fall	Gutter or channel: Time	Pipe or Channel: Length	Pipe or Channel: Fall	Pipe or Channel: Time	Compoent travel time to entry point: Full area t_c	Part Area t_i
						min.	m	m/m	min.	m	m/m	min.	m	m/m	min.	m	m/m	min.	min	min
1	2	3	4	6	7	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
M		.5	NIR	7.5m road	paved 1^{st} grade	-	-	-	2	-	-	-	200	4.5	3	-	-	-	5	5
			N1	7.5m road	paved 1^{st} grade	-	-	-	2	-	-	-	290	5	4	-	-	-	6	6
				resid.	20res/ha	5	15 mins			-	-	-	180	4	2	-	-	-	17	7
																		critical storm duration	17	7
			E1	7.5m road	paved 1^{st} grade	-------------------------- by inspection, 5 mins (see Section 5.5) --------------------------													5	5
				resid.	20res/ha	5	15 mins			180	3	3	-	-	-	40	1	1	19	9
M		.4	N1	7.5m road	paved 1^{st} grade	-	-	-	2	-	-	-	200	7	2	-	-	-	5	5
				resid.	20res/ha	5	15 mins			-	-	-	185	6	2	-	-	-	17	7
																		critical storm duration	17	7
			E1	7.5m road	paved 1^{st} grade	- - - - - - - - - - - - - - 5 mins - - - - - - - - - - - - - -													5	5
				resid.	20res/ha	5	15 mins			185	7	2	-	-	-	40	0.2	1	18	8
			E1L	7.5m road	paved 1^{st} grade	- - - - - - - - - - - - - - 5 mins - - - - - - - - - - - - - -													5	5
M		.3	NIR	10m road	paved 1^{st} grade	- - - - - - - - - - - - - - 5 mins - - - - - - - - - - - - - -													5	5
			N1L	10m road	paved 1^{st} grade	-	-	-	3	-	-	-	310	6	4	-	-	-	7	7
				park	pervious	-	110	2.5	32	-	-	-	-	-	-	-	-	-	32	7
																		critical storm duration	32	7
			E1	7.5m road	paved 1^{st} grade	-	-	-	2	-	-	-	250	4	4	-	-	-	6	6
				park	pervious	-	110	2.5	32	-	-	-	-	-	-	-	-	-	32	6
																		critical storm duration	32	6
M		.2	NIR	10m road	paved 1^{st} grade	-	-	-	3	-	-	-	200	5.5	2	-	-	-	5	5
			N1L	10m road	paved 1^{st} grade	-	-	-	3	-	-	-	280	8	3	-	-	-	6	6
				resid.	20res/ha	5	15 mins			-	-	-	180	5.5	2	-	-	-	17	7
																		critical storm duration	17	7
			E1	7.5m road	paved 1^{st} grade	- - - - - - - - - - - - - - 5 mins - - - - - - - - - - - - - -													5	5
				resid.	20res/ha	5	15 mins			180	5	2	-	-	-	45	1	1	18	8
M		.1	NIR	10m road	paved 1^{st} grade	-	-	-	3	-	-	-	200	4.5	3	-	-	-	6	6
			N1L	10m road	paved 1^{st} grade	-	-	-	3	-	-	-	200	4.5	3	-	-	-	6	6
				park	pervious	-	175	3	40	-	-	-	-	-	-	-	-	-	40	6
																		critical storm duration	40	6
			E1	7.5m road	paved 1^{st} grade	-	-	-	2	-	-	-	290	7	3	-	-	-	5	5
				park	pervious	-	175	3	40	-	-	-	-	-	-	-	-	-	40	5
																		critical storm duration	40	
			E1L	7.5m road	paved 1^{st} grade	- - - - - - - - - - - - - - 5 mins - - - - - - - - - - - - - -													5	5

Table D.9: Flow estimation in primary drainage lines *(Continued)*

Rainfall intensities		Area	Coefficient of Runoff	Component (CA)		Type of Contribution	Primary contributing Area to toal flow		Guideline Limits	Remarks
Full area	Part Area			Full area	Part Area		Full area	Part Area		
mm/h	mm/h	ha		ha	ha	D or C	L/s	L/s	L/s	
32	33	34	35	36	37	38	39	40	41	42
76	76	0.12	0.72	0.09	0.09	D	18.2	18.2	< 65	Determine M.5N1R
		0.19	0.72	0.14	0.14	D				
		0.86	0.52	0.45	0.45	D				
47	68	Total		0.58	0.58		76.2	110.3	> 65	Indeterminate
76	76	0.12	0.72	0.09	0.09	D	18.2	18.2	< 76	Determine M.5E1
39	63	0.8	0.52	0.42	0.42	C	45.1	72.8	> 20	To underground network
		0.13	0.72	0.09	0.09	D				
		0.89	0.52	0.46	0.46					
47	68		Total	0.56	0.56		72.6	105.1	> 65	Indeterminate
76	76	0.06	0.72	0.04	0.04	D	9.1	9.1	< 75	Determine M.4E1
46	66	0.8	0.52	0.42	0.42	C	53.2	76.3	> 20	To underground network
76	76	0.06	0.72	0.04	0.04	D	9.1	9.1	< 75	Determine M.4E1L
76	76	0.07	0.72	0.05	0.05	C	10.6	10.6	< 70	Determine M.3N1R
		0.25	0.72	0.18	0.18	D				
		0.38	0.11	0.04	0.04	D				
33	68		Total	0.22	0.22		20.3	41.9	< 70	Determine M.3N1L
		0.14	0.72	0.10	0.10	D				
		1.28	0.11	0.14	0.03	D				
33	72		Total	0.24	0.13		22.1	25.4	< 65	Determine M.3E1
76	76	0.15	0.72	0.11	0.11	D	22.8	22.8	< 65	Determine M.2N1R
		0.21	0.72	0.15	0.15	D				
		0.77	0.52	0.40	0.40					
47	68		Total	0.55	0.55		72.0	104.2	> 65	Indeterminate
76	76	0.12	0.72	0.09	0.09	D	18.2	18.2	> 66	Determine M.2E1
46	66	0.8	0.52	0.42	0.42	C	53.2	76.3	> 20	To underground network
72	72	0.15	0.72	0.11	0.11	D	21.6	21.6	< 76	Determine M.1N1R
		0.15	0.72	0.11	0.11	D				
		0.84	0.11	0.09	0.09	D				
29	72		Total	0.20	0.20		16.1	40.1	> 65	Determine M.1N1L
		0.15	0.72	0.11	0.11	D				
		0.68	0.11	0.07	0.07	D				
29	76		Total	0.18	0.18		14.7	38.6	<75	Determine M.1E1
76	76	0.06	0.72	0.04	0.04	D	9.1	9.1	<75	Determine M.1EL

The one hour duration storm with a ten year ARI for Adelaide is 26 mm/h (${}^{10}I_1$). The ${}^{10}I_{15min.}$ was found to be 59 mm/h. The design storm has an ARI of 5 years, therefore F_5 = 0.95 from Table 3.1. The variable, C^1_{10}, is the same for all areas in the catchment and is:

C^1_{10} = 0.1 + 0.0133 (${}^{10}I_1$ - 25)

C^1_{10} = 0.1 + 0.0133 (26 - 25) = 0.1133.

For the road reserve areas it is assumed that 85% of the area is impervious:

$C_{10} = 0.9 \times f + C^1_{10} \times (1 - f) = 0.9 \times 0.85 + 0.1133 \times (1 - 0.1133) = 0.78$

$C_Y = F_Y \times C_{10} = 0.95 \times 0.78 = 0.72$.

For the residential areas at a density of 20 dwellings per hectare:

$f = 3 \times RD - 5 = 3 \times 20 - 5 = 55\%$ of the area is impervious

$C_{10} = 0.9 \times f + C^1_{10} \times (1 - f) = 0.9 \times 0.55 + 0.1133 \times (1 - 0.55) = 0.55$

$C_Y = F_Y \times C_{10} = 0.95 \times 0.55 = 0.52$.

For the park areas, assume there are no impervious areas:

$C_{10} = 0.9 \times f + C^1_{10} \times (1 - f) = 0.9 \times 0 + 0.1133 \times (1 - 0) = 0.11$

$C_Y = F_Y \times C_{10} = 0.95 \times 0.11 = 0.11$.

Columns 36 and 37 give the component impervious area of the different catchment surface types. For the carriageway and residential area, the impervious area is similar for the full area and part area, which is the product of column 34 and 35. For the park area, the impervious full area is obtained in similar steps to the carriageway and residential area. For the park impervious part area, it is simply the fraction t_i/t_c (column 31 divided by column 30) of the impervious full area.

Column 38 identifies if the flow at the inlet is distributed or concentrated. It is identical to column 9 of Table D.2.

Columns 39 and 40 give the primary contributing area total flow at the nominated entry point for full-area and part area analyses. It is determined using the Rational method equation (Equation 3.9).

Column 41 gives the gutter capacity limits calculated earlier in column 15 of Table D.2.

Column 42 identifies which primary drainage lines conform to limits set by guidelines 1, 2 and 4 of Table D.3. Those that conform are called determinate whereas those that fail are termed indeterminate.

In Step 5 the modified hydrological model is presented in its final form, Hydrological Model - Stage 2 (see Figure D.6). This model is generally more complex than its predecessor, contains more information about the minor system drainage network and has proven ability to satisfactorily model design flows in all primary drainage paths within the constraints imposed by Table D.7 Guidelines 1 - 4 inclusive. This model forms the basis for all subsequent flow estimation computations.

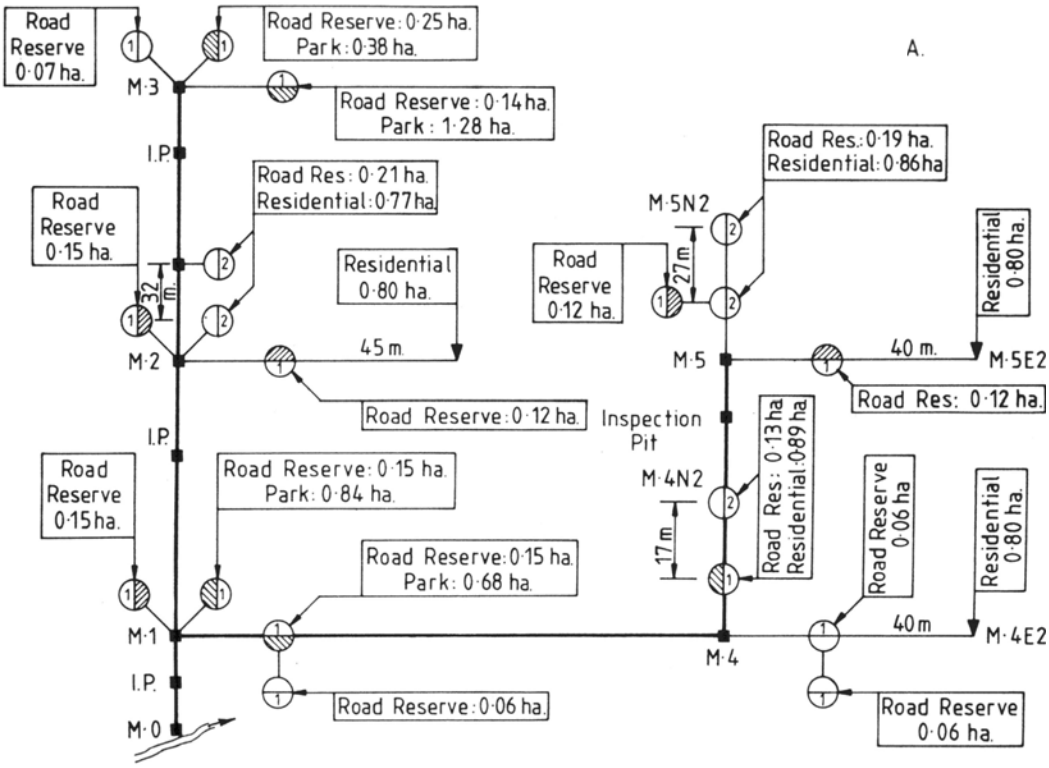


Figure D.6: Stage 2 hydrological representation of drainage scheme

Table D.10: Outcomes of Steps 4 and 5

Description	Output
Stage 1 hydrological model	Figure D.5
Tabular computation of primary drainage line design flows, comparisons, etc. Sizes and locations of gutter inlets in *determinate* drainage lines.	Table D.9
Graphical determination of gutter inlet positions, etc., in *indeterminate* drainage lines.	Figure D.9
Stage 2 hydrological model including information on gutter inlet locations, types, sizes, concentrated flow entries, etc.	Figure D.6

Step 6: Design flow distribution in primary and main drainage lines

The tasks executed in Steps 4 and 5 to derive a valid hydrological model for a given sub-catchment led to design flows being declared in all segments of its primary drainage lines. These involve both surface and underground flow paths, the latter being introduced as a consequence of the failure of some primary drainage lines investigated in Step 4 as wholly surface flow conveyors to meet adopted guideline limits. The primary drainage line design flows are presented in Figure D.7.

Emphasis in the procedure from Step 3 to this point is clearly on design flows generated in primary drainage lines by design storms that are critical in their individual catchment areas. Flows determined in this way lead to the selection and design of primary drainage line components.

Flows conveyed by the main pipeline arise from design storms that are critical in collections of primary drainage areas, the aggregate flows entering the mainline at or in the vicinities of its node pits. The catchment area contributing to successive node pits increases with distance down the main drain. Flows determined from these collections of primary drainage areas form the basis for the selection and design of main drain components.

To determine these design flows requires a set of computations similar to those of Table D.9 (Step 4) but reflecting the collective nature of the contributing catchment. Execution of these calculations requires the Stage 2 hydrological model to be interpreted as a system of catchment aggregations (see Figure D.8). The flow computations are carried out in Table D.11 and the results presented in Figure D.10.

Data Example

This step concerns the design of flow distribution in the primary and main drainage lines. Computations similar to those of Step 4 are carried out but reflect the collective nature of the contributing catchment (Figure D.7).

The network involves an extensive system of main and lateral pipelines. Its design must therefore consider 'total catchment' aspects of flow estimation. These aspects find expression in Figure D.8, Table D.11 and Figure D.10 of Step 6.

Discussion of Figure D.8

The interpretation of the Stage 2 Hydrological Model presented in Figure D.8 shows, schematically, how catchment sub-areas are gathered at successive node pits. The procedure begins at the most remote, to provide the hydrological basis for flow estimation in successive reaches of main drainage pipelines. Where main lines are branched, then the two (or more) branches must be considered separately down to their point of bifurcation.

The design flow conveyed from such a junction must be determined for the total catchment contributing to that point. Application of the Partial Area Rational Method (see Chapter 4) to determine this flow requires that critical storm durations in the contributing branches be evaluated and compared. Node to node travel times are shown in Figure D.8 to enable these durations to be determined.

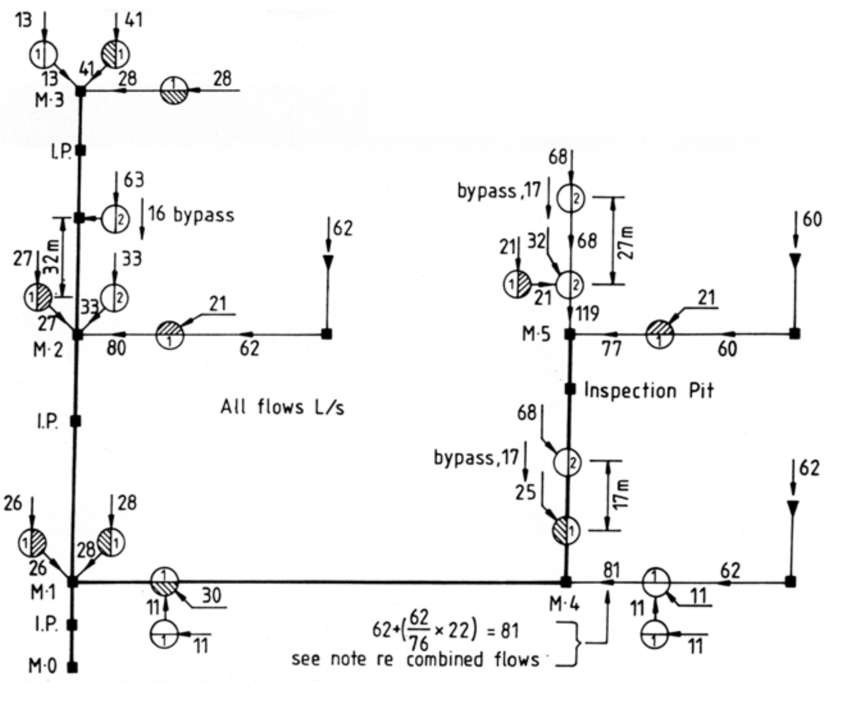


Figure D.7: Design flows in primary drainage lines

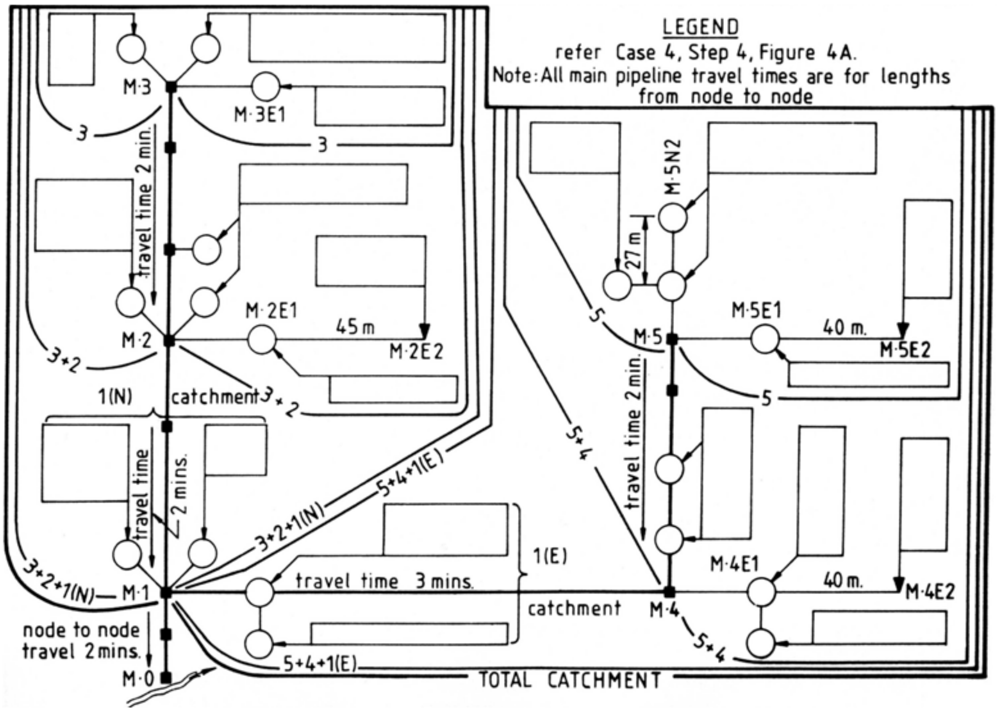


Figure D.8: 'Sub-area gathering' schematic hydrological model

Discussion of Table D.11

Table D.11 relates to Figure D.8 in the same way Table D.9 relates to Figure D.6 and includes many items transferred directly from the earlier tabulation. With the aid of Table D.11, the designer is able to execute the required sub-area gathering and storm duration comparison processes, and consequently determine design flows in each segment of the main pipeline network. The origins of the following notes are found in Table.D.11

1. The 'from upstream' component entered under node pit M.4 is that contributed from its adjacent (upstream) sub-area, M.5. Thus, the value for $(CA)_5$ = 1.07 ha entered in columns 51 and 52 has been transferred directly from columns 53 and 54 respectively.

2. Critical storm durations shown at node pit M.5 are:

   t_c = 19 mins and t_i = 9 mins (columns 22 and 23)

   Because travel time between M.5 and M.4 is 2 minutes (see Figure D.8), travel times for sub-area M.5 relative to node pit M.4 are 21 minutes and 11 minutes respectively (columns 45 and 46).

3. The flow I, (172 L/s) listed in column 56 is the design flow contributed from sub-area M.5 relative to node pit M.4. It can only be determined after all entries under M.4 have been evaluated to find critical storm durations (t_c and t_i) and, hence, design rainfall intensities. Thus the 172 L/s listing is derived from $(CA)_5$ = 1.07 ha and I_5 = 58 mm/h

   Rational method: $Q = \frac{CIA}{0.36} = \frac{1.07\,x\,58}{0.36} = 172$ L/s

   The flow, 187 L/s shown in column 56 is the design flow into and from node pit M.5, and should only be used for design in the immediate vicinity of M.5. Where flows in pipe segments near node pit M.4 are required, the sub-area M.5 'relative flow' of 172 L/s should be applied (see FigureD.10).

1. In general, entry of flow to main drain pipelines takes place at or near node pits. This is not always the case, e.g.:
    - segment M.5→M.4 : flow entries at M.4NI and M.4N2
    - segment M.3→M.2 : flow entry at M.2N2.

Table D.11: Flow estimation in main drain pipelines

Catchment	Sub-catchment	Sub-area node pit	Sub-area component: Entry point	Component description	Hydrolgical surface classification	Component travel time to node pit: Full Area	Component travel time to node pit: Part Area	Progressive citical storm duration it total upstream area: Full Area	Progressive citical storm duration it total upstream area: Part Area
						min.	min.	min.	min.
1	2	3	4	6	7	43	44	45	46
M		.5	NIR	7.5m road	paved 1st grade	5	5		
			N1	7.5m road	paved 1st grade	6	6		
				resid.	20res/ha	17	7		
						17	7		
			E1	7.5m road	paved 1st grade	5	5		
				resid.	20res/ha	19	9		
								19	9
M		.4	N1	7.5m road	paved 1st grade	5	5		
				resid.	20res/ha	17	7		
			E1	7.5m road	paved 1st grade	5	5		
				resid.	20res/ha	18	8		
			E1L	7.5m road	paved 1st grade	5	5		
								21	11
M		.3	NIR	10m road	paved 1st grade	5	5		
			N1L	10m road	paved 1st grade	7	7		
				park	pervious	32	7		
			E1	7.5m road	paved 1st grade	6	6		
				park	pervious	32	6		
M		.2	NIR	10m road	paved 1st grade	5	5		
			N1L	10m road	paved 1st grade	6	6		
				resid.	20res/ha	17	7		
						17	7		
			E1	7.5m road	paved 1st grade	5	5		
				resid.	20res/ha	18	8		
M		.1	NIR	10m road	paved 1st grade	6	6		
			N1L	10m road	paved 1st grade	6	6		
				park	pervious	40	6		
						40	6		
			E1	7.5m road	paved 1st grade	5	5		
				park	pervious	40	5		
						40	0		
			E1L	7.5m road	paved 1st grade	5	5		

Table D.11: Flow estimation in main drain pipelines *(Continued)*

Rainfall intensities for progressive critical storm duration in upstream catchment		Area	Runoff Coefficient	Component $(CA)_n$		Cunmulative $(CA)_n$		Cumulative mainline flows form total upstream catchment		Remarks
Full area	Part area			Full area	Part area	Full area	Part area	Full area Q_n	Part area Q_n	
mm/h	mm/h	ha		ha	ha	ha	ha	L/s	L/s	
47	48	49	50	51	52	53	54	55	56	57
				0.10	0.10					
				0.16	0.16					
				0.37	0.37					
				0.10	0.10					
				0.34	0.34					
45	63					1.07	1.07	134	187	Design flow at M.5
				1.07	1.07				(172)	
				0.11	0.11					Input to M.5/M.4 = (0.49x58)/0.36 = 79L/s
				0.38	0.38					
				0.05	0.05					
				0.34	0.34					Input at M.4 = 71L/s
				0.05	0.05					
42	58					2.00	2.00	233	322	Design flow at M.4
				2.00	2.00				(288)	
				0.13	0.13					
				0.05	0.05					Input at M.1E1 = 30L/s
				0.07	0.07					
29	52					2.25	2.25	181	325	Design flow to M.1
				0.06	0.06					
				0.21	0.21					
				0.12	0.12					
				0.04	7/32 x 0.04					
				0.13	7/32 x 0.13					
33	68					0.56	0.43	51	81	Design flow at M.3
									(75)	
				0.56	0.43					Input at M.2 = 23L/s
				0.13	0.13					input to M.3/M.2 = (0.51 x 63) / 0.36 = 89L/s
				0.18	0.18					
				0.33	0.33					
				0.10	0.10					Input at M.2 = 77L/s
				0.34	0.34					
32	63					1.64	1.51	146	264	Design flow at M.2
				1.64	1.51				(243)	
				0.13	0.13					Input at M.1 = 23L/s
				0.13	0.13					input at M.1 = 24L/s
				0.08	11/40 x 0.08					
29	58					1.98	1.79	160	289	Design flow to M.1
				2.25	2.20					
				1.98	1.79					
29	52					4.23	3.99	341	576	Design flow from M.1
				4.23	3.99				(543)	
				0.00	0.00					
28	49					4.23	3.99	329	543	Design flow at M.0

Determination of flows input at these points in design storms which are critical for mainlines involves two tasks:

Task 1: Find the flows contributed from their connected primary drainage areas in such design storms.

Task 2: Find the distribution of these flows, including gutter inlet capture, in the respective drainage lines.

The flows required in Task 1 for the M.5/M.4 and M.3/M.2 pipeline segments can be derived from information contained in Table D.11. The results are reported under 'Remarks' column 57.

They are:

segment M.5/M.4 :

$$\frac{(0.11+0.38)\ x\ 58)}{0.36} = \frac{0.49\ x\ 58}{0.36} = 79 \text{ L/s}$$

segment M.3/M.2 :

$$\frac{(0.18+0.33)\ x\ 63}{0.36} = \frac{0.51\ x\ 63}{0.36} = 89 \text{ L/s}$$

The distribution of these flows (Task 2), and consequently, the magnitudes of flows passing to the main pipeline(s) at the points of interest, can be determined using a graphical procedure (see Figure D.9). The accumulated flow along the gutter length is linear and varies with gutter profile (constant throughout this example) and slope. At each inlet, a proportion is captured (80 per cent in this example) and the remainder proceeds to the next inlet. The graphical procedure is simple but a spreadsheet approach allows greater flexibility when assigning placement of inlets.

1. Flow input to the main drain pipelines at other significant locations can be found in the manner indicated above (Task I). Results of these computations, derived from the data presented in Figure D.10 are also listed under 'Remarks' column 57.
2. Flow estimation in Branch M.5+M.4+M.1 concludes with branch design input to node pit M.1 of 318 L/s. Corresponding input from Branch M.3+M.2+M.1 to M.1 is 288 L/s. These flows arise from design storm durations that are critical in their individual catchments and hence storm rainfall intensities that are different (52 mm/h versus 58 mm/h*)*. The storm duration and hence intensity which is critical in the remaining segment of the network (Branch M.1 + M.O), must be resolved in accordance with Rational Method theory (Chapter 4).

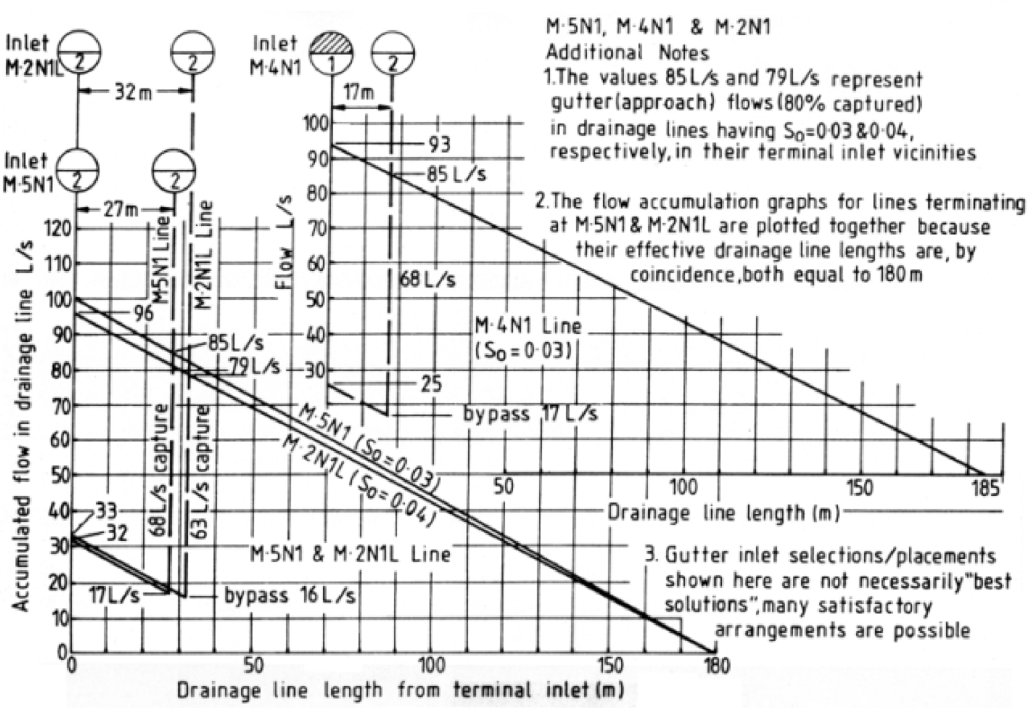


Figure D.9: Graphical procedure for flow accumulation and inlet capture

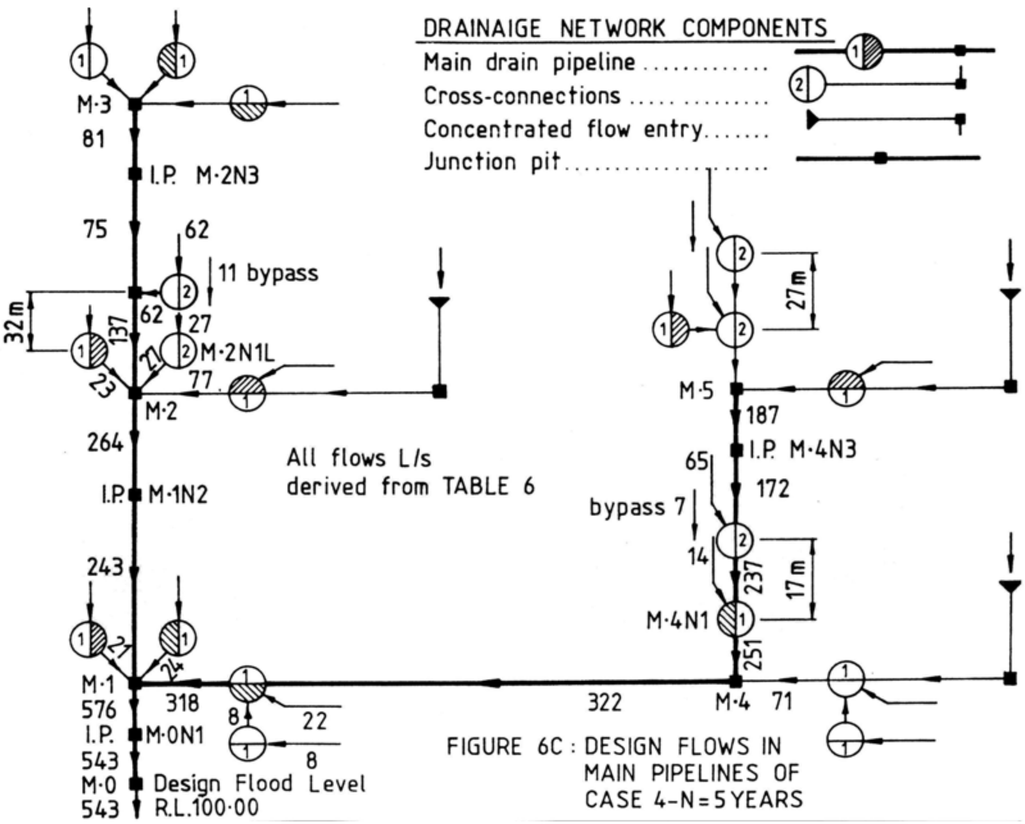


Figure D.10: Drainage network components

Table D.12: Outcomes of Step 6[5]

Description	Output
Primary drainage line flow distributions (design ARI = N years)	Figure D.7
Stage 2 hydrological model 'collective' interpretation	Figure D.8
Tabular computation of main drain pipeline design flows	Table D.11
Main drain pipeline flow distribution (design ARI = N years)	Figure D.11

Step 7: Design flow compilation - all components of sub-catchment

Data and information extracted from Figure D.7 and Figure D.10 enable the flow distribution compilation sought in Step 7 to be determined. The compilation presents in graphical form the magnitude of the greater design flow, where there is choice (Figure D.7 and Figure D.10 values compared), which can occur in each component surface or underground flow path of a sub-catchment. The compilation is presented inFigure D.11.

Where the design of a minor drainage system for a complex urban landscape is required, an additional task must be performed. This involves the computation of flows moving underground between sub-catchment disposal points towards their (catchment) central disposal point (see Design Procedure -Phase II above). The outcomes of these computations are presented in Figure D.7; FigureD.8, etc.

Data Example

In this step, data and information extracted from Figure D.7 and Figure D.10 are used to compile the flow distribution presented graphically asFigure D.11.

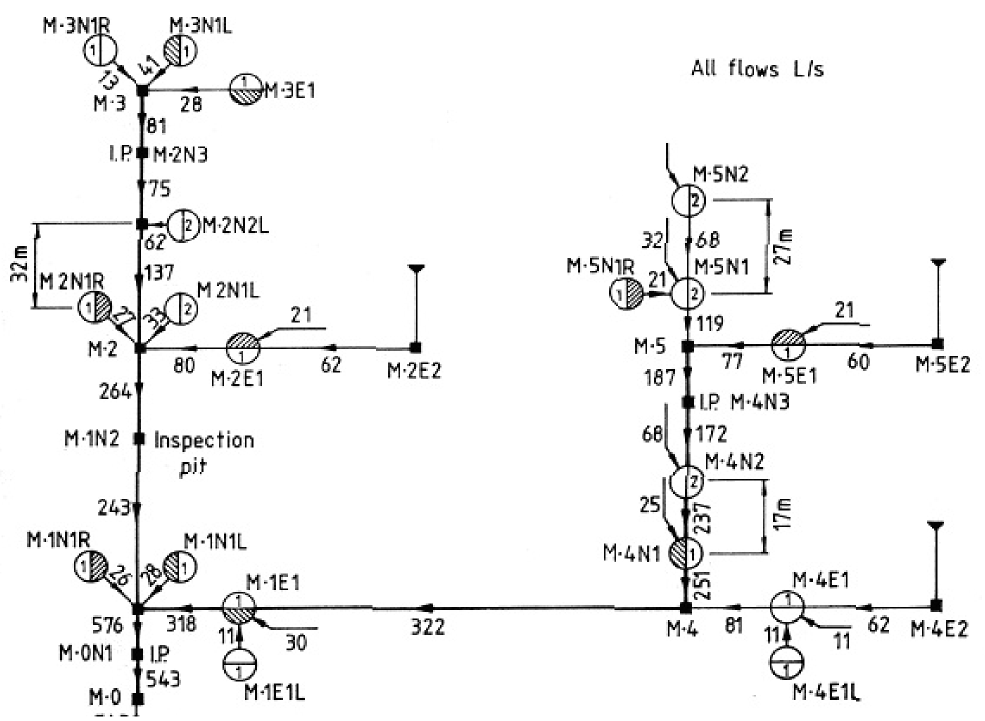


Figure D.11: Design flows in all pipe components

The design flow distribution set out in Figure D.11 uses the greater flow where there is choice, shown for each component in Figure D.7 and Figure D.10. While this gives appropriate values for use in the design of the components themselves. It represents a loss of data that may be of importance in some design practices.

Consider the alternative flows presented for pipe M.4EI+M.4:

from Figure D.7 (primary drainage considerations) flow = 81 L/s

from Figure D.10 (total catchment considerations) flow = 71 L/s

Pipe M.4E1 + M.4 should be designed to convey a design flow of 81 L/s.

The design of mainline component M.4 to M.IE1, however, involves not only its design flow of 322 L/s but also the flows entering node M.4 under corresponding design conditions. These flows determine the pit headloss (water level) value, ‘k’ (see Table 6.10), which should be used at the junction pit and are:

- 251 L/s from the northern pipe
- 71 L/s from pipe M.4E1 + M.4.

The procedure adopted for selecting values for ‘k’ (see Table 6.10) is an approximate one that is insensitive to the difference discussed here.

A further interpretation which has been made in preparing Figure D.11 should be observed and this concerns the aspect discussed in Step 6, Note 3, Table D.9, above.

Pipes carrying discharge directly from node pits are shown with the appropriate values collated from Figure D.9. Flows from the locations where these pipes are interrupted downstream by a junction pit, inspection pit, etc., use the 'relative flow' value (shown bracketed in Figure D.9).

Table D.13: Outcome of Step 7 — design flow distribution

Description	Output

Design flow distribution compilation for sub-catchment	Figure D.11
Design flow distribution compilation for other sub-catchments (same procedures applied to sub-catchments I, N & P but not addressed in this example)	Equivalent to Figure D.11
Design flows between terminal node pits of parallel sub-catchments catchments (same procedures applied to sub-catchments I, N & P but not addressed in this example)	Equivalent to Figure D.11

The declaration of these flow compilations completes Phase II of the minor system design procedure and provides the basis for the underground network design steps and final design detailing of Phase III.

Step 8: Pit water levels and first-round pipe

Design of the underground network commences with the setting of water levels called 'assigned water levels' (AWL's) in all junction pits, with or without gutter inlets, located on sub-catchment lateral and main pipelines. AWL's are set first along lateral pipelines and main pipeline branches beginning at their upstream extremities and then along main pipeline trunks commencing from their discharge points (see Table D.5 Guideline 6).

In setting pit water levels the drainage designer should observe the requirements of Table D.5 Guidelines 4 and 5.

Data Example

The layout of Table D.14 enables these opening tasks of Step 8 to be accomplished in a rapid and orderly fashion.

These tasks are followed by 'first approximation' pipe size selection for all components of lateral and main drain pipelines. Sizes are fixed according to Minimum Grade design procedures using simple charts of the type presented in Figure 6.12 and Table 6.10. Pit head losses are ignored at this stage. Design flows are obtained from the Step 7.

Priority should be observed in selecting sizes for pipe components. Lateral pipeline components should be selected before pipes of the main drain (see Table D.5 Guideline 8). This task is performed using Table D.14 that has been designed to facilitate pipe selection.

(See Table D.14 over page)

Table D.14: Pit water levels, pipe diameters and HGL analysis

CATCHMENT	SUB-CATCHMENT	SUB-AREA NODE PIT	JUNCTION PIT AND GUTTER INVERT R.L's	COMPONENT DESCRIPTION	FLOW L/s	LENGTH m	LONGITUDINAL SLOPE	PIT WATER LEVEL GUIDELINE 4 (Table 6.4)	GUIDELINE 5 (Table 6.4)	R.L. ASSIGNED WATER LEVEL (AWL)	m TRIAL PIPE DIAMETER D	VELOCITY, $V_o = Q_o / A_o$ m/s	PIT LOSS CASE (Tables 6.5 & 6.6)	K_w	$\frac{V_o^2}{2g}$ m	m $h_f = 0.017 L / D_o \times (V_o^2 / 2g)$	m $K_w (V_o^2 / 2g)$	TEST 1 $h_f + K_w [\frac{V_o^2}{2g}]$? (AWL - BWL) m	m AWL minus BWL	TEST 2 $h_f + 1.5D_o$? (AWL - BWL) m	mm ADOPTED DIAMETER	BWL & HYDRAULIC GRADE LINE (Pit WL's)	REMARKS AWL = assigned (pit) water level BWL = bottom (pit) water level
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
LATERAL PIPELINES																							
M		.5	E2 111.35	L Junction pit				111.20	n.a.	111.20												111.21	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 110.79 by eqn (6.21) max. BWL at E2, RL 111.20 (=AWL)
				pipe E2 ? E1	60	40	.024				.300	.85	J-3	2.0	.04	.08	.07	0.16	0.97	0.53	300		
			E1 110.40	JP with gutter flow				110.25	111.10	110.25												110.24	TESTS 1 and 2 OK for D_o = 300; upstream inv. max. RL 109.87 by eqn (6.19) ; max. BWL at E1, RL 110.23 by eqn (6.20)
				pipe E1 ? M.5	77	5	.020				.300	1.09	I-2A/2B	1.0	.06	.02	.06	.08	0.10	0.47	300		
			NODE M.5 110.30	T junction pit				110.15	110.15	110.15		BWL FROM MAIN PIPELINE (BRANCH) COMPUTATIONS →										110.16	
			N2 111.30	JP with gutter flow				111.15	n.a.	111.15												111.15	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 110.56 by eqn (6.21) max. BWL at N2, RL 111.15 (=AWL)
				pipe N2 ? N1	68	27	.024				.300	.96	I-1	4.0	.05	.07	.19	0.26	0.68	0.52	300		
			N1 110.65	JP, gutter flow, lat's				110.50	111.05	110.50												110.47	TESTS 1 and 2 OK for D_o = 300; upstream inv. max. RL 109.95 by eqn (6.19) ; max. BWL at N1, RL 110.47 by eqn (6.20)
				pipe N1 ? M.5	119	12	.029				.300	1.68	I-3B/3C	1.5	.14	.10	.22	0.31	0.35	0.55	300		
			NODE M.5 110.30	T junction pit	previously assigned →			110.15	110.40	110.15		BWL FROM MAIN PIPELINE (BRANCH) COMPUTATIONS →										110.16	
M		.4	E2 103.25	L Junction pit				103.10	n.a.	103.10												103.11	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 102.68 by eqn (6.21) max. BWL at E2, RL 103.10 (=AWL)
				pipe E2 ? E1	62	40	.006				.300	.88	J-3	2.0	.04	.09	.08	0.17	0.58	0.54	300		
			E1 103.03	JP, gutter flow, lat's				102.88	103.00	102.88												102.53	TESTS 1 and 2 OK for D_o = 300; upstream inv. max. RL 102.19 by eqn (6.19) ; max. BWL at E1, RL 102.52 by eqn (6.20)
				pipe E1 ? M.4	81	5	.006				.300	1.15	I-3A	0.5	.07	.02	.03	0.05	0.41	0.47	300		
			NODE M.4 103.00	thro' pipe with lat's				102.85	102.78	102.78		BWL FROM MAIN PIPELINE (BRANCH) COMPUTATIONS →										102.48	
M		.2	E2 108.40	L Junction pit				108.25	n.a.	108.25												115.15	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 107.83 by eqn (6.21) max. BWL at E2, RL 108.25 (=AWL)
				pipe E2 ? E1	62	45	.020				.300	.88	J-3	2.0	.04	.10	.08	0.18	1.00	0.55	300		
			E1 107.50	JP with gutter flow				107.35	108.15	107.35												114.15	TESTS 1 and 2 OK for D_o = 300; upstream inv. max. RL 106.88 by eqn (6.19) ; max. BWL at E1, RL 107.25 by eqn (6.20)
				pipe E1 ? M.2	80	7	.029				.300	1.13	I-2A/2B	1.0	.07	.03	.07	0.09	0.20	0.48	300		
			NODE M.2 107.30	multi-pipe JP				107.15	107.25	107.15		BWL FROM MAIN PIPELINE (BRANCH) COMPUTATIONS →										114.06	

Table D.14: Pit water levels, pipe diameters and HGL analysis *(Continued)*

MAIN PIPELINE : BRANCH M.5-M.4-M.1																							
M		.4	NODE M.5 110.30	thro' pipe with lat's	previously assigned			→		110.15												110.16	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 109.31 by eqn (6.21) max. BWL at M.4, RL 110.15 (=AWL)
				pipe M.5 ? N3	187	100	.036				.300	2.65	J-2B	1.0	.36	2.02	.36	2.38	3.60	2.47	300		
			N3 106.70	thro' pipe JP				106.55	110.05	106.55												106.56	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max. RL 106.16 by eqn (6.21) max. BWL at N3, RL 106.55 (=AWL)
				pipe N3 ? N2	172	83	.036				.300	2.43	J-1	0.2	.30	1.42	.06	1.48	3.00	1.87	300		
			N2 103.70	JP with gutter flow				103.55	106.45	103.55												103.56	TEST 1 OK for D_o = 375; fails TEST 2; u/s inv. max. RL 102.99 by eqn (6.21) max. BWL at N2, RL 103.55 (=AWL)
				pipe N2 ? N1	237	17	.032				.375	2.15	I-2B	0.5	.23	.18	.12	0.30	0.89	0.74	375		
			N1 103.15	JP with gutter flow				103.00	103.45	103.00												102.67	TESTS 1 & 2 OK for D_o = 375; u/s inv. max. RL 102.15 by eqn (6.19) max. BWL at N1, RL 102.66 by eqn (6.20)
				pipe N1 ? M.4	251	5	.030				.375	2.27	I-2B	0.5	.26	.06	.13	0.19	0.53	0.62	375		
M		.1	NODE M.4 103.00	thro' pipe with lat's	previously assigned →			102.78	102.90	102.78												102.48	TESTS 1 & 2 OK for D_o = 525; u/s inv. Max RL 101.72 by eqn (6.19); max. BWL at M.4, RL 102.47 by eqn (6.20)
				pipe M.4 ? E1	322	100	.008				.450 .525	2.02 1.49	J-2C	2.0	.21 .11	.79 .37	.42 .23	1.21 0.59	0.89	- 1.15	525		
			E1 102.25	JP, gutter flow, lat's				102.10	102.68	102.10												101.89	TESTS 1 & 2 OK for D_o = 525; upstream inv. Max RL 101.30 by eqn (6.19) ; max. BWL at E1, RL 101.89 by eqn (6.20)
				pipe E1 ? M.1	318	8	.006				.525	1.47	I-3A	0.5	.11	.03	.05	0.08	0.30	0.82	525		
			NODE M.1 102.20	multi-pipe JP				102.05	102.00	102.00		BWL FROM MAIN PIPELINE (TRUNK) COMPUTATIONS →										101.80	
MAIN PIPELINE : TRUNK M.3-M.2-M.1-M.0																							
M		.2	NODE M.3 114.20	multi-pipe JP				114.05	110.55	114.05												114.06	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max RL 113.45 by eqn (6.21) ; max. BWL at M.3, RL 114.05 (=AWL)
				pipe M.3 ? N3	81	100	.036				.300	1.15	multi-pipe JP	3.0	.07	.38	.20	0.58	3.60	0.83	300		
			N3 110.60	thro' pipe JP				110.45	108.35	110.45												110.46	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max RL 110.13 by eqn (6.21) ; max. BWL at N3, RL 110.45 (=AWL)
				pipe N3 ? N2	75	68	.032				.300	1.06	J-1	0.2	.06	.22	.01	0.23	2.20	0.67	300		
			N2 108.40	thro' pipe with lat's				108.25	107.25	108.25												108.26	TEST 1 OK for D_o = 300; fails TEST 2; u/s inv. max RL 107.66 by eqn (6.21) ; max. BWL at N2, RL 108.25 (=AWL)
				pipe N2 ? M.2	137	44	.025				.300	1.94	J-2B	1.0	.19	.48	.19	0.67	1.41	0.93	300		
M		.1	NODE M.2 107.30	multi-pipe JP	previously assigned →			107.15	104.75	107.15												106.85	TEST 1 OK for D_o = 375; fails TEST 2; u/s inv. max RL 105.60 by eqn (6.19) ; max. BWL at M.2, RL 106.84 by eqn (6.20)
				pipe M.2 ? N2	264	100	.025				.375	2.39	multi-pipe JP	3.0	.29	1.32	.87	2.19	2.50	1.88	375		
			N2 104.80	thro' pipe JP				104.65	102.10	104.65												104.65	TEST 1 OK for D_o = 375; fails TEST 2; u/s inv. max RL 104.19 by eqn (6.21) ; max. BWL at N2, RL 104.65 (=AWL)
				pipe N2 ? M.1	243	105	.025				.375	2.20	J-1	0.2	.25	1.17	.05	1.22	2.85	1.74	375		
M		.0	NODE M.1 102.20	multi-pipe JP	previously assigned →			102.00	100.85	102.00												101.80	TESTS 1 & 2 OK for D_o = 600; u/s inv. max RL 100.57 by eqn (6.19); max. BWL at M.1, RL 101.80 by eqn (6.20)
				pipe M.1 ? N1	576	100	.013				.525 .600	2.66 2.04	multi-pipe JP	3.0	.36 .21	1.17 .60	1.08 .63	2.25 1.23	1.43	- 1.50	600		
			N1 100.90	thro' pipe JP				100.75	100.10	100.75												100.57	TESTS 1 & 2 OK for D_o = 600; u/s inv. max RL 99.93 by eqn (6.19); max. BWL at N1, RL 100.57 by eqn (6.20)
				pipe N1 ? M0	543	100	.009				.600	1.92	J-1	0.2	.19	.53	.04	0.57	0.75	1.43	600		
			M.0	DESIGN FLOOD LEVEL RL 100.000 = BWL →																		100.00	

Table D.15: Outcomes of Step 8

Description	Output
Junction pit water levels (AWL's) are assigned along all lateral and main drain pipelines.	Table D.14
First-round pipe sizes are selected for all lateral and main drain components	Table D.14

Step 9: HGL and pipes of approximate network design

The tasks of reviewing the first round of selected pipes and making alterations where necessary in the light of Hydraulic Grade Line analysis, is conducted in Step 9 as a combined exercise. It is commenced at the downstream extremity of the sub-catchment and its main calculations are reported inTable D.14

The tabular procedure includes two Hydraulic Grade Line tests. One designed to maintain HGL below assigned pit water level (AWL), the other aimed at keeping pit floor level above 'AWL - 2.5 D_o'. Generally, the first of these aims can be achieved by appropriate pipe diameter selection. The second can be achieved in the majority of cases, but sometimes fails at locations where high discharges pass through multi-pipe junction pits or where a severe direction change is forced upon such flows.

In practice, Table D.5 Guidelines 6 and 7 often limit options and give rise to situations where HGL falls significantly below a pipe obvert. In such cases, underground moving flows take the part full or open channel form and normal pressure pipeline head loss and pit head loss (water level) values cannot be validly applied. A design approach to meet these situations is offered in Table 6.10.

Examples of such situations occur most frequently in the upper extremities of moderate and steep grade catchment networks, but they can occur in valley bottoms where there is a local increase in terrain slope for one reach of pipeline. These latter cases are the more serious cases for the designer since they are likely to be preceded and followed by reaches that behave as normal pressure pipelines.

The two possible outcomes reviewed here, junction pit outflow full or part-full, lead to different algorithms for fixing pipe inverts and, hence, mainline junction pit floor levels (see Section 6.4.3) The computations which arise as a consequence of these outcomes are entered in the Remarks column of Table D.14.

Where underground networks are to be designed for catchments comprising a number of sub-catchments, then a set of tables similar to Table D.14, one for each sub-catchment, must be prepared. HGL and diameters for pipes linking sub-catchment terminal node pits must also be determined.

Table D.16: Outcomes of Step 9

Description	Output
Hydraulic Grade Line and pipe sizes of approximate underground network design for sub-catchment (ARI = N years)	Table D.14
Junction pit outflow pipe maximum invert levels (approximate network design) (ARI = N years)	Table D.14
H.G.L. and pipe sizes for approximate underground network designs for other sub-catchments and pipes linking their terminal node pits.	Equivalent to Table D.14

Step 10: Approximate system design

The first task in Step 10 is to extract information on lateral and main pipeline adopted pipe diameters (Table D.14) and present these on a Stage 2 hydrological model network layout. Diameters of the remaining pipes cross-connections from gutter and ‘sag’ inlets, etc., to the lateral and main pipelines are fixed by reference to Table D.5 Guideline 9 using design flow information for these components gathered from Figure D.11.

These data are then collated with information on surface drainage components declared in Step 5 gutter and 'sag' inlet locations, types, sizes, etc., to produce an approximate design for the sub-catchment minor drainage system. The design is that for ARI = N years and is presented in schematic form showing:

- gutter and ‘sag’ inlet pits locations, types, sizes
- combined inlet / junction pits locations, types, sizes, pipe entries and exits
- junction and inspection pits
- locations, pipe entries and exits;
- concentrated flow entry points locations; and
- pipe components (all pipes) locations, types, lengths, diameters.

Where catchments comprising a number of sub-catchments are involved, then a set of approximate minor system designs must be prepared, one for each sub-catchment. Diameters for pipes linking sub-catchment terminal node pits (see Step 9) form part of the outcome of Step 10.

The design is described as 'approximate' because its final form cannot be determined until site constraints and construction requirements have been taken into account.

Data example

This step involves a schematic representation of the sub-catchment approximate minor stormwater drainage system. Information on lateral and main pipeline adopted pipe diameters from Step 9 and data on surface drainage components from Step 5 are collated to produce Figure D.12.

Table D.17: Outcomes of Step 10

Description	Output
Schematic representation of sub-catchment approximate minor stormwater drainage system for design.	Figure D.12
Schematic representations of parallel sub-catchment approximate minor system designs including pipes linking their terminal node pits.	Equivalent to Figure D.12

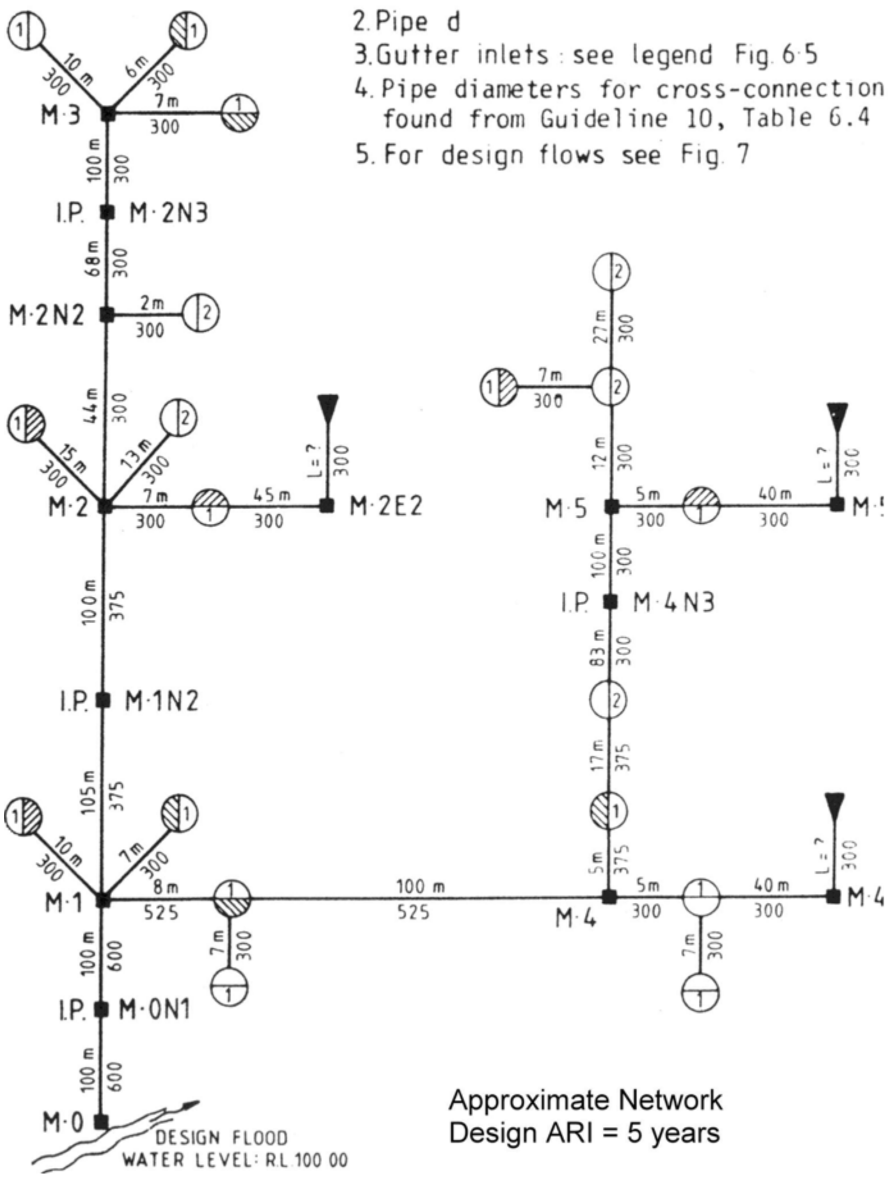


Figure D.12: Approximate network design

Step 10A: Cost / frequency insert

The question of what ARIs should be applied in the design of minor stormwater drainage systems varies depending upon the nature of the project and where it lies, as well who has jurisdiction for drainage scheme. Design ARI tends to be fixed by local government policy or the recommendation of a funding authority or by application of guidelines / suggestions such as those listed in Table 6.5.

Use of 'policy' is appropriate provided it is soundly based on assessment of sample developments covering the range of likely land uses. One item of economic evaluation that should be part of any such enquiry is cost / frequency analysis (see Chapter 10 to Risk Assessment).

The first task in conducting a cost / frequency analysis is to assign a range of values to the design ARI parameter 'N'. Typical values used are:

- residential catchments are N = 0, 1, 2, 5 and 10 year values; and
- industrial / commercial developments are N = 0, 2, 5, 10 and 20 year values.

With these values assigned, the procedure described in Steps 1- 10 inclusive must be carried out for each nominated non-zero value of N commencing with the greatest. This is less repetitive than it may appear since Steps 1 - 5 inclusive are common to all designs for a particular sample area.

The design resulting from the ‘0 year’ case is the hypothetical layout of inlets, underground pipes and junction pits which satisfied the minimum requirements of the guidelines adopted in Steps 2 and 3. It realises the Stage 1 hydrological model (Figure D.5). All of its gutter inlets are minimum size, and all of its pipelines are minimum size (Guideline 7,Table D.5); discharges from all concentrated flow entries are accepted into their respective surface drainage lines.

The result of these determinations is a set of approximate minor system designs, each one similar in form to that presented in Figure D.10. This constitutes the main part of the database for the analysis.

The next task involves costing the set of designs. The level of sophistication employed here need not be high but should include:

- costs of all drainage network main components
- junction pits
- concentrated flow entries
- gutter inlets of all types
- pipes
- excavation and installation costs
- roadway reinstatement costs.

The outcome of such an analysis is a set of minor system network costs, e.g.

- design ARI = 10 years $177,500
- design ARI = 5 years $171,500
- design ARI = 2 years $156,000
- design ARI = 1 year $151,200
- design ARI = 0 year $136,500

These cost / frequency data are plotted. The cost versus design ARI relationship can take many forms which differ with the type of development, unit cost rates used, adopted guidelines (Steps 2 and 3), etc. The 'double cusp' form of curve is typical of residential catchments. The convex form is obtained, typically, for industrial / commercial developments.

These considerations provide designers with valuable information which aids policy making. They are urged, however, to resist the temptation to adopt design ARIs which have been decided solely on cost considerations of the type described here.

Step 11: Final design detailing

In Step II the schematic representation of the sub-catchment minor stormwater drainage system, collated in Step 10, is converted to a set of working drawings used for construction. The conversion is accomplished in two stages:

Stage A: pit floor levels according to equations (Section 6.4.3) collated from the 'Remarks' column of Table D.14 and Table D.5 Guidelines 11 and 12 are applied to the approximate design declared in Step 10. It is also appropriate for pipe class and cover requirements (Table D.5 Guideline 13) to be considered at this stage.

Stage B: site constraints, service locations in particular, are applied to the Stage A design to produce a set of working or construction drawings. Some recalculation may be involved.

It is unusual for the Stage A design to be directly transferable to site without alterations being required. These may range from minor changes (small differences in pipe lengths and alignments that have no appreciable effect on the design) to major redesigns. In the latter case, significant alterations to the network, e.g. replacement of a number of conventional junction pits with 'drop' pits or alternative pipeline modifications to avoid existing underground services, may force recalculation of some reaches of the network. Such recalculation should not extend further back into the procedure than Table D.14.

In extreme circumstances, e.g. where pit overflow appears to be inescapable, pipe diameter selection and Hydraulic Grade Line computations may have to be repeated. Consideration should also be given to the hydraulic modelling of particular installations, e.g. multi-pipe junction pits, whose behaviour under design flow conditions is crucial to the success or failure of a system.

Much of this need for redesign can be pre-empted if a thorough site survey and investigation is carried out as part of catchment definition [Item (c), Step 1]. The drainage designer who is aware of the locations of all major services and other underground installations will arrange his basic network layout (Figure D.4) to avoid these, and all subsequent segments of the design will reflect this knowledge. The alternative is a set of major computational difficulties to be overcome in Step 11 or, worse still, following the discovery of an unexpected service line unearthed by a construction crew.

Data Example

In this step, the schematic representation of the sub-catchment minor stormwater drainage system collate in Step 10 is converted to a set of working drawings used for construction. The conversion is accomplished in two stages. In the first stage, pit floor levels are applied to the approximate design declared in Step 10. The result is a schematic representation of sub-catchment final design minor stormwater drainage system shown in Figure D.10. In the second stage, site constraints are applied to the first stage design to produce a set of working or construction drawings, which may involve some recalculation and these are shown in Figure 11.

Table D.18: Outcomes of Step 11

Description	Output
Schematic representation of sub-catchment final design (Stage A) minor stormwater drainage system.	Figure D.11
Schematic representation of parallel sub-catchment final design (Stage A) minor drainage systems including pipes linking their terminal node pits.	Equivalent to Figure D.11
Construction drawings of catchment minor stormwater drainage system (Stage B).	Drawings

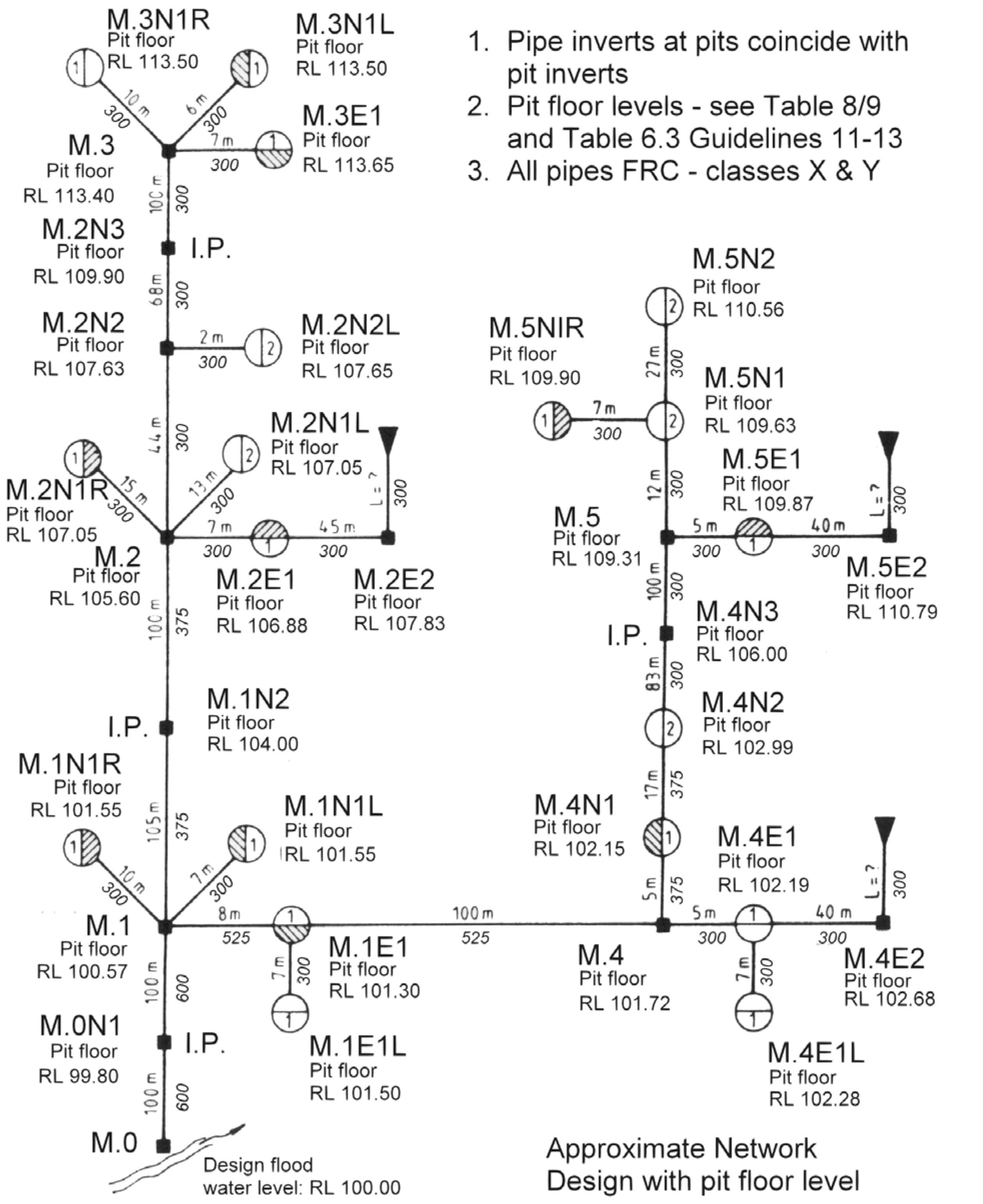


Figure D.13: Approximate network design with PIT floor levels

APPENDIX E

MANNING'S ROUGHNESS COEFFICIENTS (after VicRoads, 1995)

Situation			Manning's Roughness Coefficient
Closed conduits:			
Concrete pipe			0.011 to 0.013
Corrugated-metal pipe or pipe-arch			
	66-50 mm corrugation (riveted pipe)		
		Plain or fully coated	0.024
		Paved invert (range values are for 25 and 50 percent of circumference paved	
		Flow full depth	0.021 to 0.018
		Flow 0.8 depth	0.021 to 0.016
		Flow 0.6 depth	0.019 to 0.013
	150 X 50 mm corrugated (field bolted)		0.03
	Helical corrugated metal pipe		
		68 x 13 mm corrugation	0.012 to 0.024
		125 x 25 mm corrugation	0.024
Vitrified clay pipe			0.012 to 0.014
Cast iron pipe, uncoated			0.013
Steel pipe			0.009 to 0.011
Brick			0.014 to 0.017
Monolithic concrete			
	Wood forms rough		0.015 to 0.017
	Wood forms smooth		0.012 to 0.014
	Steel Forms		0.012 to 0.013
Cemented rubble masonry walls			
	Concrete floor and top		0.017 to 0.022
	Natural floor		0.019 to 0.025
Laminated treated wood			0.015 to 0.017
Vitrified clay liner plates			0.015

Situation		Manning's Roughness Coefficient
Open channels, lined (straight alignment):		
Concrete with surfaces as indicated		
	Formed finish	0.013 to 0.017
	Trowel finish	0.012 to 0.014
	Float finish	0.013 to 0.015
	Float finish, some gravel on bottom	0.016 to 0.019
	Gunite, good section	0.016 to 0019
	Gunite, wavy section	0.016 to 0.022
Concrete bottom, float finished, sides as indicated		
	Dressed stone in mortar	0.015 to 0.017
	Random stone in mortar	0.017 to 0.020
	Cement rubble masonry	0.020 to 0.025
	Cement rubble masonry plastered	0.016 to 0.020
	Dry rubble(riprap)	0.020 to 0.030
Gravel bottom, sides as indicated		
	Formed concrete	0.017 to 0.020
	Random stone in mortar	0.020 to 0.023
	Dry Rubble(riprap)	0.023 to 0.033
Brick		0.014 to 0.017
Asphalt		
	Smooth	0.013
	Rough	0.016
Wood planed, clean		0.011 to 0.013
Concrete lined excavated rock		
	Good section	0.017 to 0.020
	Irregular section	0.022 to 0.027

Situation			Manning's Roughness Coefficient
Open channels, excavated (straight alignment, natural lining):			
Earth, uniform section			
	Clean, recently completed		0.016 to 0.018
	Clean after weathering		0.018 to 0.020
	With short grass, few weeds		0.022 to 0.027
	In gravelly soil uniform section, clean		0.022 to 0.025
Earth, fairly uniform section			
	No vegetation		0.022 to 0.025
	Grass, some weeds		0.025 to 0.030
	Dense weeds or aquatic plants in deep channels		0.030 to 0.035
	Sides clean, gravel bottom		0.025 to 0.030
	Sides clean, cobble bottom		0.03 to 0.04
Dragline excavated or dredged			
	No vegetation		0.028 to 0.033
	Light brush on banks		0.035 to 0.050
Rock			
	Based on design section		0.035
	Based on actual mean section		
		Smooth and uniform	0.035 to 0.040
		Jagged and irregular	0.040 to 0.045
Channels not maintained, weeds and brush uncut			
	Dense weeds, high as flow depth		0.08 to 012
	Clean bottom, brush on sides		0.05 to 0.08
	Dense brush, high stage		0.10 to 014

Situation			Manning's Roughness Coefficient
Highway channels and drains with maintained vegetation (values shown are for velocities between 0.6 and 2 m/sec.):			
Depth of flow up to 0.20 m			
	Bermuda grass, Kentucky bluegrass, buffalo grass		
		Mowed to 50 mm	0.07 to 0.05
		100-150 mm	0.09 to 0.05
	Good stand, any grass		
		Length about 300 mm	0.18 to 0.09
		Length about 600 mm	0.30 to 0.15
	Fair stand, any grass		
		Length about 300 mm	0.14 to 0.08
		Length about 600 mm	0.25 to 0.13
Depth of flow 0.20-0.45 m			
	Bermuda grass, Kentucky bluegrass, buffalo grass		
		Mowed to 50 mm	0.05 to 0.035
		Length 100-150 mm	0.06 to 0.04
	Good stand, any grass		
		Length about 300 mm	0.12 to 0.07
		Length about 600 mm	0.20 to 0.10
	Fair stand, any grass		
		Length about 300 mm	0.10 to 0.06
		Length about 600 mm	0.17 to 0.09
Street and expressway gutters:			
Concrete gutter, trowelled finish			0.012
Asphalt pavement			
	Smooth texture		0.013
	Rough texture		0.016
Concrete gutter with asphalt pavement			
	Smooth		0.013
	Rough		0.015
Concrete pavement			
	Float finish		0.014
	Broom finish		0.016
For gutters with small slope, where sediment may accumulate increase above values by			0.002

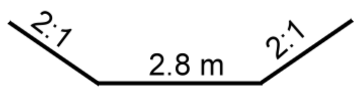
Situation	Manning's Roughness Coefficient
Natural stream channels:	
Minor streams (surface width at flood stage less than 30 m)	
Fairly regular section	
Some grass and weeds little or no brush	0.30 to 0.035
Dense growth of weeds, depth of flow materially greater than weed height	0.035 to 0.05
Some weeds, light brush on banks	0.035 to 0.05
Some weeds, heavy brush on banks	0.05 to 0.07
Some weeds, dense willows on banks	0.06 to 0.08
For trees within channel, with branches submerged at high stage, increase all above values by	0.01 to 0.02
Irregular sections, with pools, slight channel meander increase values by	0.01 to 0.02
Mountain streams no vegetation in channel, banks usually steep trees and brush along banks submerged at high stage.	
Bottom of gravel, cobbles and few boulders	0.04 to 0.05
Bottom of cobbles, with large boulders	0.05 to 0.07
Flood plains (adjacent to natural streams)	
Pasture, no brush	
Short grass	0.030-0035
High grass	0.035-0.05
Cultivated areas	
No crop	0.03-004
Mature row crops	0.035-0.045
Mature field crops	0.04-0.05
Heavy weeds, scattered brush	0.05-0.07
Light brush and trees	
Winter	0.05-0.06
Summer	0.06-0.08
Medium to dense brush	
Winter	0.07-0.11
Summer	0.10-0.16
Dense willows. summer, not bent over by current	0.15-0.20
Cleared land with tree slumps, 250-400 per hectare	
No sprouts	0.04-0.05
With heavy growth of sprouts	0.06-0.08
Heavy stand of timber, a few down trees, little undergrowth	
Flood depth below branches	0.10-012
Flood depth reaches branches	0.12-0.16

Situation			Manning's Roughness Coefficient
Major streams (surface width at flood stage more than 30 m):			
	Roughness coefficient is usually less than for minor streams of similar description on account of less effective resistance offered by irregular banks or vegetation on banks. Values may be somewhat reduced. The value for larger streams of most regular section, with no boulders or brush may be in the range of		0.028-0.033
		Regular section with no boulders or brush	0025-0.060
		Irregular and rough section	0.035-0 100

APPENDIX F

F.1 Worked example of Culvert

A discharge of 8.2 m^3/s is required to pass through a concrete pipe under a road for a length of 25 m at a slope of 1 in 200. The outlet is a trapezoidal channel with the dimensions as shown in the figure and is lined with 100 mm gravel. Standard construction is to use a headwall with a square edge. The headwater must be less than 2.5 m. What size culvert meets these requirements?



Q = 8.2 m^3/s

Allowable velocity = 2.5 m/s (Table 8.6)

Required area = 8.2 / 2.5 = 3.3 m^2 (single pipe of 2.05 m diameter, or, three pipes of 1.2 m diameter)

Trial pipe size: 3 of 1.35 m diameter

Q/N = 8.2 / 3 = 2.73 m^3/s

HWi/D (from an inlet control nomograph) = 1.6

HWi = 1.6 x 1.35 = 2.16 m

Critical depth in pipes (from nomographs) = 0.9 m

Determination of tailwater depth

Cross sectional area in outlet channel, A = 2[(2d x d) /2] + 2.8 x d = $2d^2$ + 2.8d

Where, d is the tailwater depth

Wetted perimeter, P = 2.8 + 2 x d x $(5)^{0.5}$

Hydraulic radius, R = A/P = (2.8d + $2d^2$)/[2.8 + 2d√5]

Velocity, V = $R^{0.667}$ x $S^{0.5}$ / n

Capacity, Q = V A

Assume Manning's n of 0.03

Try d = 0.5 m

A = 1.9 m^2;

P = 5.04 m;

R = 0.38 m;

V = 1.23 m/s;

Q = 2.34 m^3/s: insufficient capacity at this depth

Try d = 1.0 m

A = 4.8 m^2;

P = 7.27 m;

R = 0.66;

V = 1.79 m/s;

Q = 8.56 m^3/s: excess capacity at this depth

By interpolation the depth required to produce a capacity in the channel of 8.2 m^3/s was found to be 0.97 m.

Tailwater depth is less than the diameter of culvert pipes, but greater than the critical depth.

(hc + D) / 2 = (0.9 + 1.35) / 2 = 1.13 m; which is greater than the tailwater depth. In this instance the flow breaks away from the top of the pipe somewhere inside the culvert and the tailwater depth is assumed to be (hc + D) / 2 = 1.13 m

Calculate losses within culvert due to energy losses, entrance losses (from nomographs) = 0.28 m

Headwater depth based on outlet control = 0.28 + 1.13 – 25 x 1/200 = 1.285 m

Head water is the greater of the two calculated headwaters = 2.16 m (under inlet control)

This is less than the maximum head water permitted and therefore the culvert meets requirements. Need to check that the flow velocity does not exceed the permissible limit of 2.5 m/s (for the 100 mm gravel used to line the channel).

Flow depth calculated previously was 0.97 m and the velocity is determined using V = Q/A.

A = $2.8d + 2d^2 = 4.6\ m^2$

Q = 8.2 m^3/s

V = 8.2 / 4.6 = 1.78 m/s (less than allowed).

Therefore use three 1.35 m diameter pipes and the outlet velocity will not scour the outlet channel. In real life practice, several alternate culvert designs would be trialed and the design that best met all conditions including costs, cover depth, upstream and downstream water levels, etc. would be selected. It may also be useful to check the consequences associated with storm events greater than the design event occurring.

F.2 Worked example of impact dissipator

Design discharge of 8.2 m^3/s flows through a culvert consisting of 3 pipes each 1.35 m in diameter. The pipes are under inlet control with a slope of 1 in 50. The downstream channel is grass lined.

Flow conditions within culvert

Flow in each pipe = 8.2/3 = 2.7 m^3/s = (Q_p)

Capacity of the pipes when flowing full is found from Figure 6.14 and for this example each pipe can carry about 8 m^3/s (= Q_f).

Q_p/Q_f = 2.7/8 = 0.34

From Figure 7.4 d_p/D = 0.4 therefore the partial depth = 0.4 x 1.35 = 0.54 m

The velocity of flow when full = $V_f = Q_f/A = 8 / (1.35^2 \times \pi /4) = 5.59$ m/s

The velocity of the part full pipe is given in Figure 7.4 using the partial depth ratio of 0.4.

V_p/ V_f = 0.9 therefore V_p = 5.59 x 0.9 = 5.0 m/s

From Figure 7.6 at a partial depth ratio of 0.4, A/BD = 0.3 therefore A/B = 1.35 x 0.3 = 0.41

Froude No. (equation 7.1) = V_p / (g x A/B)0.5 = 5.1 / (9.81 x 0.41)0.5 = 2.5

As the Froude No. is greater than one, then it is likely a dissipator will be required.

From Figure 6.16 the critical depth is estimated to be 0.9 m (= d_c)

Flow Conditions downstream of the culvert

The channel downstream of the culvert has the geometry shown in Figure F.1.

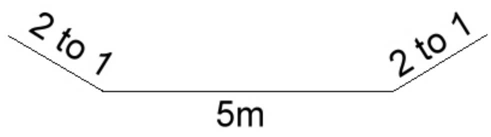


Figure F.1 Channel cross section

The cross sectional area is given by

A = (d x 2d) /2 x 2 + 5 x d where d = depth

A =d(2d +5)

Wetted perimeter (W_p) = 5 + 2(d^2 + $[2d]^2$)0.5

Hydraulic radius (R) = A/W_p

The down stream depth can be found by trial and error, balancing the discharge against the cross sectional area.

Try a depth of 0.5 m

R = 0.5(2 x 0.5 +5) / (5 + 2(0.52 + [2 x 0.5]2)0.5) = 0.415

Average velocity, V = $R^{0.667} \times S^{0.5}/n$,

assume Manning's n of 0.03

V = $0.415^{0.667} \times 0.05^{0.5} / 0.03 = 4.14$ m/s and

Q = AV = 0.5(2 x 0.5 + 5) x 4.14 = 12.4 m^3/s

Depth of 0.5 m, the discharge is too great when compared to the actual discharge of 8.2 m3/s.

Try depth of 0.25m:

R = 0.225

V = 2.76 m/s

Q = 3.8 m^3/s.

At a depth of 0.25 m the discharge is lower than actual. Interpolate linearly to estimate depth at 8.2 m^3/s: d = 0.38 m.

The depth required to support a hydraulic jump is approximately $2d_c$ = 2 x 0.9 m = 1.8 m and the tailwater depth of 0.38 m is insufficient. It could be expected that the flow would not lose significant velocity in a stilling basin and would result in scour of the grass lined channel.

Try an impact type dissipator.

Flow area in one pipe, A = Q/V = 2.7/5.1 = 0.53 m^2

Equivalent depth in a rectangular section (for one cell) $d_e = (A/2)^{0.5} = (0.53/2)^{0.5}$ = 0.514 m

Froude No. for dissipator = $V^p/(g \times de)^{0.5} = 5.1/(9.81 \times 0.514)^{0.5}$ = 2.3

Head at end of pipe, $H = de + V_p^2/2g = 0.514 + 5.1^2/(2 \times 9.81)$ = 1.84 m

From Figure 7.11 it is estimated that the height to width ratio for a Froude No. of 2.3 is 0.62, therefore, the required width is H/0.62 = 1.84/0.62 = 2.97 m. The nearest standard width in Table 7.1 is 3 m with a length of 4 m. Therefore the width of the 3 culvert pipes need to spaced over a 3 x 3 m = 9 m width. As the channel base width is 5 m, a transition zone will be required with bank and bed protection.

APPENDIX G

G.1 Worked Example of an Infiltration Basin

The basin is to be sited in an area of sandy loam (permeability of $5x10^{-5}$ m/s) which has a catchment area of 5 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. The following refers to Table G.1:

- Using the procedures in Australian Rainfall and Runoff the rainfall intensities (col. 2 of Table G.0.1) were calculated for storms of varying durations (col. 1) with a 10 year ARI.
- The Rational Method was used to estimate the runoff 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, 60 m long with a width of 25 m was adopted.
- The depth of the runoff in the basin is then calculated assuming that there are no losses (col 5) (note: the infiltrated runoff is dealt with in col.10).
- The depth in the basin is halved (col. 6).
- Half of the hydraulic radius is calculated (col. 7).
- Radius of influence is calculated (Equation 8.4) (col. 8).
- Outflow is calculated (Equation 8.3) (col. 9).
- Storage is calculated by subtracting the outflow from the inflow (col. 10).
- For the new storage volume a new depth can be calculated to check that it doesn't exceed the recommended maximum of 0.6 m.

Depth at maximum storage volume = $((w^2 + 4[bs]*V/L)^{0.5} – w)/(2bs)$

where

W width of basin floor (m)

bs batter slope (1:x)

V storage volume (m^3)

L length of basin floor (m)

Depth $= ((25^2 + 4[8] * 1070/60)^{0.5}) – 25)/(2*8)$

$= 0.60m < 0.6m$, therefore acceptable

Check the detention time (equation 8.6), $= 3/(5x10^{-5}((0.60/2+3)/3) = 54{,}500$ s

= 15.2 hours < 72 hours, therefore acceptable

Need also to check the detention time at minimum inundation level

$= 3/(5x10^{-5}((0/2+3)/3)$

= 16.7 hours > 12 hours, therefore acceptable.

Table G.0.1: Tabulation of inflow and outflow hydrographs

Storm Duration (hr)	^{10}I mm/hr	Q_{10} m^3/s	Inflow m^3	Depth m	Depth/2 m	r m	R m	Outflow m^3	Storage Required m^3
1	2	3	4	5	6	7	8	9	10
0.125	175	1.215	550	0.33	0.17	15.15	16.33	20	530
0.25	119	0.826	740	0.43	0.22	15.97	17.18	40	700
0.5	76.0	0.528	950	0.54	0.27	16.82	18.07	80	870
0.75	57.2	0.397	1070	0.60	0.30	17.29	18.56	130	940
1	46.3	0.322	1160	0.64	0.32	17.63	18.92	180	980
2	29.0	0.201	1450	0.77	0.39	18.70	20.03	380	1070
3	21.9	0.152	1640	0.86	0.43	19.36	20.73	590	1050
6	13.5	0.094	2030	1.02	0.51	20.66	22.08	1260	770
9	10.2	0.071	2300	1.13	0.56	21.52	22.97	1970	330
12	8.3	0.058	2490	1.20	0.60	22.10	23.58	2700	0
18	6.6	0.046	2970	1.38	0.69	23.50	25.05	4340	0
24	5.5	0.038	3300	1.49	0.74	24.42	26.01	6040	0

G.2 Worked Example of an On-Site Detention (OSD) Facility

This example is based upon the Rational Method.

Location Perth, WA.

Site dimensions: 40 m x 20 m = 800 m^2 consisting of 300 m^2 of impervious areas.

Slope is diagonally across the block at 3%.

Run off coefficients are 0.1 for the porous areas and 0.9 for the impervious areas.

The site lies within a small catchment which has a time of concentration of 37 minutes and the time for the peak flow from the top of the catchment to the site is 22 minutes.

Using the overland flow equation, the time of concentration for the site itself was determined at 14 minutes and the time to the OSD for the directly connected paved areas was 5 minutes.

The rainfall intensities are; $^{10}I_5$: 120 mm/hr; $^{10}I_{37}$: 40 mm/hr.

The permissible site discharge (PSD) will either be specified by the local regulatory authority or can be estimated from (further details at Department of Irrigation and Drainage Malaysia webpage)

$$PSD = \frac{a - \sqrt{a^2 - 4b}}{2}$$ where

a for above ground storage $\left(4\frac{Q_a}{t_c}\right)\left(0.333t_c\ \frac{Q_p}{Q_a} + 0.75t_c + 0.25t_{cs}\right)$ or a for below ground storage $\left(8.458\frac{Q_a}{t_c}\right)\left(0.333t_c\frac{Q_p}{Q_a}+0.35t_c+0.65t_{cs}\right)$

b for above ground storage $4Q_aQ_p$ or

b for below ground storage $8.548\ Q_aQ_p$

t_c time of concentration of catchment

t_{cs} time from top of catchment to site

Q_a peak post-development discharge from site for a storm with a duration of tc

Q_p peak pre-development discharge from site for a storm with a duration of tc.

For this example the storage requirements are small and therefore 90% of the storage will be in an underground facility.

The equivalent area for the site = [0.9 x 300 + (5/14 x 0.1 x 500)]/10,000 (conversion of area to hectares)

= [270 + 17.9]/10,000 = 0.029

Q_a = 0.029 x 40/0.36 = 3.20 L/s

Q_p = 0.1 x 800/10000 x 40 / 0.36 = 0.89 L/s.

Therefore the 'a' factors are 12.7 and 22.7 for above and below ground storage respectively.

The 'b' factors are 11.4 & 24.3 for above and below ground storage respectively.

Therefore PSD is 0.97 and 1.13 L/s for the above and below ground storage respectively.

Assume that 90% of storage is below ground therefore PSD = 0.97 x 0.1 + 1.13 x 0.9 = 1.11 L/s.

The storage required to limit the peak discharge to 1.11 L/s can be calculated based on simple hydrographs. Analysis assumes that outflow is constant until all runoff is discharged.

Table G.0.2: Inflow and outflow characteristics

Time mins	Percentage area contributing		Inflow rate (L/s)	Outflow rate (L/s)	Storage volume (L)
	Pervious	Impervious			
0	0.00	0.00	0.00	0.00	0.00
5	1.00	0.36	3.11	1.11	600
14	1.00	1.00	3.32	1.11	2452
20	0.00	0.57	0.18	1.11	1335
25	0.00	0.21	0.07	1.11	0
28	0.00	0.00	0.00	0.00	0

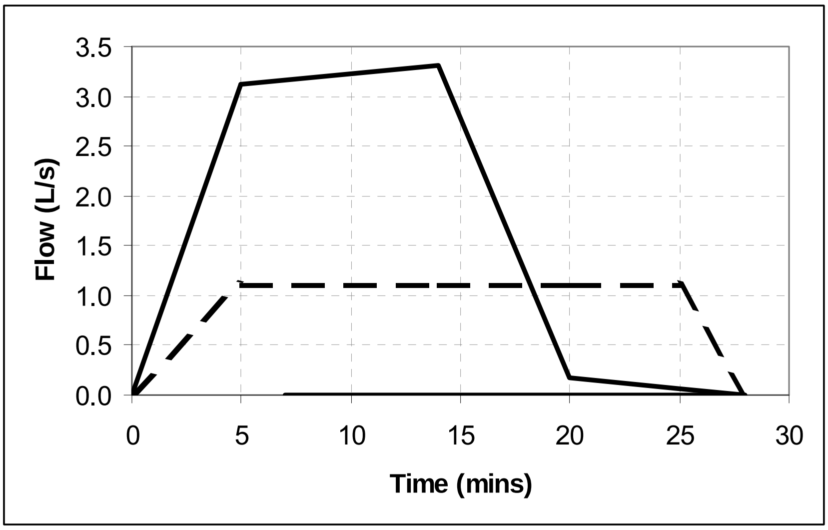


Figure G.1: Inflow hydrograph represented by solid line. Outflow hydrograph represented by dashed line.

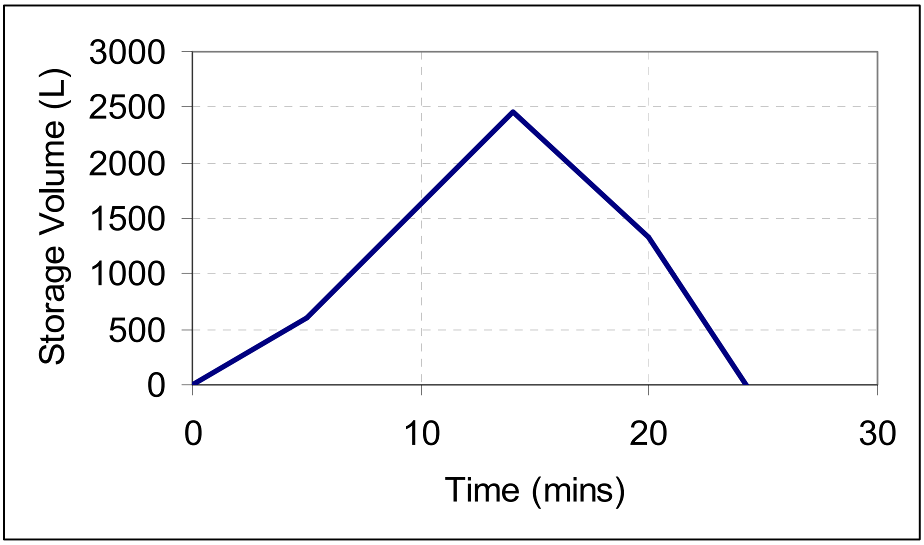


Figure G.2: Storage requirements resulting from a 14 minute storm

From the above simple analysis it can be seen that the total onsite storage is about 2500 L of which 450 L would be stored above ground. Assuming that the area to be used as above ground storage is a landscaped area then it is necessary to increase the storage volume by 20% (say to 550 L) to cater for increased vegetation with time and discrepancies in the accuracy in construction. Therefore a 2000 L underground tank is required and an area 2 m by 3 m with a depth of less than 100 mm is required for above ground storage.

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