Queensland Urban Drainage Manual

Third edition 2013—provisional

(Feedback required by 31 October 2013)

Queensland Urban Drainage Manual

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Foreword

It gives me a great deal of pleasure to present the third edition of the Queensland Urban Drainage Manual. First released in 1992, this manual remains one of the primary reference documents for stormwater practitioners in Queensland. The document also has attracted a wide use outside Queensland.

Production of this third edition originated from the state government's response to the recommendations of the Queensland Floods Commission of Inquiry. However, the government did not limit the update to just those issues raised within the inquiry; it also addressed various issues raised by the industry during the consultation phase.

There are, however, a few of issues—particularly in reference to inter allotment drainage—that remain unresolved. Consequently the government has decided to release this edition as a provisional version. Further consultation will occur throughout 2013 with a final version due in late 2013. In the meantime, stormwater designers should consider this edition as representing current best practice, and local governments should give appropriate consideration to the recommendations of this manual when developing their drainage codes.

This edition sees an increased focus on building communities and stormwater systems that are more resilient to severe storms—a key thrust of the Floods Inquiry recommendations. No longer should stormwater designers limit their considerations to the nominated 'Major Storm' event. Appropriate consideration must be given to the impact of severe storms to ensure that the consequences are acceptable, and the community is able to quickly return their lives and businesses to a state of normality after such events.

The expanding objectives of stormwater management have meant that this manual must continue to be used in partnership with other design manuals on topics such as floodplain management, total water cycle management, water sensitive urban design, and natural channel design.

I believe this provisional third (2013) edition of the Queensland Urban Drainage Manual provides stormwater managers with an extensive guideline on current best practices for the planning and design of urban drainage systems that aid in improving the state's resilience to flooding and drainage problems associated with severe storms.

Honourable Mark McArdle MP
Minister for Energy and Water Supply

Contents

Preface

In March 2012 the Queensland Floods Commission of Inquiry presented its final report to the Premier of Queensland. The recommendations contained within this report, specifically recommendation 10.8, suggested the Queensland Urban Drainage Manual (QUDM) be reviewed 'to determine whether it requires updating or improvement, in particular, to reflect the current law and to take into account insights gained from the 2010/2011 floods'.

This recommendation not only implied QUDM should be updated to reflect the outcomes of the Inquiry, but also any other relevant insights gained from other sources in regards to the 2010–11 floods. As a consequence, the development of this third edition of QUDM has involved extensive literature reviews and consultation with local governments across Queensland.

The recommendations from the Queensland Floods Commission of Inquiry's final report (the report) that are considered most relevant to the QUDM are summarised below:

- The need to update QUDM with respect to current legislation (Recommendation 10.8).
- The need for improved consideration of flows in excess of the nominated major storm (Recommendation 2.13).
- The need to design stormwater systems to improve the state's resilience to extreme storm and flood events (general discussion within Chapter 2 of the report).
- The need for greater consideration of flood protection of essential community infrastructure and the management of flood evacuation routes (Recommendations 7.24, 7.25, 8.7, 10.11 & 10.20). Even though QUDM is not intended as a floodplain management guideline, it does provide guidance on design standards for cross drainage structures such as culverts, which is linked to the flood immunity of some evacuation routes.
- The need for better design guidance on preventing the flooding of commercial buildings, basements and non-habitable floors of buildings (Recommendation 7.4). The link to QUDM is through the setting of freeboards for major storm flows along roads.
- The need for better design guidance on the management of flood impacts on areas of manufacture or storage of bulk hazardous materials (Recommendations 7.11 & 7.13). The link to QUDM is through the design of overland flow paths that pass through industrial areas.
- The need for better guidance on the design and usage of stormwater backflow devices (Recommendation 10.14).

Based in part on the above report recommendations, the main outcomes of the 2012–13 review of QUDM are summarised below:

- Increased emphasis on investigating the consequences of flows in excess of the major storm design discharge. It is noted that this does not necessarily mean the design standard has increased, or that a drainage system designed to the 2013 standard will be measurably different to one designed to the 2007 standard.
- Increased use of the *annual exceedence probability* (AEP) to define design storms.
- Introduction of the concept of *Severe Storm Impact Statements* as a part of the consideration of flows in excess of the major storm design discharge.
- Recognition of the growing importance of Regional Flood Frequency Methods in the analysis of ungauged rural catchments.
- Improved discussion on planning issues for stormwater detention and retention; and removal of the *initial sizing* equations that previously existed in the first and second editions of QUDM.

- Recognition that when flows, previously passing longitudinally along a roadway, spill across the roadway (i.e. at a change in road crossfall) a higher drainage standard may be required.

- Increased discussion on the blockage factors applied to stormwater systems based on the reports of Australian Rainfall and Runoff Project 11.

- Inclusion of an overview of rock sizing equations for use in the design of scour protection within drainage structures.

- Improved procedures for assessing the safety risks associated with stormwater inlets.

The QUDM partners recognise that this Manual is not a stand-alone planning and design guideline for stormwater management. It must be used in coordination with other recognised manuals covering topics such as:

- Floodplain management policies/guidelines

- Water Sensitive Urban Design

- Water Sensitive Road Design

- Natural Channel Design

- Waterway management including fauna passage

- Erosion & Sediment Control

- Bridge and culvert design manuals

- Australian Rainfall and Runoff (ARR)

- Australian Runoff Quality (ARQ)

- various Australian Standards on product manufacture and installation

Whilst there are significant areas of overlap, QUDM is **not** intended to act as a floodplain management manual. Where appropriate, this Manual directs stormwater designers and regulators to other publications for information on floodplain management issues.

The information presented within this edition of QUDM on stormwater quality treatment and the management of environmental impacts is not comprehensive and should not be used to supersede other more comprehensive and locally relevant manuals and guidelines.

Acknowledgments

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List of tables

List of figures

Appendix 1 – Pipe flow design charts

Appendix 2 – Structure pressure change coefficient charts

A2-18   Water surface elevation coefficients (K_w) for 45° bends at pit junctions with branch point located on the upstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-19   Water surface elevation coefficients (K_w) for 45° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-20   Pressure head change coefficients (K_u) for 67.5^o bends at pit junctions with branch point located near the downstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-21   Pressure head change coefficients (K_u) for 67.5^o bends at pit junctions with branch point located near the downstream face of pit for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-22   Water surface elevation coefficients (K_w) for 67.5^o bends at pit junctions with branch point located near the downstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-23   Water surface elevation coefficients (K_w) for 67.5^o bends at pit junctions with branch point located near the downstream face of pit for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-24   Pressure head change coefficients (K_u) for 67.5^o bends at pit junctions with branch point located near the upstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-25   Pressure head change coefficients (K_u) for 67.5^o bends at pit junctions with branch point located near the upstream face of pit for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-26   Water surface elevation coefficients (K_w) for 67.5^o bends at pit junctions with branch point located near the upstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-27   Water surface elevation coefficients (K_w) for 67.5^o bends at pit junctions with branch point located near the upstream face of pit for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-28   Pressure head change coefficients (K_u) for 90° bends at pit junctions for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

A2-29   Pressure head change coefficients (K_u) for 90° bends at pit junctions for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-30   Water surface elevation coefficients (K_w) for 90° bends at pit junctions for a submergence ratio, $S/D_o = 2.5$ (Source: Hare, 1980)

A2-31   Water surface elevation coefficients (K_w) for 90° bends at pit junctions for submergence ratios $S/D_o = 1.5, 2.0, 3.0$ and 4.0 (Source: Hare, 1980)

A2-32   Rectangular inlet with in-line upstream main and 90° lateral pipe, with or without grate flow (Source: DOT, 1992)

A2-33   Rectangular pit with opposed lateral pipes each at 90° to outlet, with or without grate inflow (Source: DOT, 1992)

A2-34   Rectangular pit with offset opposed lateral pipes each at 90° to outlet, with or without grate inflow (Source: DOT, 1992)

A2-35   Pressure loss coefficients for a circular junction pit with upstream pipe entering at angles from 0° to 90° (Source: Cade and Thompson, 1982)

A2-36   Pressure head change coefficients for drop chambers with upstream pipe entering above the outfall pipe and at angles of 0° to 90° (Source: Cade and Thompson, 1982)

A2-37   Pressure head change coefficients (K_L) for 90° lateral inflow pipe (Source: DOT, 1992)

A2-38   Pressure head change coefficients (K_u) for through flow pipeline at junction of 90° lateral inflow pipe (Source: DOT, 1992)

A2-39 Pressure head change coefficients (Ku & KL) for through flow pipeline at junction of 90° lateral inflow pipe for conditions outside the range of Charts A2-37 & 38 (Source: DOT, 1992)

A2-40 'Energy loss' coefficients for lateral and upstream pipe for a non-chamber junction with branch angle of 15° (Source: Miller, 1978)

A2-41 'Energy loss' coefficients for lateral and upstream pipe for a non-chamber junction with branch angle of 30° (Source: Miller, 1978)

A2-42 'Energy loss' coefficients for lateral and upstream pipe for a non-chamber junction with branch angle of 45° (Source: Miller, 1978)

A2-43 'Energy loss' coefficients for lateral and upstream pipe for a non-chamber junction with branch angle of 60° (Source: Miller, 1978)

A2-44 'Energy loss' coefficients for lateral and upstream pipe for a non-chamber junction with branch angle of 90° (Source: Miller, 1978)

Appendix 3 – Road flow capacity charts

1. Introduction

1.1 Use of this manual

This Manual has been prepared for the purpose of assisting engineers and stormwater designers in the planning and design of urban drainage systems within Queensland. Reference to this document as a Manual should not infer that it is anything more than an engineering guideline.

The procedures outlined in the Manual aim to encourage uniformity in urban drainage design practices throughout Queensland. Designers are nevertheless responsible for conferring with relevant local authorities to determine local design requirements.

The aim of the Manual is to provide details of technical and regulatory aspects to be considered during the planning, design and management of urban stormwater drainage systems, and to provide details of appropriate design methods and computational procedures. Both hydrologic and hydraulic procedures are considered as well as environmental and legal aspects.

The prime objective of Queensland Urban Drainage Manual (QUDM) is to address:

- the design of stormwater conveyance structures (not water quality) that exist from the down-slope allotment boundary to the edge of the defined watercourse
- the hydraulic design of structures that cross floodplains, such as constructed open drains and cross-drainage structures.

The Manual does not present comprehensive advice on waterway management or floodplain management issues, but does address issues relating to stormwater flooding and the management of overland flow paths.

Changes introduced into this edition of QUDM are intended to improve the state's resilience to severe storms and climate change through the proper design and management of urban drainage systems.

The hydrologic procedures provided in the Manual are considered appropriate for small urban catchments of up to 500 hectares. These procedures are generally not considered appropriate for the determination of design flood levels along large vegetated (non-grassed) waterways. Readers should refer to the latest version of Australian Rainfall & Runoff (ARR) for guidance on:

- the assessment of urban catchments larger than 500 hectares
- the determination of design flood levels along vegetated waterways
- the hydrologic assessment of gauged and ungauged rural catchments.

Even though the focus of the Manual is on urban drainage systems, the regulating authorities may specify that parts of the Manual shall apply to the design of specific aspects of rural drainage systems, including road works. It would, however, not be considered appropriate for the design standards presented within the Manual to automatically be applied to the design of minor service roads, unless specifically required by the regulator or asset manager.

Use of this Manual requires professional interpretation and judgement to ensure the guidelines are appropriately adapted to local conditions. The document is not a recipe book for persons acting outside their field of competence or experience. Users of the Manual must make informed decisions regarding the extent to which the guidelines are applied to a given situation, including appropriate consideration of local conditions and local data.

Throughout the Manual, use of the term 'should' shall imply that all reasonable and practicable measures must be taken to achieve the intent/outcome of the clause in question. If the Manual refers to a specific action or task, then an alternative solution may be adopted provided it has an outcome or performance at least equivalent to that presented in that particular clause of the Manual. Where it is not considered reasonable or practicable to achieve the intent/outcome, the designer may be required to provide—to the satisfaction of the regulating authority—justification for the decision.

The Manual is not to be regarded as prescriptive. There will be circumstances and conditions where designers will need to adopt alternative design procedures, or innovative methods, commensurate with accepted engineering and scientific practice.

Regulating authorities may require designers to certify that they have designed and documented their proposed stormwater systems in accordance with this Manual, or at least to a standard no less than that presented in the Manual.

The Manual does not address catchment or regional planning, floodplain management, or provide detailed procedures for the design of stormwater treatment systems, waterway rehabilitation, or Natural Channel Design (NCD).

The reader should refer to the *Glossary of terms* (Chapter 13) for the distinction this document makes between the terms regulating authorities, local authorities and local governments. In most cases the term local authority will refer to either the local government or the State Government depending on which body has jurisdiction over specific activities on the land. Readers should also refer to the Glossary for the definition of a wide range of common industry related terms used within the Manual.

Any general reference to an external guideline, document or publication shall infer reference to the latest version of that publication or its replacement document.

1.2   Consideration of regional factors

An endeavour has been made in the preparation of this Manual to make it applicable across the wide variety of geologic and climatic conditions existing throughout Queensland. Issues that may influence the appropriate application of this Manual to local conditions include:

- local community expectations and their relative tolerance of drainage and flooding issues
- variations in the design standards specified by various local governments
- a local government's ability, preference and willingness to fund various stormwater infrastructure construction, operational and maintenance activities
- regional climatic factors
- the types of receiving environments, including variations in ecological characteristics
- local geologic and soil conditions, e.g. natural nutrient sources and sinks, and variations in stormwater infiltration rates
- variations in pollutant runoff rates—collection and use of local data is always preferred
- variations in local building regulations and architectural design
- historic factors and the success of specific past practices within a given region.

1.3   Objectives of stormwater management

The primary aim of an urban stormwater management system is to ensure stormwater generated from developed catchments causes minimal nuisance, danger and damage to people, property and the environment. This requires the adoption of a multiple objective approach, considering issues such as (ARMCANZ and ANZECC, 2000):

- ecosystem health, both aquatic and terrestrial
- flooding and drainage control
- public health and safety
- economic considerations
- recreational opportunities
- social considerations
- aesthetic values.

The above issues may be developed into a list of broad stormwater management objectives. Each of the objectives presented below may not be relevant in all circumstances, and individual objectives may need to be expanded to focus on site-specific issues.

- Protect and/or enhance downstream environments, including recognised social, environmental and economic values, by appropriately managing the quality and quantity of stormwater runoff.
- Limit flooding of public and private property to acceptable or designated levels.
- Ensure stormwater and its associated drainage systems are planned, designed and managed with appropriate consideration and protection of community health and safety standards, including potential impacts on pedestrian and vehicular traffic.
- Adopt and promote water sensitive design principles, including appropriately managing stormwater as an integral part of the total water cycle, protecting natural features and ecological processes within urban waterways, and optimising opportunities to use rainwater/stormwater as a resource.
- Appropriately integrate stormwater systems into the natural and built environments while optimising the potential uses of drainage corridors.
- Ensure stormwater is managed at a social, environmental and economic cost that is acceptable to the community as a whole, and that the levels of service and the contributions to costs are equitable.
- Enhance community awareness of, and participation in, the appropriate management of stormwater.

These objectives may need to be addressed in a number of different contexts depending on the degree of past catchment changes and the potential for future change. Such contexts would include the following:

- retaining or restoring natural stormwater systems
- rehabilitating existing stormwater systems to ecologically sustainable, but not necessarily natural, systems
- creating new, ecologically sustainable, stormwater systems within heavily modified environments.

In order to achieve the key objectives of stormwater management, designers need to appropriately manage several different design parameters associated with stormwater. These parameters and the desired outcomes are outlined in Table 1.3.1.

Table 1.3.1 – Key stormwater parameters and desired outcomes

Parameter	Desired outcomes
Drainage efficiency	• Public health (e.g. mosquito control) • Pedestrian and vehicular safety • Minimisation of storm-related nuisance to the public
Flood control	• Urban communities protected from flooding • Pedestrian and vehicular safety • Resilient to severe floods in excess of nominated design events
Runoff volume	• Flood control • Control of bed and bank erosion in waterways • Reduction in annual pollutant load to waterways • Optimum use of stormwater as a resource • Protection of aquatic ecosystems within receiving waters
Peak discharge	• Flood control • Minimisation of legal disputes between neighbouring landowners and communities • Control of bed and bank erosion in waterways
Flow velocity	• Flood control within downstream waterways • Pedestrian and vehicular safety • Control of bed and bank erosion in waterways • Protection of aquatic ecosystems within receiving waters
Flow depth	• Flood control • Pedestrian and vehicular safety • Minimisation of storm-related nuisance to public
Water quality	• Protection of aquatic ecosystems and public health • Optimum use of stormwater as a resource • Structural integrity of waterways through the control of sediment inflows
Aesthetics	• Attractive urban landscapes • Retention of natural drainage systems • Protection/restoration of environmental values
Infrastructure and maintenance cost	• Acceptable financial cost • Sustainable operational and maintenance requirements • Stormwater systems resilient to damage from severe flood events

Stormwater managers and designers should be aware that the establishment of engineered infrastructure—whilst still central to the delivery of stormwater management outcomes—is not the entire picture. There is a much wider range of measures that are used in addressing stormwater management issues (such as community education and enforcement of regulations) to ensure objectives are met, particularly in respect to water quality. This wider range of measures make-up an overall Urban Stormwater Management Strategy (refer to section 2.2).

The planning and design of stormwater management systems must appropriately integrate the following management philosophies:

- Integrated Catchment Management (ICM)
- Total water cycle management (TWCM)
- Best practice floodplain management
- Ecologically Sustainable Development (ESD)
- Water Sensitive Urban Design (WSUD)
- Building and construction phase Erosion and Sediment Control (ESC)
- Best Management Practice (BMP)

Stormwater planners also need to ensure they meet the expectations of higher levels of government expressed through state legislation and national agreements. Such expectations include the National Water Initiative and the National Framework for the Management of Water Quality presented within the National Water Quality Management Strategy (NWQMS).

1.4 Integrated Catchment Management

Integrated Catchment Management (ICM) incorporates catchment-wide relationships that aim to integrate and improve land, water and related biological resources for the purpose of achieving the sustainable use of these resources. It embraces (ARMCANZ & ANZECC, 2000a):

- a holistic approach to natural resource management within catchments, marine environments and aquifers, with linkages between water resources, vegetation, land use, and other natural resources recognised
- integration of social, economic and environmental issues
- co-ordination between all the agencies, levels of government and interest groups within the catchment
- community consultation and participation.

It is through an ICM process that stormwater managers will be able to appropriately integrate proposed stormwater management practices with other geomorphologic, ecologic, soil, land use and cultural issues within a drainage catchment. The outcome of an ICM process is often the development of a Catchment Management Plan or Strategy.

1.5 Total Water Cycle Management

Total Water Cycle Management (TWCM) recognises water as a valuable and finite resource that must be managed on a total water cycle basis. Unlike the ICM process that integrates water resources with other catchment-based resources, the TWCM process aims to integrate stormwater planning with the planning units of other water industries.

TWCM recognises that:

- All aspects of the water cycle (e.g. water supply, wastewater, stormwater, groundwater and environmental flows) within a catchment are interdependent.
- The management practices applied to any single component of the water cycle must appropriately integrate with all other elements.
- Infrastructure planning within any component of the water cycle must appropriately integrate with all other components of the water cycle.

Key to the TWCM process is the development of a TWCM Plan, which outlines a local government's TWCM strategy and implementation plan.

1.6 Best practice floodplain management

QUDM is **not** intended to act as a floodplain management manual. Stormwater designers and regulators are directed to the following publications if information is required on floodplain management issues.

- Queensland Department of Local Government 2012, *Mitigating the Adverse Impacts of Flood, Bushfire and Landslide*, State Planning Policy 1/03 (replaced in 2013).

- Queensland Natural Resources and Mines 2002, *Guidance on the Assessment of Tangible Flood Damages*

- CSIRO 2000, *Floodplain Management in Australia – Best Practice Principles and Guidelines*, SCARM Report 73, CSIRO Publishing, Victoria.

- Zevenbergen, C. et al. 2008, *Urban Flood Management,* CRC Press/Balkema, The Netherlands.

Zevenbergen et al. (2008) provides a European perspective to floodplain management and thus does necessarily represent the focus and direction recommended by the Queensland Government. It does, however, provide alternative concepts that may assist floodplain managers to discover site specific solutions to site specific problems.

1.7 Ecologically Sustainable Development

Ecologically Sustainable Development (ESD) aims to meet the needs of existing communities, while conserving ecosystems for the benefit of future generations. This is achieved by designing management systems and new developments that improve the total quality of life in a way that maintains the ecological processes on which life depends.

While there is no universally accepted definition of ESD, in 1990 the Australian Government suggested the following definition for ESD in Australia:

'Using, conserving and enhancing the community's resources so that ecological processes, on which life depends, are maintained, and the total quality of life, now and in the future, can be increased.'

The principles of ESD as outlined in ARMCANZ & ANZECC (2000a) are:

- *The precautionary principle.* Namely, that if there are threats of serious or irreversible environmental damage, lack of full scientific certainty should not be used as a reason for postponing measures to prevent environmental degradation.

- *Inter-generational equity.* The present generation should ensure that the health, diversity and productivity of the environment are maintained or enhanced for the benefit of future generations.

- *Conservation of biological diversity and ecological integrity.* Conservation of biological diversity and ecological integrity should be a fundamental consideration.

- *Improved valuation, pricing and incentive mechanisms.* Environmental factors should be included in the valuation of assets and services.

1.8 Water Sensitive Urban Design

Water Sensitive Urban Design (WSUD) is a holistic approach to the planning and design of urban development that aims to minimise negative impacts on the natural water cycle and protect the health of aquatic ecosystems. It promotes the integration of stormwater, water supply and sewage management at the development scale. The aims/objectives of WSUD are to:

- protect existing natural features and ecological processes

- maintain natural hydrologic behaviour of catchments
- protect water quality of surface and ground waters
- minimise demand on the reticulated water supply system
- minimise sewage discharges to the natural environment
- integrate water into the landscape to enhance visual, social, cultural and ecological values.

It is recommended that the principles of WSUD are applied wherever practical to greenfield urban developments as well as to infill developments and urban redevelopment programs.

1.9   Erosion and sediment control

This Manual does not present guidelines on the design and application of *erosion and sediment control* principles for construction and building sites; however, the importance of these pollution control measures to stormwater quality is recognised.

The need to protect permanent stormwater treatment systems from the adverse effects of sediment runoff during the construction phase of new development is also recognised as **critical** if these systems are to operate satisfactorily after the construction phase has been completed.

Practitioners are referred to IECA (2008) for guidance on *erosion and sediment control* practices and the management of stormwater on building and construction sites. IECA (2008) also provides expanded discussion on the application of hydrology and hydraulics to construction site stormwater management.

1.10   Best management practice

Best management practice (BMP) refers to the design, construction and financial management of an activity which achieves an ongoing minimisation of the activity's environmental harm through cost effective measures assessed against the measures currently used nationally and internationally for the activity.

BMP in stormwater quality management includes a broad range of treatment measures from those with a highly predictable performance outcome, to those that can be assumed to be beneficial, but for which a clear and predictable performance outcome has yet to be developed.

As noted previously in section 1.7, *'if there are threats of serious or irreversible environmental damage, lack of full scientific certainty should not be used as a reason for postponing measures to prevent environmental degradation'*. Adoption of current best management practice is required to ensure the delivery of an acceptable stormwater management system.

1.11   Principles of stormwater management

The recommended 'objectives' of an urban stormwater management system are presented in section 1.3. The following discussion expands on those objectives to develop a set of key principles that outline the current (2013) approach to the management of urban stormwater.

The following principles are presented as an overview and have been provided for educational purposes. Not all of the principles are equally appropriate in every situation. The appropriate application of these principles requires experience and professional judgement. For example, even though it is highly desirable to ensure that the maintenance requirements and costs of a stormwater system are sustainable, it is not reasonable to expect a stormwater designer to conduct a detailed financial and technical capabilities study of the proposed asset manager (usually the local government) prior to designing the system. Also, in many cases the responsibilities of the

designer will be limited by the requirements of the various design codes adopted by the local authority.

However, the above discussion does not negate the expectation that the designer will adopt a professional approach and seek such additional information from the local authority and/or client as necessary to facilitate a thorough design. For example, the designer should seek resolution of any unspecified parameters or issues considered relevant to the outcome of the design.

1.11.1   Protect and/or enhance downstream environments, including recognised social, environmental and economic values, by appropriately managing the quality and quantity of stormwater runoff

(i)   *Minimise changes to the quality and quantity of the natural flow regime of urban waterways*

The focus of stormwater management should not concentrate solely on the control of flow velocity and peak discharge, but also on minimising changes to a catchment's natural water cycle—including the volume, rate, frequency, duration and velocity of stormwater runoff (refer to the expanded discussion in Chapter 3).

By minimising changes to runoff volume, and thereby minimising changes to the natural water cycle, the following economic, ecological and social benefits are likely to be gained:

* reduced pollutant runoff
* reduced risk of increases in downstream flooding
* reduced risk of accelerated erosion within urban waterways
* reduced cost of providing stormwater detention systems within new urban developments
* improved health of aquatic ecosystems through the replenishment of natural groundwater supplies
* reduced demand on the provision of new potable water supplies through the use of stormwater as a secondary (non-potable) water supply.

(ii)   *Identify and control the primary sources of stormwater pollution*

The selection and design of stormwater treatment systems needs to be based on local data that adequately reflects local conditions, land use practices and community values. The focus should firstly be on assessing and/or ranking the threats to the identified local values, then developing treatment systems commensurate with 'actual' rather than 'perceived' risks.

In most urban environments the greatest threat to stormwater quality will usually be associated with:

* Stormwater runoff from soil disturbances such as building and construction sites. On a site-by-site basis this may be a short-term activity, but across a developing catchment it can represent a long-term threat.

* Stormwater runoff from roads and car parks, particularly those areas where there is significant turning and braking by motor vehicles, such as off ramps, intersections and roundabouts.

(iii)   *Develop stormwater systems based on a preferred management hierarchy*

The preferred hierarchy for the selection of stormwater management practices is:

* Retain and restore (if degraded) existing valuable elements of the natural drainage system, such as natural channels, wetlands and riparian vegetation.

- Implement source control measures using non-structural techniques to limit changes to the quality and quantity of stormwater at the source of change.

- Implement source control measures using structural techniques to limit changes to the quality and quantity of stormwater at or near the source of change.

- Install in-system constructed management techniques within stormwater systems to manage stormwater quality and quantity prior to discharge into receiving waters.

(iv)   *Develop robust stormwater treatment systems that do not rely on a single treatment system or focus on a single target pollutant*

To achieve the best results, stormwater quality treatment systems should always be part of a comprehensive approach to controlling stormwater pollution. Such an approach would include regulation and enhanced community awareness, as well as structural controls.

Wherever practical, stormwater treatment systems should incorporate diversity so that the failure of one type of treatment system does not result in a total system failure.

Stormwater treatment systems should also incorporate an appropriate balance of primary, secondary and tertiary treatment measures (refer to section 11.4.3) so that the system is capable of working efficiently on a variety of pollutants over a wide range of expected storm intensities.

1.11.2   Limit flooding of public and private property to acceptable or designated levels

(i)   *Limit the frequency and severity of flooding of public and private assets to appropriate levels given the community expectations and the community's ability and willingness to afford such flood protection*

The degree of resources used to achieve flood protection depends on many factors including: community expectations; social, environmental and economic considerations; site limitations; and the assessed flood risk. The latter incorporates both the likelihood and consequences of flooding.

(ii)   *Take all reasonable and practicable measures to enhance the State's resilience to all floods, including those that exceed specified design standards*

It should be recognised that the costs associated with severe flooding can extend far beyond just the affected drainage catchment. These costs can include long-term increases in flood insurance, the cost of rebuilding major state infrastructure, the cost of rehabilitating stormwater quality devices damaged by floodwaters, impacts on employers and employees associated with the temporary closures of businesses, and the expenditure of state and federal disaster relief funding.

Stormwater designers need to be aware of those measures they can take to design stormwater systems so that the following outcomes are achieved:

- impacts at a 'local level' are acceptable to the community

- potential flow-on effects to the 'wider community' in terms of recovery time and use of emergency services resources are both affordable and acceptable to the community.

(iii)   *Preserve the alignment and capacity of major drainage corridors such as waterways and major overland flow paths*

Flood risks are not limited to just floodplains. Property flooding and public safety risks can occur in any area subject to stormwater runoff. A large part of urban drainage design focuses on the design

and management of overland flow paths, particularly major overland flow paths that receive stormwater runoff from more than one property.

These drainage corridors require sufficient land allocation, and must be recognised as a legitimate land use that needs to be appropriately considered during the planning of new urban developments and the redevelopment of existing urban areas.

1.11.3   Ensure stormwater and its associated drainage systems are planned, designed and managed with appropriate consideration and protection of community health and safety standards, including potential impacts on pedestrian and vehicular traffic

(i)   *Establish and maintain a safe, affordable and socially equitable and acceptable level of urban drainage and flood control*

Management objectives for the minimisation of public health and safety risks can include:

* designing urban drainage systems to minimise the existence of dangerous waters and the risk of people entering or being trapped within such waters

* minimising the risk of injury to the public and maintenance personnel resulting from the operation and maintenance of stormwater systems

* minimising public risks associated with such things as mosquitoes and water-borne diseases.

1.11.4   Adopt and promote water sensitive design principles, including appropriately managing stormwater as an integral part of the total water cycle, protecting natural features and ecological processes within urban waterways, and optimising opportunities to use rainwater/stormwater as a resource

(i)   *Minimise the quantity of directly connected impervious surface area*

There is growing evidence (Maxted & Shaver, 1996 and Walsh, et al. 2004) linking the risk to aquatic wildlife in urban waterways to the degree of directly connected impervious surface area.

Minimising the total impervious surface area helps to reduce changes to the natural water cycle, pollutant runoff rates and the cost of providing stormwater management systems.

The adverse effects of increased impervious surface area can be further mitigated by minimising those areas that have a direct connection to an impervious drainage system. Surrounding impervious surfaces with a porous surface will reduce pollutant runoff, increase stormwater infiltration, and improve the quantity and quality of dry weather flows within urban streams through improved groundwater inflows. Where practical, stormwater runoff from roads and roofs should first pass as sheet flow over a grassed surface before being concentrated within a drain, whether or not the drain is lined with pervious or impervious materials.

(ii)   *Identify and optimise opportunities for stormwater to be valued and used as a resource*

Stormwater planning should be integrated with water supply and wastewater strategies during the planning and design of urban developments in a manner that uses water in a resource sensitive and ecologically sustainable manner.

Better management of the water cycle, both within a local and regional context, needs to be achieved to reduce demand on traditional water supplies. Where circumstances allow, urban

stormwater can be used to recharge aquifers provided groundwater quality is protected. This requires very careful management as potential issues include rising water tables, salinity problems and disputes over groundwater extraction rights.

The 'natural' stormwater drainage system can also provide social, environmental and economic resources. The loss or modification of natural urban streams can adversely affect the amenity of surrounding areas, ecological health and water quality.

(iii)   *Maintain and protect natural drainage systems and their ecological health*

The traditional focus of stormwater management has broadened to embrace issues of aquatic ecosystem and waterway health, including environmental flows, channel stability and the protection of riparian values.

Wherever practical, natural drainage channels and flow corridors should be preserved and/or rehabilitated to maintain the natural passage and flow times of stormwater through a catchment.

Effective protection of the natural drainage system and its ecological health not only relies on maintaining the pre-development catchment hydrology and pollutant export rates, but also on:
- maximising the value of indigenous riparian, floodplain and foreshore vegetation
- maximising the value of physical habitats for aquatic and riparian fauna within the stormwater system.

It is noted that the control of building and construction site soil erosion and sediment runoff is essential for the sustainable management of most natural drainage systems. Local governments wishing to embrace the principles of Natural Channel Design **must** be prepared to actively control sediment runoff from building and construction sites.

1.11.5   Appropriately integrate stormwater systems into the natural and built environments while optimising the potential uses of drainage corridors

(i)   *Ensure adopted stormwater management systems are appropriate for the site constraints, land use and catchment conditions*

Stormwater management practices should reflect proposed land use practices, climatic conditions, soil properties, site constraints, identified environmental values, and the type of receiving waters.

Certain land uses produce concentrations of specific stormwater pollutants, thus requiring the adaptation of specialist stormwater treatment systems that may not be as effective within other areas of the catchment.

Certain receiving waters may also be sensitive to certain pollutant inflows, thus requiring a further refinement to the list of preferred stormwater management systems. As a general guide, large receiving water bodies, such as lakes, rivers and bays, benefit from any and all measures that reduce total pollutant loads, independent of 'when' the pollutant runoff occurs. On the other hand, small receiving water bodies, such as ponds, wetlands and creeks, greatly benefit from stormwater systems that produce:
- high quality inflows during regular minor storm events
- persistent high quality groundwater inflows during the days or weeks following the less frequent larger storm events.

Maintaining the natural infiltration rates of rainwater into the catchment soils can greatly benefit the ecological health of urban creek systems by helping to maintain natural groundwater inflows into these creeks. Thus the design of the stormwater system must reflect local soil conditions and their natural infiltration rates. In essence, the type of stormwater system utilised within a 'black soil' region of Queensland is likely to be very different from one used within a 'red soil' or 'sandy soil' region.

(ii)   *Appropriately integrate both wildlife and community land use activities within urban waterway and drainage corridors*

Waterways and drainage corridors can represent the most abundant, if not important, wildlife (terrestrial and aquatic) habitat areas and movement corridors within the urban landscape. These *values* can be greatly diminished if not appropriately integrated with the human activities, both passive and active, planned for the area. The development of an inter-catchment Wildlife Corridor Map is a highly desirable prerequisite to the development of an Open Space Plan, Master Drainage Plan or Waterway Corridor Map (refer to Figure 2.1 and section 2.9).

Urban waterways can also represent important vegetation conservation areas, sometimes requiring the protection of a corridor width greater than that required for flood control.

1.11.6   Ensure stormwater is managed at a social, environmental and economic cost that is acceptable to the community as a whole, and that the levels of service and the contributions to costs are equitable

(i)   *Assess the economics of stormwater management systems on the basis of their full lifecycle costs (i.e. capital and operational costs)*

Stormwater management systems should be based on solutions that are economically sustainable.

Developers of new urban communities must give appropriate consideration to the anticipated ongoing operational (maintenance) costs of stormwater management systems even if they are not required to furnish such maintenance costs.

Similarly, asset managers (including local governments) must, wherever practical, give appropriate consideration to the capital cost of new stormwater systems and the equitable flow-on costs to the community, even if they are not responsible for the initial funding of the system.

(ii)   *Ensure adopted stormwater management systems are sustainable*

Stormwater designers have a responsibility, within reason, to ensure that their designs can function effectively throughout their specified design life based on the financial and technical abilities of the proposed asset manager. Such consideration should include:

- safety of the operating personnel
- availability of required maintenance equipment
- the expected technical knowledge of the asset managers, especially for systems intended to remain in private ownership
- the provision of suitable maintenance access.

Where practical, stormwater treatment systems should separate high-maintenance and low-maintenance systems so that the function and aesthetics of the low-maintenance systems are not compromised by the regular disturbance of adjacent high-maintenance systems.

(iii)   *Ensure appropriate protection of stormwater treatment measures during the construction phase*

Stormwater treatment measures, especially filtration and infiltration systems, need to be isolated or otherwise protected during the construction phase of urban development so that their ultimate function is not compromised by sediment or construction damage.

1.11.7   Enhance community awareness of, and participation in, the appropriate management of stormwater

(i)   *Engage the community in the development of parameters for the development and evaluation of stormwater management solutions*

Stormwater management should focus on a 'value system' where the identified values are used to set priorities and rank design objectives. Community values are constantly changing and stormwater managers should ensure that the adopted values reflect both current and, to the maximum degree practical, expected future community values.

Community participation helps to (ARMCANZ & ANZECC, 2000b):
- identify strategies which are responsive to community concerns
- explore problems, issues, community values and alternative strategies openly
- increase public ownership and acceptance of proposed solutions
- generate broader decision making perspectives not limited to past practices or interests
- reflect the community's life style values and priorities.

2.   Stormwater planning

The purpose of this chapter is to assist local governments in the development of an integrated set of management plans to ensure the delivery of a holistic Stormwater Management Strategy.

2.1   General

The long-term impact of stormwater runoff on both the natural and built environments greatly depends on the extent to which stormwater issues are integrated into the overall urban planning process.

Stormwater planning may be used to define the following outcomes:

- The objectives of stormwater systems (e.g. should the primary focus be on flood control, water quality, stormwater harvesting, the adoption of low cost solutions, or a combination thereof).

- Those planning options for improving the State's resilience to severe storms, including the resilience of stormwater infrastructure to flood damage.

- The preferred stormwater systems and design standards for greenfield and infill developments.

- The objectives and design standards for stormwater upgrades and relief drainage schemes.

- Funding needs, cost constraints and a ranking system for retro-fitting existing drainage networks.

- The means of providing stormwater infrastructure in an equitable manner for all landowners within a catchment.

- The required protection of environmental values.

- The means of optimising existing opportunities for the placement of stormwater infrastructure.

The strategic stormwater planning undertaken by individual local governments and regional bodies should occur within an Integrated Catchment Management framework in cooperation with all relevant stakeholders.

The planning of stormwater systems needs to be integrated with land use planning (e.g. open space) as well as planning for other infrastructure (e.g. water supply) so as to maximise the benefits of complementary measures and to ensure that conflicting outcomes are avoided. Under the principles of Water Sensitive Urban Design, stormwater planning should be integrated with water supply and wastewater planning as well as the management of ground waters.

The planning and design of relief drainage schemes and the retro-fitting of stormwater quality improvement systems should be based on current best management practice.

Stormwater planning within a local government can exist on three levels:

 (i)   An area wide Stormwater Management Strategy

(ii)   Catchment-based Stormwater Management Plans—including Urban Stormwater Quality Management Plans (USQMPs)

(iii) Site-based Stormwater Management Plans—including Site-based Stormwater Management Plans (SMPs)

2.2   Stormwater management strategy

To achieve coordination of the many disciplines and objectives, a local government should develop a Stormwater Management Strategy that covers its entire area and encompasses all stormwater-related activities in a manner that achieves the principal stormwater objectives. Even though the development of such a 'strategy' is not a legislative requirement, it does represent best practice.

A Stormwater Management Strategy may be used to:

- Assist in the development of catchment-based Urban Stormwater Quality Management Plans that appropriately reflect local issues and design standards.

- Guide councils in the planning, design and management of stormwater infrastructure.

- Guide the development industry in the design of water sensitive urban communities.

- Guide council in the operation of its general business activities in a manner consistent with its stormwater management objectives.

A Stormwater Management Strategy must integrate with a local government's other strategic plans such as the various Catchment Management Plans, Waterway Management Plans, Floodplain Management Plans, Open Space Plans, and Water Supply and Wastewater Strategies. The potential linkages between the Stormwater Management Strategy and associated management plans are shown in Figure 2.1 and Table 2.2.1. This figure is not exhaustive and does not include the links to things such as Open Space Plans and Water/Wastewater Strategies.
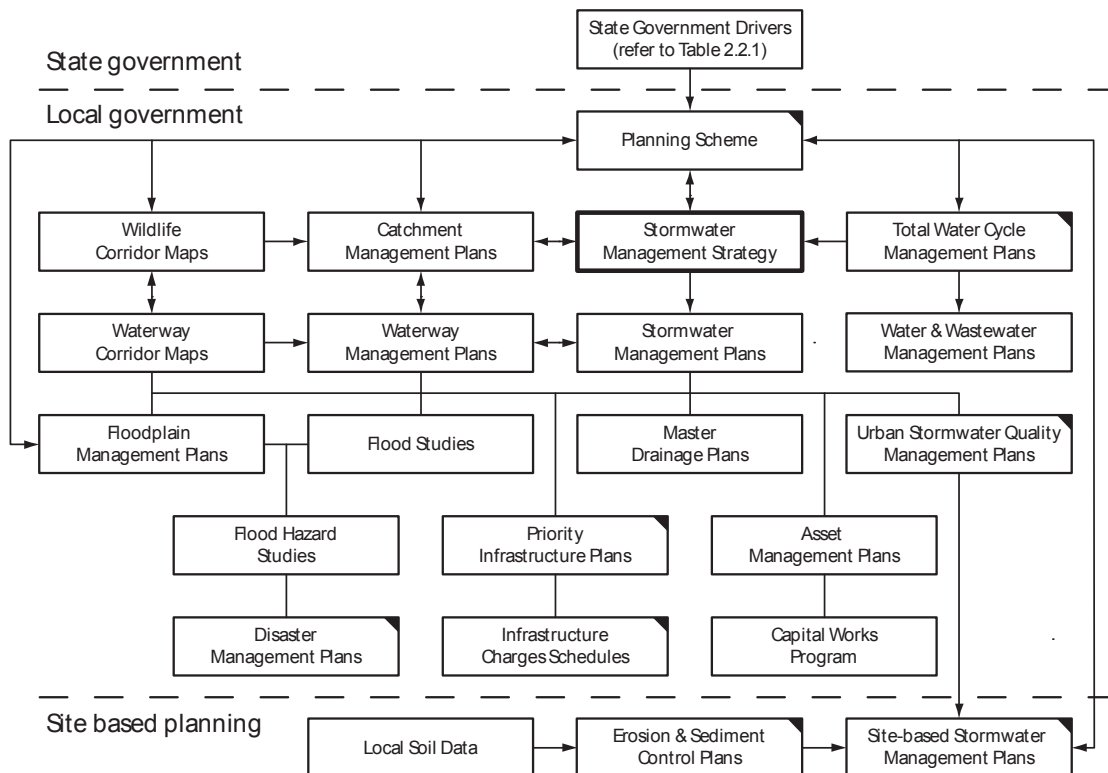
Figure 2.1 – Linkage between a Stormwater Strategy and various management plans
(Tagged boxes indicate plans required by specific legislation as of 2013)

The Stormwater Management Strategy should be consistent with the aims of the *Environmental Protection Act 1994* and the Environmental Protection (Water) Policy, and where practical should incorporate the following:

- catchment-based policies that reflect the local catchment resources, environmental and community values, development limitations and soil conditions
- policies applicable to the various land use, topography, soil, environment and economic conditions
- acknowledgment of the need to assess the cumulative impacts of pollutants, land use changes, and changes in stormwater runoff, rather than the impact of works in isolation
- encouragement of creativity and forward thinking
- policies equally applicable to all land users, including council works, developers, builders, the public and agricultural industry (where appropriate)
- policies that encourage cooperation and open communication between the community, land users and the various authorities
- policies that encourage cooperation and coordination between water supply, sewerage, groundwater and stormwater managers with respect to Total Water Cycle Management
- appropriate allocation of resources for implementation, maintenance, training and policing.

Table 2.2.1 – Brief outline of various plans

Area basis	Plan/study	Main output
Council wide	Planning Scheme	Development controls
	Total Water Cycle Management Plan	Coordination of all water service providers in the delivery of optimum cost and benefit outcomes
	Wildlife Corridor Maps	Identification and protection of significant wildlife corridors
	Stormwater Management Strategy	Local government approach to stormwater management
	Disaster Management Plan	Strategic coordination of local government and State Emergency Services
	Priority Infrastructure Plan	Strategic planning on the development of local government infrastructure
	Asset Management Plan	Strategic planning on the management of local government infrastructure assets
	Capital Works Program	Strategic planning on the financing of local government infrastructure
Catchment based	Catchment Management Plans	Environmental and social management of waterway catchments
	Waterway Corridor Maps	Identification of minimum floodway and riparian widths
	Waterway Management Plans	Management strategy for the protection of urban waterways, floodways and riparian areas
	Stormwater Management Plans (SMPs)	Management strategy for urban stormwater quality and flood control
	Urban Stormwater Quality Management Plans	Management strategy of the urban stormwater quality
	Floodplain Management Plans	Strategic planning and management of full floodplain, including flood risk and land use planning
	Flood Studies	Numerical modelling of extent and frequency of waterway flooding
	Flood Hazard Studies	Degree of flood hazard within a floodplain
	Infrastructure Charges Schedules	Strategic assessment of stormwater infrastructure charges
Local area study	Master Drainage Plans	Strategic management of sub-catchment flooding
Site based	Local soil data	Site specific soil testing
	Erosion and Sediment Control Plans (ESCPs)	Site specific erosion and sediment control strategy for a low-risk/small development
	Site-based Stormwater Management Plans	Site specific environmental management plan for a high-risk/large development

2.3 Stormwater management plans

Stormwater Management Plans (SMPs) set out how stormwater is to be managed within a catchment. These plans set out the proposed management of activities within a catchment which are likely to:

- alter stormwater runoff volume, velocity, rate, duration and frequency

  or

- adversely affect the environmental values of receiving waters either through physical modification or changes to runoff quantity or quality.

In effect, Stormwater Management Plans define the proposed management of stormwater quantity and quality, and the protection of receiving water features, such as the protection of existing waterways, lakes and wetlands. They also provide the basis for determining developer charges for trunk stormwater infrastructure.

Stormwater Management Plans may vary widely in their content depending on what studies or management plans already exist and the needs/interests of the target audience (e.g. community, local government officers, and state government departments).

Different state government departments will look for Stormwater Management Plans to address different aspects of stormwater management. Some of these aspects are legislative requirements and others are just good practice.

As a general guide, Stormwater Management Plans should include consideration of the following issues:

- protection from flooding
- acceptable health risk
- measures to reduce changes to the volume and velocity of stormwater runoff and changes to the natural flow regime of urban waterways (waterway stability, frequent flow management, catchment imperviousness)
- measures to maximise the infiltration of stormwater into the ground, thus providing long-term environmental flows to minor streams
- measures to minimise harm to receiving waters by stormwater
- opportunities to prevent the initial contamination of stormwater and to remove introduced contaminants
- opportunities for roadside pollution containment systems (i.e. the temporary trapping of pollutants from accident and traffic spills for later removal and treatment)
- community needs, including education and participation in the planning process
- aesthetics, public safety and other social concerns
- water conservation and recycling
- recreational, open space, landscape and ecological values of waterway corridors
- protection or rehabilitation of riparian vegetation along waterways
- rehabilitation of degraded drainage corridors
- integration of stormwater planning with catchment and land use planning
- consideration of alternatives to the release of stormwater across beaches or into poorly circulated waters
- any other issues relating to the objectives of stormwater management as outlined in sections 1.3 and 1.11.

When preparing a Stormwater Management Plan, each local government should consider the range of issues most relevant to the particular catchment and how best the SMP may address these issues. Table 2.3.1 sets out the broad areas of state government interest and the drivers for addressing these issues within a SMP.

Table 2.3.1 – Key aspects of SMPs for various state government departments

Government department	Key aspects of SMP	Drivers
Environment and Heritage Protection (EHP)	• Water quality • Environmental values • Waterway features (e.g. protection from impacts of physical modification or changes to runoff quantity and rehabilitation of natural water bodies)	*Environmental Protection Act 1994*
Natural Resources and Mines (DNRM)	• Water allocation • Riverine protection	*Water Act 2000*
Energy and Water Supply (DEWS)	• Water supply and/or sewerage planning and management • Dam safety	*Water Supply (Safety and Reliability) Act 2008*
Community Safety	• Water quantity (e.g. flood control and land use planning)	State Planning Policy 1/03
Agriculture, Fisheries and Forestry (DAFF)	• Fish passage (aquatic corridor management) • Waterway features (e.g. protection and rehabilitation of mangroves and fish habitats)	*Fisheries Act 1994*
Aboriginal and Torres Strait Islander and Multicultural Affairs	• Recognition, protection and conservation of Aboriginal cultural values within associated waterways	*Aboriginal Cultural Heritage Act 2003*
State Development and Infrastructure Planning (DSDIP)	• Priority Infrastructure Plans • Infrastructure Charges Schedules	*Sustainable Planning Act 2009*

The planning of urban drainage systems, flood management systems and stormwater treatment systems often use specialised numerical models, thus the development of a Stormwater Management Plan may incorporate the following modelling exercises:

• flood studies

• master drainage studies

• stormwater quality studies

• infrastructure studies

2.4　Flood studies and floodplain management plans

Flood studies primarily focus on the modelling and prediction of creek and river flooding.

Floodplain Management Plans are developed for the purpose of managing flood risk across the full width of the floodplain, not just the designated floodways.

Flood Studies may be used to provide the following information:

- master planning for waterway flood control
- design standards for stormwater detention/retention systems possibly varying within different regions of a given drainage catchment
- design standards for stormwater volume and peak discharge control possibly varying within different regions of a given drainage catchment
- design standards for the flood immunity of roadways and evacuation routes
- allowable planting densities for floodways and assessment of opportunities to rehabilitate riparian zones.

In addition to providing essential flood level information, flood studies should be integrated with Waterway Corridor Mapping and Floodplain Management Plans to develop an envelope of minimum floodway corridor widths and development controls.

A growing component of flood studies is the mapping of major overland flow paths. The importance of mapping overland flow paths was recognised in the final reports of the Queensland Floods Commission of Inquiry. Traditionally, the mapping of overland flow paths was seen as a component of 'Master Drainage Planning', but this role is now often incorporated into flood studies.

It needs to be recognised that the probable maximum flood (PMF) only identifies the extent of flooding within floodplains. Property flooding can still occur outside the PMF zone as a result of severe flows passing along overland flow paths. Such flooding is often termed 'stormwater flooding'.

2.5　Master drainage plans

Master Drainage Planning provides the basis for the provision of stormwater infrastructure to address traditional drainage, local flooding and safety issues; however, these plans may also address water quality issues.

Master Drainage Planning involves a detailed hydraulic analysis of the required stormwater drainage system having regard for the objectives of the Stormwater Management Strategy or Stormwater Management Plan.

Master Drainage Planning may be used to provide the following information:

- master planning for local flood control
- master planning for drainage control, including relief drainage
- master planning for stormwater detention and retention, including on-site detention standards
- master planning for aspects of stormwater quality.

Master Drainage Planning may be performed as a precursor to the development of a Stormwater Management Plan, or as a supplement to an existing Stormwater Management Plan.

2.6 Urban stormwater quality management plans

The national framework for the management of water quality, including stormwater management, is presented within the National Water Quality Management Strategy (a series of documents and guidelines).

Queensland's Environmental Protection (Water) Policy (2009) requires specific local governments to plan for the management of urban stormwater within a total water cycle management (TWCM) context. A local government's total water cycle management plan must include provisions about stormwater quality management to improve the quality and flow of stormwater in ways that protect the environmental values of affected waters.

The local government must consider including in these plans provisions about:

• identifying urban stormwater quality management needs for developed and developing areas that are consistent with the local government's priority infrastructure plan (refer to section 2.7)

• opportunities for stormwater harvesting, recycling or re-use

• adoption of Water Sensitive Urban Design principles

• managing urban stormwater quality and flows according to locally relevant documents

• monitoring and reporting processes for stormwater quality management.

The Australian and New Zealand Guidelines for Fresh and Marine Water Quality (ARMCANZ & ANZECC, 2000a) adopt three desirable levels of protection in respect to ecosystems:

• Pristine to slightly modified systems – requiring protection

• Slightly to moderately modified systems – requiring restoration

• Highly modified systems – requiring local identification of the values to be secured.

The Australian Guidelines for Urban Stormwater Management (ARMCANZ & ANZECC, 2000b) under the NWQMS indicate that *'the primary purpose of Stormwater Management Plans is to identify actions that will improve the environmental management of urban stormwater and protect environmental values of receiving waters'*.

The Urban Stormwater Quality Planning Guidelines 2010 (DERM, 2010b) provide advice on the preparation and required content of USQMPs. One of the first tasks should be to determine the required degree of complexity of the plan and any associated catchment modelling. This should be related to the complexity of the catchment and the assessed environmental risk.

In addition to the catchment-based USQMPs, site-based Stormwater Quality Management Plans may need to be developed for a particular development or land activity. The existence of a catchment-based plan does not negate the need for a site-based plan, but the site-based plan must achieve a level of protection no less than that established within the catchment-based plan. Site-Based Stormwater Management Plans are discussed further in Chapter 11.

2.7 Priority infrastructure plans

The *Sustainable Planning Act 2009* provides for local governments to levy infrastructure charges to fund the supply of development infrastructure items. Development infrastructure items are limited to land and capital works for: urban water cycle management infrastructure (water, sewerage, stream management, disposing of water and flood mitigation); circulation networks (roads, dedicated public transport corridors, public parking, cycle ways, pathways); public recreation infrastructure, and land for local community purposes.

The priority infrastructure plan is an important strategic planning tool that aims to align the local government's ability to service with infrastructure, the areas identified for future urban growth in the planning scheme. It is also the core element of the infrastructure charging framework in the *Sustainable Planning Act 2009*. It provides a clear, transparent and certain basis for the calculation of infrastructure charges.

The assumptions underpinning each plan are critical elements of the priority infrastructure plan. Their purpose is to provide a logical and consistent basis for the detailed infrastructure planning in the plan. Together with the desired standards of service they assist in the development of the plans for trunk infrastructure, which provide a detailed infrastructure planning benchmark for the calculation of infrastructure charges and upon which additional infrastructure cost assessments may be based.

Priority infrastructure plans for stormwater infrastructure are a requirement under the Act where it is intended to levy infrastructure charges for trunk elements of the system, (i.e. system elements serving more than one development or new and existing development) such as:

- major drainage and flood mitigation elements (e.g. regional detention basins, stream hydraulic improvements, levees, culverts)
- regional water quality improvement infrastructure (e.g. wetlands, in-stream GPTs, stream rehabilitation).

2.8 Infrastructure charges schedules

Before an infrastructure charge is set the item must be identified in an 'infrastructure charges schedule' which is part of the local government's 'priority infrastructure plan'.

The infrastructure charges schedule:

- provides a transparent account of the cost of the trunk infrastructure being charged for
- indicates when new trunk infrastructure is likely to be provided
- quantifies existing and expected new users
- shows how costs are to be apportioned to those users
- states the charges various users will be required to pay.

An infrastructure charges schedule must state either or both of the following:

- Timing—the estimated time (year) that the trunk infrastructure forming part of the network will be provided.
- Thresholds—the thresholds for providing the trunk infrastructure forming part of the network (e.g. when a demand level is reached it triggers the provision of certain trunk infrastructure).

2.9   Associated mapping and planning schemes

The preparation of the following planning tools can greatly assist local governments in the development of Stormwater Management Plans.

2.9.1   Soil maps

Regional soil maps may be used for a variety of purposes including:

- To assist local governments in the preparation of Stormwater Quality Management Plans.

- To assist local governments to prepare a list of preferred stormwater management systems for different soil regions. Such a listing may assist local government officers in the review of development applications. For example:

  o constructed urban lakes may not be desirable within regions of highly dispersive soils

  o a local government may prefer the use of swales only in regions of soils of a specified minimum porosity.

- To assist in the development of Erosion Risk Maps that help to identify those development areas that require a higher erosion and sediment control standard during the construction and building phases, or those regions where the natural waterways are likely to be more susceptible to channel erosion following urbanisation.

- Assist in the development of site-based Erosion and Sediment Control Plans (ESCPs).

Soil properties of greatest interest to stormwater designers are the erosion potential (slope, texture, dispersion index) and the soil's infiltration capacity.

Erosion Risk Mapping can be used to assign the erosion risk or development potential of a region. It is important that the ranking system clearly identifies outcomes that produce **actual** variations in stormwater management practices within different areas of erosion risk; otherwise the mapping exercise provides little value.

The Urban Stormwater Quality Planning Guideline (DERM, 2010b) and Best Practice Erosion & Sediment Control (IECA 2008) provide erosion hazard assessment templates that can incorporate erosion hazard mapping into planning schemes to target increased planning requirements in higher risk areas.

2.9.2   Wildlife corridor maps

Wildlife Corridor Maps identify essential terrestrial and aquatic movement corridors that link habitat and breeding areas, specifically the terrestrial linkage of bushland reserves. The importance of these maps to the development of a Stormwater Management Strategy is in relation to the required cohabitation of stormwater issues and wildlife requirements within floodplains. Waterway corridors often act as essential wildlife corridors within urbanised catchments. The development of a Wildlife Corridor Map is often an essential precursor to the development of a Waterway Corridor Map.

2.9.3   Waterway corridor maps

Waterway Corridor Maps identify:

- those waterways that are required for aquatic habitat and fish passage (fish passage mapping has been conducted by Queensland Fisheries)

- those waterways that act as terrestrial wildlife corridors

- minimum waterway corridor widths (e.g. 30, 60 and 120 metre minimum corridor width)

- minimum desirable overbank riparian vegetation widths

- any Ramsar listed wetlands linked to waterways—the Ramsar Convention on Wetlands of 1971 was held in the Iranian town of Ramsar which resulted in a United Nations treaty enacted in 1975.

Ideally, Waterway Corridor Maps should also identify and rank (in order of potential impact) existing or potential fish passage barriers.

2.9.4 Catchment management plans

Catchment Management Plans may address a wider range of issues, possibly including:

- land use needs e.g. recreational and open space requirements possibly linking to Open Space Master Plans
- community needs e.g. community education on catchment and waterway related issues
- flora and fauna needs, including catchment and inter-catchment movement corridors
- threats to sustainable land use and/or conservation needs such as weed control.

2.9.5 Asset management plans

All stormwater infrastructure requires ongoing maintenance to ensure its performance. Traditionally, ensuring that adequate maintenance occurs has been somewhat problematic. This is typically because stormwater infrastructure is only required to perform its function intermittently or infrequently; however, timely maintenance must be given a high priority if the objectives of stormwater management are to be met.

Preposed new infrastructure should be considered on both its ability to meet design objectives and its whole of life operation and maintenance needs.

3. Legal aspects

This chapter contains general information on the main legal issues relevant to stormwater and drainage projects, including information on key legislation, tenure and approvals which may be relevant to such projects. It does not purport to provide specific legal advice, and should be used as a general reference guide only.

Independent legal advice based on the specific circumstances in each case should be sought in relation to stormwater and drainage projects, and stormwater and drainage management generally.

3.1 Where legal issues might arise

Urban development generally modifies the naturally occurring drainage regime, thus potentially altering the volume, rate, frequency, duration and velocity of stormwater runoff, as well as the water quality.

Urban drainage works may also divert flows between natural catchments, modify existing flow paths, and/or concentrate flow along drainage paths and at outlets. These changes may affect the natural and built environment, safety, and enjoyment of persons and property, possibly resulting in legal disputes.

Legal disputes arising from the planning and construction of stormwater and drainage works may be avoided by negotiating with the people potentially affected by the works prior to seeking approvals or commencing the works. This includes, for example, liaising and negotiating with landowners of affected properties and the local authority, which generally has jurisdiction over stormwater and drainage management in its local government area.

The risk of legal disputes may also be minimised by undertaking a due diligence assessment of the location and nature of the proposed works, and the legal requirements applicable to them. Further information on due diligence assessment is set out in section 3.2.

Legal issues relating to stormwater and drainage projects arise from both state law and common law. State laws provide specific legal requirements for stormwater and drainage works, usually under planning laws, and under water management laws. In addition to requirements under State laws, there may be applicable legal or policy requirements under the common law, local government planning schemes, local laws and/or stormwater drainage manuals/codes.

Legal issues may arise in the context of many stormwater management actions, including the following examples.

(a) Diversion of stormwater

Often it may be considered necessary to divert runoff from a sub-catchment to a different point of discharge than that occurring naturally. This, however, should not be contemplated without consideration of the possible legal issues associated with the increase in discharge at the new outfall.

In addition, when stormwater runoff discharges from the 'new' drainage system into the 'existing' downstream drainage system, the outlet works may need to play an important role in dissipating energy, preventing scour, limiting siltation and possibly controlling water quality.

The outlet structure may include a headwall, wingwalls, apron, energy dissipater, pollution trap, a transition section of lined or unlined open channel, and a low-flow pipe or channel.

Legal issues arising in this example may include new approval or tenure requirements arising out of a change in location of discharge, approvals/permits associated with the construction of the outlet works, and the potential breach of environmental laws (such as a breach of approval conditions).

(b) Concentration of stormwater flows

Where surface flow (as distinct from water flowing in a natural watercourse) is diverted or collected either by open channel or conduit resulting in an increase in the flow at a particular point, the flow may be said to be concentrated at that location.

The concentration of flows is dependent on many factors. For example the construction of buildings and paved areas can increase the volume and rate of runoff, the construction of property fencing (including noise control barriers) can change the spread or location of flows, and the discharge from stormwater pipes can concentrate flows discharging from one property to the next.

Legal issues arising in this example may include the risk of property damage where concentrated flows are not dissipated by the time the flow reaches an allotment or development boundary.

(c) Changes in stormwater flows and water quality, sedimentation, etc.

Adverse impacts to downstream properties may also be the result of changes in peak discharge and/or changes in the frequency, duration, velocity, volume or quality of regular flows.

These changes are more likely to be an issue for a proposed project where the downstream property already experiences one or more of the following conditions:

- For peak discharge – the property currently experiences inundation of buildings, including non-habitable buildings and floors, as a result of local runoff.

- For frequency and duration of stormwater discharge – the property currently experiences problems associated with waterlogged soils. Depending on the sub-surface drainage characteristics of the local geology, the construction of stormwater infiltration systems within an upstream property could either increase or decrease soil waterlogging within adjacent properties.

- For velocity of inflows – the property currently experiences soil scour problems. Changes to the velocity of inflows often depend on differences between the existing boundary fence and the proposed boundary fence. Such issues can be made more complex by the construction of sound-control barriers between adjoining properties.

- For the volume of stormwater runoff – the property currently experiences flooding problems that result from the pooling of stormwater on their property.

- For water quality – the downstream property has a water supply dam. Temporary water quality issues may also arise from where sedimentation problems during the construction phase of the upstream development.

Where, the impacts of stormwater or drainage works or associated development are such that the downstream landowner suffers a loss of enjoyment of their property, or suffers actual physical damage to their property, these impacts may be considered to be 'nuisance' in the legal sense.

Legal issues arising may include the risk of an actionable nuisance claim by an affected landowner. Common law principles may entitle the affected landowner to claim compensation for

such impacts. A discussion on the common law principles of nuisance and its relevance to the management of stormwater and surface water drainage is provided below.

It must be emphasised that compensation paid to a property owner in respect of land or easement acquisition is merely payment for the right to acquire or use that land for the relevant purpose that instigated the acquisition (e.g. stormwater discharge / drainage). Such payment is not intended to 'compensate' for any potential nuisance which may be caused to the property owner as a result of the discharge. A discussion on easements and land acquisition is provided in sections 3.8 to 3.10.

3.2   Nuisance at common law

Drainage disputes are generally a matter of common law, as modified by legislation. The issues are often unclear and legal opinion is often necessary to determine an appropriate course of action.

In carrying out works or actions that modify existing stormwater and surface water drainage patterns the rights of adjoining landowners at common law must be taken into account. A person (including a local government) may be liable under common law principles of nuisance where there has been an unlawful or unreasonable interference with a person's use or enjoyment of land, or of some rights over or in connection with the land.

If a person's actions, for example, result in the concentration of additional surface water over and above flows that would occur naturally which cause a direct impact to another person's land, liability for nuisance may arise.

The impact may be in the form of actual physical damage to land or impairment of the owner's ability to enjoy their land.

The leading case on nuisance arising from stormwater and surface water drainage is *Gartner v Kidman* (1962) 108 CLR 12. It set out the following principles:

(i)   The person from whose land the water flows (upstream owner) is not liable merely because surface water flows naturally from that person's land onto another person's land (downstream owner).

(ii)   The upstream owner may be liable if water flows from the upstream owner's land in a more concentrated form than it naturally would due to man-made alteration of the level or conformation of land.

(iii)   The upstream owner will not be liable for a more concentrated flow caused by the works of a third-party over which the upstream owner has no control (e.g. works separately carried out by a local government).

(iv)   A nuisance will not arise where the damage is caused by the upstream owner's natural and ordinary use of the land (e.g. where the upstream owner has not carried out works to change natural drainage patterns).

(v)   The downstream owner can put in place measures to prevent the natural unconcentrated flow of water on their land, even where doing so damages the upstream owner's land, as long as the downstream owner uses reasonable care and skill in implementing such measures and does no more than is reasonably necessary to protect the enjoyment of their land.

(vi)   In putting in place measures to prevent the natural unconcentrated flow of water on their land, the downstream owner cannot divert the water onto the land of a third landowner to which it would not have naturally flowed.

The remedies available to the downstream owner in circumstances where a nuisance has occurred are damages for loss or damage caused by the nuisance or an injunction against the upstream owner.

The trial judge in the case of *Alamdo Holdings Pty Ltd v Bankstown City Council* (2003) 134 LGERA 114, found that even a significant increase in the frequency with which land will be inundated can constitute a significant interference with the use and enjoyment of land and hence give rise to an actionable nuisance. On appeal, the High Court of Australia found that s773 of the *Local Government Act 1993* (NSW) indemnified the defendant Council against liability in respect of such a nuisance (see *Bankstown City Council v Alamdo Holding Pty Ltd* [2005] HCA 46 (although this was in the context of the specific NSW legislation).

Where a local government has commissioned works giving rise to a nuisance, the local government may be liable for that nuisance. There is also a possibility that a local government will be liable along with an upstream owner in circumstances where it has issued a development approval to the upstream owner allowing the works which cause the nuisance to be carried out.

An upstream owner will not be liable for a nuisance where the downstream owner has consented to the discharge of stormwater or drainage onto their land. For that reason, conditions imposed on a development approval by a local government may require an adjoining landowner's consent to the receipt of additional stormwater and surface water drainage generated by the works covered by the development approval. It should be noted however, that such arrangements do not attach to the land and are generally not binding on subsequent owners of the land.

This chapter does not contain an exhaustive commentary on the potential common law liabilities which can arise in the context of drainage and stormwater works. Although nuisance is the most frequent common law principle invoked in relation to such works and their consequences, other common law principles can also apply in some circumstances. For example, common law principles of negligence will require a proponent of works to ensure that they satisfy their common law duty of care in terms of designing and constructing works with adequate care and skill.

3.3   Due diligence assessment

A due diligence assessment is the management function for ensuring that a proposed development complies with the relevant laws. It is a specific element of *risk management* and an essential step for minimising the risk of enforcement action or any associated economic loss arising from non-compliance. A due diligence assessment can assist in identifying circumstances in which the legal issues listed above (or other legal issues) may arise.

In the context of stormwater and drainage projects, a due diligence assessment will include at least the following elements:

- Defining the nature and extent of all proposed works, including the intended location of works and points of stormwater discharge.

- Identifying the relevant land and issues affecting it. This includes determining the owners and occupiers, and the unique property and planning considerations, of all land on which:
    - proposed works will take place; and
    - any land (including upstream and downstream sites) which may be impacted during the works or as a result of the works once completed;

   (see section 3.6).

- Identifying access rights and tenure needed for the project. This includes determining if:

- o any access rights and tenure may be required, both for the construction phase, and ongoing operation of the proposed works; and then

- o identifying the actions needed to obtain those rights and tenure of an appropriate nature. Appropriate tenure may range from a simple easement up to the acquisition of freehold title over a site;

(see sections 3.7 to 3.10).

- Identifying planning and other approval requirements. This includes licences, permits or other statutory approvals required for the construction and ongoing operation of the proposed works, and the actions needed to obtain such approvals (see sections 3.5 and 3.11).

- Considering other legal requirements, including *statutory duties of care* and *common law requirements,* and any risk management strategies or actions needed to comply with them (see section 3.2).

3.4 Lawful point of discharge

A 'lawful point of discharge' test has been used by the industry to assess whether discharge at a particular location is lawful. The test is focused on whether all applicable regulatory and other legal requirements have been met or consent has been obtained to allow stormwater to discharge in a particular location.

The term lawful point of discharge has no prescribed legal meaning, but it is commonly used by the industry to refer to a point where the stormwater flow discharge is 'lawful'.

Ensuring a lawful point of discharge exists will be necessary if stormwater discharge has changed since the development was initially designed. This may not be the case where the changes to discharge flows are not significant and happen in the natural and ordinary use of the upstream property.

Where there is doubt as to whether a lawful point of discharge exists it is recommended that it be sought (through the undertaking of a due diligence assessment and by obtaining the required consents, approvals and/or tenure) in order to minimise the risk of legal disputes arising with affected landowners or the local government.

3.4.1 Lawful point of discharge test

When proposing a development, the developer may need to demonstrate to the local government that a lawful point of discharge exists.

The two-point test may be helpful in assessing whether a lawful point of discharge exists at a particular location. The test consists of being satisfied that:

(i) The location of the discharge is under the lawful control of the local government or other statutory authority from whom permission to discharge has been received. This will include park, drainage or road reserve, stormwater drainage easement.

(ii) In discharging to that location, the discharge will not cause an actionable nuisance (i.e. a nuisance for which the current or some future neighbouring proprietor may bring an action or claim for damages arising out of the nuisance), or environmental or property damage.

Where the conditions in (i) are not in place prior to the development being proposed, it will be necessary to seek a lawful point of discharge. This will usually be achieved by the acquisition of stormwater drainage easements or drainage reserves over one or more downstream properties. It will normally be necessary for a large part of the design to have been completed prior to

determining the extent of any necessary easements. Refer to sections 3.7 to 3.10 for information on drainage reserves and acquiring easement rights.

Note: A watercourse may not necessarily constitute a lawful point of discharge, unless the requirements of the above two tests and other applicable legal requirements have been satisfied.

In addition, the proposed stormwater discharge cannot cause nuisance as outlined in (ii) above.

3.5   Discharge approval

In lieu of drainage easements or another form of tenure allowing stormwater discharge into freehold properties downstream of the development, some local governments may be prepared to accept a letter from the downstream owner to the developer granting 'discharge approval'.

In this letter, the downstream owner usually agrees to accept the discharge from the upstream property provided that the works proposed by the developer are constructed in accordance with drawings approved by the local government.

The 'discharge approval' is a form of contract between the developer and the owner of the downstream property under which some consideration will usually be paid for the right to discharge. This situation essentially relies on the goodwill of the parties. 'Discharge approval' may be revoked by the downstream owner, unless there is a binding contract, or a grant of easement.

A subsequent purchaser will not be bound by the previous owner's contract unless the subsequent owner agrees to be so bound, either as a condition of the contract of purchase or by the executing of an appropriate agreement. It would then be left to the aggrieved party to resolve the situation by, for example, attempting to enforce the contract, if one existed. The local government would have no ability to enforce any such contract if it was not a party to it. This approval is therefore not ideal in situations where it is important that the discharge arrangement is in place and enforceable on a long-term basis.

3.6   Tenure for proposed drainage works

The project proponent should identify details of land, including current ownership and tenure, on which the proposed development will take place, or which may be affected by the proposed works. This will assist in assessing what form of interest in the affected land the developer will need to seek (if any) for the proposed works, and approval requirements that may apply on that land.

The appropriate form of tenure may vary depending on whether the land on which the proposed drainage works will take place is freehold or non-freehold land, and on the nature of the works and any permanent infrastructure or impacts resulting from the works.

Freehold land is regulated under the *Land Title Act 1994* (Land Title Act), while non-freehold land is regulated under the *Land Act 1994* (Land Act). Non-freehold land includes unallocated State land, State land subject to reserves, State land leases (including pastoral leases), most foreshores and most waterways.

3.6.1   Non-freehold land

There are various types of interests in non-freehold land that can be granted (although not all can be granted in all circumstances) —usually by the State of Queensland through the Department of Natural Resources & Mines. Those interests include the following:

- dedication of reserves (including new reserves or alteration to the boundaries of existing reserves)
- perpetual and term leases

- permits to occupy unallocated State land or reserves

and

- public utility easements.

In most cases, the form of interest that the proponent is likely to seek over non-freehold land will be the dedication of a new drainage reserve or an easement for drainage purposes (see section 3.7).

3.6.2   Freehold land

Where freehold land is not owned by the project proponent or the local government responsible for the stormwater and drainage in the area, the land may need to be purchased or compulsorily acquired for the purposes of the project, depending upon whether:

- the works involve any permanent infrastructure or there is another reason why the tenure should be as long-term and secure as possible

and

- whether the existing landowner's interests will be significantly affected. Early negotiation with landowners and local authorities is always recommended.

Easements for drainage purposes are also a common type of interest that may be sought for drainage works proposed on freehold land (see section 3.8).

3.7   Drainage reserves

Drainage reserves are dedicated under the Land Act by the responsible Minister on behalf of the State of Queensland through the registering of a dedication notice or plan of subdivision for the reserve. The dedication process will involve notifying and seeking submissions from all persons with a registered interest in the unallocated State land over which the reserve is proposed. Once dedicated, the reserve is put in the care of trustees who manage the land on behalf of the State.

Reserves can only be dedicated over unallocated State land. This means that other existing interests in the State land may first need to be removed.

New reserves can only be dedicated for one or more of the community purposes listed in Schedule 1 of the Land Act. Those purposes include *drainage*. The dedication of a new reserve for drainage purposes may be appropriate where a large area of State land is required for flood mitigation or the like.

As an alternative to a drainage reserve, the Minister may grant in fee simple in trust unallocated State land for use for a community purpose (such as drainage).

More frequently, a public utility easement for drainage purposes will be sought over non-freehold land. Public utility easements may be registered over State land or freehold land.

3.8   Drainage easements

3.8.1   Easements generally

Easements are property interests to which land may be subject, in order to benefit adjacent land in a particular way. The most common example is where a landowner grants an easement over his or her lot to enable landowners of adjoining lots to pass over the land to access a road or another lot.

The exceptions to this rule are 'easements in gross' or 'public utility easements', which do not have a benefited lot. These easements are granted to public utility providers, including a local

government, for certain specific purposes. Examples of public utility easements include easements for drainage purposes and for the supply of electricity, water and gas.

In Queensland, an easement is typically defined by two documents: the survey plan which shows the location and dimensions of the easement, and the easement document which sets out the rights granted by the grantor to the grantee and the conditions under which those rights may be exercised.

3.8.2   Need for easements in stormwater and drainage projects

In an urban development context, easements may be required to permit access for stormwater and drainage works to be performed, or to secure a right for stormwater flow to be directed and discharged over or to properties in the vicinity of the works.

A drainage easement in favour of the local government is often required where the stormwater and drainage infrastructure (e.g. drains, whether open or underground) is located within property not under the control of the local government.

The drainage easement, once granted, will enable the local government to access land to carry out drainage works on the stormwater and drainage infrastructure located within the easement (or to install new infrastructure within the easement), in accordance with the easement's terms. Where a landowner does not agree to the grant of an easement in favour of the local government, the land may need to be purchased or compulsorily acquired.

An easement may also be required in relation to private development for the benefit of the proponent over an adjoining owner's land. The easement in these circumstances is generally agreed between the proponent and the adjoining landowner. Where agreement cannot be reached, the proponent may negotiate the purchase of the land or request that the local government exercise its compulsory acquisition powers (see section 3.9).

Although overland flow should not normally be directed through private freehold property, where this situation cannot be avoided and is acceptable to the local government, an easement to use that freehold property for drainage may be required.

Where a drainage easement is necessary in relation to a private development, the local government may make the creation of the easement a condition of the development approval for the development.

A general rule is that a landowner should not alter stormwater flows that modify discharge patterns from his or her property to a person's downstream property without that person's agreement or without a drainage easement for the discharge. However, the exception is when stormwater flows are altered in the natural and ordinary use of the land. Refer to the discussion on 'lawful point of discharge' in section 3.4.

3.8.3   Drainage easements generally

Easements for drainage purposes may be required over freehold or non-freehold land. As such, both the Land Title Act (for freehold land) and the Land Act (for non-freehold land) are relevant.

In relation to freehold land, an easement under the Land Title Act may be either of the following:

a) A standard easement

   Such an easement can be for a wide range of purposes. These easements require that there be both a benefited lot (i.e. land benefited by the easement) and a burdened lot (i.e. land burdened by the easement).

Such an easement may be granted by an adjoining landowner (owner of the servient property) for the benefit of the developer (owner of the dominant property) who intends to carry out works on infrastructure located on the adjoining land.

The easement must be registered on the freehold land register.

b) A public utility easement

Such an easement does not need to specify a benefited lot, as it is granted in favour of a public utility provider and for certain specific purposes only (including drainage). A public utility provider includes a local government.

Such an easement may be granted where, for example, the local government needs to carry out works on stormwater and drainage infrastructure located within freehold property not under its ownership.

A public utilities easement granted over freehold land must also be registered on the freehold land register.

Public utility easements may also be sought over land granted in trust or non-freehold land under the Land Act subject to consent by the Minister for Natural Resources and Mines as the landowner. This includes a lease of non-freehold land, unallocated State land and dedicated reserves. The easement may be granted for drainage purposes and is registered on the relevant State land register.

3.8.4 Creation or acquisition of easements and existing easements

Creating or acquiring an easement involves either of the following approaches:

a) **Voluntary acquisition** by agreement / private treaty between the landowner and the proponent or the local government. This approach involves the voluntary grant of an easement under the Land Act or the Land Titles Act.

b) **Involuntary acquisition**

i. By way of compulsory acquisition of the land itself or an easement over the land. This approach may be required if the owner is not prepared to agree, or if reasonable terms of agreement cannot be negotiated.

or

ii. By court order under the *Property Law Act 1974* (PLA) for a statutory right of user order.

In all cases, the easement must be properly registered on the appropriate land register. Many survey plans show easements that may not necessarily be legally recognised or binding on the burdened landowner and its successors, because the easement document has not been properly registered. Details of a registered easement will appear on the survey plan for the burdened lot, and on a title search for that lot. Where a title deed still exists for a lot, a notation may appear on the deed.

Where a developer seeks to use an existing easement which is part of the local government's stormwater system, the local government's consent will be required under the *Local Government Act 2009*.

The State Government has standard forms, which may assist in the preparation of an easement document.

The two approaches for acquiring an easement—voluntary acquisition or involuntary acquisition— are discussed in section 3.9.

Section 3.10 sets out the recommended steps to be followed by a private developer seeking a drainage easement or drainage reserve over a downstream property.

3.8.5 Drainage easement dimensions

Easements need to be of such width, length and location to enable necessary works (e.g. construction, maintenance and site inspection) to be carried out. Easement widths should be not less than the greater of the following:

- 3 m for all single pipes from 300 mm up to 1350 mm diameter (in new developments) or as otherwise determined by the local government.

- 1 m wider than the distance between outer edges of the pipes or box culverts (in new developments) or as determined by the local government.

- Width of flow path required to carry the difference between the peak discharge for the *Defined Flood Event* (refer to section 7.2.3) and the capacity of the underground system together with an allowance for freeboard as outlined in section 7.3.12.

- For the purposes of this sub-section the capacity of the underground system may be taken as being its capacity when carrying the discharge from the minor design storm, with provision for blockage of grates as detailed in section 7.5. The exception may be where the system is located in extremely flat ground or near an outlet that becomes fully submerged under major storm conditions, a detailed check shall be undertaken to ensure that the minor system does not have a lesser capacity under these conditions.

- Easements for open channels shall, unless agreed otherwise with the local government, be of sufficient width to provide an access track along at least one side of the channel for operation of maintenance vehicles. Refer to sections 9.2 and 9.7.2 for recommendations on minimum access and maintenance widths. The width of this access track should also take into account the type (width) of equipment required to perform the channel maintenance. An allowance must be made for freeboard as outlined in section 9.3.4.

3.9 Acquiring easement rights

3.9.1 Voluntary acquisition by private treaty

Acquisition by private treaty is the normal method by which a person deals privately with another for the **purchase of land** or **easement rights**. It is normally undertaken by a direct approach to the proprietor of the property and will usually require some consideration (usually monetary compensation).

In relation to the purchase of easement rights, first the parties will reach agreement on price so that the easement can be granted either under the provisions of the Land Act or Land Title Act and subsequently registered on the title for the burdened and benefitted lot.

There is no legal impediment to this approach save that any easement rights which are to be granted in favour of the local government must be granted in terms generally acceptable to the local government.

Most local government authorities have standard easement documents. A valuation assessment prepared by a Registered Valuer is usually the basis for commencing negotiations on price.

The local government may choose to enter negotiations for the purchase of the property itself, instead of the easement rights. In addition, the *Acquisition of Land Act 1967* (Acquisition of Land Act) includes provisions for the local government to take land by agreement.

3.9.2 Compulsory acquisition by a local government

Compulsory acquisition may be necessary where the owner of the affected freehold property does not agree to the granting of an easement in favour of the local government.

The Acquisition of Land Act provides a local government with certain compulsory acquisition powers in relation to freehold land that may be used for stormwater and drainage projects. A local government can compulsorily acquire an interest in freehold land including acquiring the whole estate or acquiring an easement over the property, when it is not necessary to acquire the whole estate.

Compulsory acquisition powers are not available to private persons, and can only be exercised for purposes set out in the schedule to the Acquisition of Land Act, or for purposes authorised or required by other legislation. The relevant purposes in the schedule include *"drainage"*, *"flood gates or flood warnings"*, "*flood prevention or flood mitigation"* and "*works for the protection of the seashore and land adjoining the seashore"*.

There are statutory processes which must be strictly followed by the local government (as a constructing authority) in order to successfully complete a compulsory acquisition. These processes involve affording affected landowners the right to serve on the constructing authority a written objection to the acquisition and if desired, to be heard in support of the grounds of the objection at an objection hearing.

The land being acquired vests in the constructing authority on and from the date of the publication of a gazette resumption notice. The estate and interest of every person entitled to the whole or any part of the land (or whose estate and interest in the land is injuriously affected by the easement) is converted into a right to claim compensation under the Acquisition of Land Act.

In addition to the provisions in the Acquisition of Land Act, section 714 of the *Sustainable Planning Act 2009* states that a local government may take or purchase land if the local government is satisfied the taking of land would help to achieve the strategic outcomes stated in the planning scheme; or, at any time after a development approval or compliance permit has taken effect, the local government is satisfied that certain conditions have been met, such as that there is a need to carry drainage over the land and reasonable measures were taken to obtain the agreement from the landowner, but that agreement was not able to be obtained and the action is necessary to allow the development to proceed.

There are also special provisions in the Land Act for the resumption of certain interests in non-freehold land (including the taking of easements over non-freehold land). Such resumptions are generally undertaken by the State on behalf of a constructing authority (such as a local government). The constructing authority must meet the costs of the acquisition.

Native title over non-freehold land can also be compulsorily acquired; however, for most stormwater and drainage projects there will usually be more straightforward options for addressing native title.

Under section 577 of the *Water Act, 2000* (Water Act), a water authority has power to take any land to which the Acquisition of Land Act or the Land Act apply for drainage purposes.

Different local governments have different requirements for when they are prepared to approve engineering design prior to the grant of any easement. In most cases, this will generally not occur before lodgement of the survey plan and easement document with the department administering the Land Title Act. A local government may accept dealing numbers (receipts) as proof of lodgement.

3.9.3 Acquisition under the Property Law Act

A property owner, in order to effectively use their land, may apply to the court under the *Property Law Act 1974* for a right to use land not under the person's ownership.

Such statutory rights of use may take the form of an easement, a licence or some other form and can be subject to conditions, including conditions as to the length of time the right may remain in existence. An order will only be made if the court is satisfied that it is consistent with the public interest for the property to be so used and that the owner of the affected land will be adequately compensated.

These orders are largely discretionary orders a court can make and the court has wide powers to make ancillary orders such as orders relating to the preparation of a plan of survey, the execution of any documents necessary for registration and directions for the conduct of proceedings generally.

3.10 Process for private developers seeking a drainage easement or drainage reserve over downstream property

The following is a suggested process for private developers to acquire an easement or drainage reserve in favour of a local government over a downstream property:

a) Advise the local government of the purpose of and the reasons for the acquisition and seek the agreement in principle from the local government to the proposed means of obtaining the lawful point of discharge.

b) Request a copy of the standard easement document used by the local government.

c) Once the 'in principle' agreement has been obtained from the local government, write to the downstream owner outlining the need for the easement and seek to meet with the owner to set out the proposal in detail including any proposed compensation.

   Note: It is obviously in the developer's interest to obtain the easement for the least possible cost. It is usual in such cases for the developer to pay all costs of the downstream owner associated with the transaction such as legal fees, valuation fees and mortgage release fee in addition to his or her own costs such as survey, legal fees, titles office fees, local government fees, valuation fees and compensation.

   The downstream owner may require compensation either in monetary terms or 'in kind'. The usual basis for monetary compensation will be a valuation prepared by a Registered Valuer. The downstream owner may negotiate with the developer on their own behalf or obtain a valuation (normally at the developer's cost). Negotiations usually proceed from that point. 'In kind' compensation usually involves some construction works in lieu of monetary compensation, but the valuation of compensation would still be on the same basis. Either method should be acceptable to the developer and the local government. Where 'in kind' compensation is accepted, it is wise to ensure that such works are acceptable by the local government.

d) Once the basis for acquisition has been agreed, the survey plan and easement document should be prepared. The survey plan and document must be approved by the local government.

   Note: Some local governments require all easements in their favour to be prepared by surveyors and/or solicitors of their choice at the developer's cost. Difficulties in relation to time can sometimes result from this requirement and the developer must pay attention to this aspect to avoid unreasonable delays.

e) Once the downstream owner has signed the survey plan and easement document these should be lodged with the local government for signature and possibly sealing.

Note: A sealing fee may be applicable.

f) The survey plan and easement document should then be lodged in the Department of Natural Resources & Mines for registration. Apart from a number of special situations, neither the plan nor the document may be removed from the Department of Natural Resources & Mines during the registration process if they have been sealed by the local government.

As mentioned in section 3.6.2 a local government may be prepared to approve engineering plans prior to the granting of necessary easements, but this will not generally occur before lodgement of the survey plan and easement document with the relevant department. Dealing Numbers (receipts) are usually acceptable proof of lodgement.

3.11   Statutory approvals and other requirements

In addition to obtaining the appropriate tenure for a stormwater or drainage project, a number of approvals under legislation may be required for the project to be carried out, depending on the project's location and nature.

Various aspects of stormwater and drainage projects constitute 'development' under the *Sustainable Planning Act 2009* (SPA). 'Development' includes building works, plumbing and drainage works and various types of operational works including excavating and filling and vegetation clearing.

A development approval under SPA is required where an aspect of the project is considered to be assessable development under a local government planning scheme or under Schedule 3 of the SPA. The development may be subject to assessment by a local government or State government department or another authority (e.g. private certifier for building works). To determine whether or not the works will constitute assessable development and require an approval, a consideration of the nature and location of the specific works should be conducted. Further details of SPA's requirements are in section 3.12.5.

SPA is not the only legislation which prescribes approvals necessary for aspects of stormwater and drainage projects. Table 3.11.1 contains examples of key statutory approvals that may also be required. Further details of these approvals are contained in section 3.12.

In addition, other legal requirements may also be relevant to these projects. These can include common law principles, statutory duties of care and prohibited conduct through the creation of offences (see section 3.2).

Failure to obtain relevant statutory approvals or to comply with other legal requirements may be punished by severe penalties under the relevant legislation.

Table 3.11.1 – Example of possible statutory approvals

Legislation	Activity	Statutory approval
SPA / local government planning schemes	Plumbing and drainage works	Development approval
SPA / Building Act	Building works	Development approval
SPA / Vegetation Management Act	Operational works for clearing native vegetation	Development approval
SPA / CPM Act	Tidal works or work within a coastal management district	Development approval
SPA/ Water Act	Certain operational works constructed or installed for taking or interfering with water.	Development approval
SPA / Water Act	All aspects of development for removing quarry material from a watercourse or lake if an allocation notice is required under the Water Act for the removal, other than development that is: (a) in an urban development area; or (b) carried out on premises to which structure plan arrangements apply.	Development approval
SPA / Water Supply Act	Operational work that is the construction of a dam or is carried out in relation to a dam if, because of the work, the dam must be failure impact assessed.	Development approval
SPA / Fisheries Act	Building or operational works in a declared fish habitat area. Certain operational works involving removal, destruction or damage to a marine plant. Constructing or raising waterway barrier works <u>if</u> it is not self-assessable development.	Development approval
EP Act / EP Regulation	Extractive and screening activities including dredging from the bed of any waters is specified as Environmentally Relevant Activity 16.	Environmental Authority
Water Act 2000	Taking water and using the water on any of the land or interfering with the flow of water on, under or adjoining any of the land.	Water licence
Water Act 2000	Destroying vegetation in a watercourse, lake or spring. Excavating in a watercourse, lake or spring. Placing fill in a watercourse, lake or spring.	Permit
Nature Conservation Act 1992	Taking a protected plant	Clearing permit
Environment Protection and Biodiversity Conservation Act 1999 (Cth)	For projects involving actions (controlled actions) which have an impact on matters of national environmental significance (MNES) (e.g. world heritage areas, Commonwealth marine areas, etc).	Approval of the action

3.12 Overview of key legislation regulating stormwater and drainage projects

It is important for those involved in managing drainage and stormwater projects to have a general understanding of the land use planning and regulatory regime under which they will need to operate.

A brief overview of key legislation relevant to stormwater and drainage projects is set out below. This overview is not intended to cover all legislation or all aspects of legislation relevant to stormwater and drainage projects.

3.12.1 Building Act 1975

Building works may be assessable development for which a development approval under the *Sustainable Planning Act 2009* (SPA) is required. The integrated development assessment system (IDAS) applies to applications for building works.

The *Building Act 1975* (Building Act) regulates building works and provides the laws and codes ('building assessment provisions') for assessing building works under SPA.

The 'building assessment provisions' are listed in section 30 of the Building Act and include the Building Code of Australia (BCA) and the Queensland Development Code (QDC). If the BCA is inconsistent with a part of the QDC, the part prevails to the extent of the inconsistency. Each of these 'building assessment provisions' are considered to be a code for IDAS.

The BCA contains certain performance requirements relating to the management of surface water drainage during the construction of a building. The deemed-to-satisfy provisions require compliance with AS 3500.3 – Plumbing and Drainage – Stormwater Drainage. AS 3500.3 is also referred to in the Queensland Development Code (QDC) NMP 1.8 - Stormwater Drainage. However, compliance with that part of the QDC is not mandatory.

The Building Act also requires that building works in an erosion prone area under the *Coastal Protection and Management Act 1995* (CPM Act) which include certain stormwater or drainage systems for a building must not be erected or altered in a way that is likely to cause erosion of the area.

Further, if a building development approval permits a building or land to be drained, the drainage must be carried out in a way that protects land, buildings and structures in the neighbourhood of the building or land.

3.12.2 Environmental Protection Act 1994

The *Environmental Protection Act 1994* (EP Act) and subordinate legislation form the main framework for environmental protection and management in Queensland and is relevant to stormwater and drainage projects.

The EP Act regulates a number of activities which can harm the environment as 'environmentally relevant activities' (ERA). A person may not undertake such activities without an Environmental Authority (EA) granted by the Department of Environment and Heritage Protection (EHP). Some aspects of a stormwater and drainage project may be considered an ERA for which an EA is required. An example is dredging activities, which are regulated as ERA 16.

In addition to considering the need to obtain an EA, the proponent should also consider offence provisions and the general environmental duty under the EP Act. The EP Act imposes a general environmental duty on all persons including local governments. This statutory duty requires a

person not to carry out any activity that causes, or is likely to cause, environmental harm unless the person takes all reasonable and practicable measures to prevent or minimise harm.

Aside from its approval requirements, the EP Act creates the offences of causing serious or material environmental harm. Environmental harm is any adverse effect, or potential adverse effect (whether temporary or permanent and of whatever magnitude, duration or frequency) on an environmental value. An environmental value is a quality or physical characteristic of the environment that is conducive to ecological health or public amenity or safety.

It is possible that a failure to adequately manage stormwater and drainage at a site could give rise to liability for an offence under the EP Act if that failure causes serious or material environmental harm.

In addition, it is an offence for any person to:

- unlawfully deposit a prescribed water contaminant—or to cause it to wash, blow, and fall or otherwise move into—waters or in a roadside gutter or stormwater drainage

  or

- unlawfully release stormwater runoff into waters, a roadside gutter or stormwater drainage that results in the build-up of earth in waters, a roadside gutter or stormwater drainage.

The proponent should put measures in place to ensure compliance with the general environmental duty (which may be used as a defence in proceedings for an environmental offence) and to minimise the risk of an offence being committed.

Section 57 of the Environmental Protection Regulation 2008 (EP Regulation) provides that in making certain decisions relating to an activity which involves the release of stormwater to the environment; the local government or department administering the EP Act may impose conditions to minimise impacts of stormwater release in the receiving environment. Conditions may be about diverting runoff away from areas contaminated or disturbed by the activity or installing appropriate control measures (e.g. first flush stormwater diversion systems).

The Environmental Protection (Water) Policy 2009 provides for the preparation of total water cycle management plans (the plan) by certain local governments. Local governments in South East Queensland must develop and commence implementing these plans by 30 June 2013. Local governments in major regional centres have until 30 June 2014 while there are no requirements for local government areas with population of less than 50000.

The plan must include, amongst other things, provisions about the collection, treatment and recycling of water including stormwater and provisions about urban stormwater quality management. A local government must also consider including in the plan opportunities for stormwater harvesting for use as a water source.

3.12.3   Local Government Act 2009

The *Local Government Act 2009* (LG Act) has provisions regulating stormwater drains and stormwater installation.

Under this Act, a stormwater drain is a drain, channel, pipe, chamber, structure, outfall or other works used to receive, store, transport or treat stormwater. A stormwater installation for a property is any roof gutters, downpipes, subsoil drains or stormwater drain for the property; but does not include any part of a local government's stormwater drain.

Under section 77 of the LG Act, a local government's consent is required to connect a stormwater installation for a property to the local government's stormwater drainage system.

Under section 78 of the LG Act, a property owner must not connect or allow the connection of the sewerage installation for his or her property to any part of the stormwater installation for the property or the local government's stormwater drainage system.

It is also an offence under section 79 of the LG Act to put trade waste or a prohibited substance into a local government's stormwater drainage system. Prohibited substances are defined in the LG Act and include a substance that can obstruct or interfere with the operation of a stormwater drain, a flammable or explosive substance, sewage, etc.

It is also an offence under section 80 of the LG Act to restrict or redirect the flow of stormwater over land in a way that could cause the water to collect and become stagnant unless the water is collected in a dam, wetland, tank or pond, if no offensive material is allowed to accumulate. Other legal requirements may apply to collecting water in a dam, wetland, tank or pond.

The Ci*ty of Brisbane Act 2010* has similar provisions prohibiting the connection of the sewerage system to the stormwater system.

Local governments are responsible for policing these provisions in their local government areas.

3.12.4   Plumbing and Drainage Act 2002

The *Plumbing and Drainage Act 2002* (P&D Act) regulates on-site sewerage facilities. Although it is not intended to regulate stormwater infrastructure, its section 128O requires stormwater drainage installation to be separate from on-site sewerage facility.

Section 128N of the P&D Act prohibits a person from discharging a prohibited substance into an onsite sewerage facility. Stormwater is amongst the list of prohibited substances described in the P&D Act.

3.12.5   State Planning Policy (SPP) for Healthy Waters

The SPP provides for urban development under SPA such as community infrastructure to be planned, constructed and operated to manage stormwater and waste water in ways that comply with the environmental values under the Environmental Protection (Water) Policy 2009 (see 3.11.2).

It provides a development assessment code for IDAS that applies to the assessment of certain new developments for urban purposes.

Developments triggered by the assessment under the code are those which involve an area greater than 2500 m² of land or 6 or more additional dwellings. It does not apply to development that is not subject to SPA (e.g. agriculture or mining) or to ERAs under the EP Act.

The code sets performance outcomes and acceptable outcomes for urban stormwater management. The stormwater provisions in the SPP help create consistency of development requirements across various local governments.

The SPP is supported by various guidelines including the Urban Stormwater Quality Planning Guidelines.

3.12.6 Sustainable Planning Act 2009

'Development' is defined in the SPA to include carrying out:

a) building work

b) plumbing or drainage work, as defined in the *Plumbing and Drainage Act 2002*

    and

c) operational work including – extracting gravel, rock, sand or soil from the place where it naturally occurs; excavating or filing that materially affects premises or their use; undertaking work in, on, over or under premises where that work materially affects premises or their use.

Accordingly, works involving the installation of drainage and stormwater infrastructure, or managing the drainage and stormwater impacts of works at a site, can constitute *development* under SPA.

An application is required for development that is *'assessable development'*. Development applications are assessed through the integrated development assessment system (IDAS) under SPA. The assessment criteria for an application will vary depending on the activities (or types of development) included in the application.

The relevant assessment criteria can be found in the SPA (which includes various planning instruments) and associated legislation. For example, an application for building works will be assessed against the criteria in the *Building Act 1975* i.e. the 'building assessment provisions'.

Assessment is generally undertaken by a local government, a State government department or another authority (e.g. private certifier).

A development approval must be granted for lawfully carrying out assessable development. When granting a development approval under SPA, a local government may include conditions in the development approval to deal with stormwater and drainage issues.

In accordance with section 345 of SPA a condition must be relevant to, but not an unreasonable imposition on, the development or use of premises as a consequence of the development, or be reasonably required in relation to the development or use of premises as a consequence of the development.

3.12.7 Water Act 2000

The *Water Act 2000* (Water Act) regulates the take and interference with water and, in general, it requires a water entitlement for taking or interfering with water.

Taking or interfering with water from a watercourse, lake or spring without a water entitlement is only allowed in limited circumstances including for stock and domestic purposes, emergency situations and camping purposes.

Taking overland flow water—including urban stormwater after having fallen as rain—without a water entitlement is allowed unless there is a moratorium notice a water resource plan or a wild river declaration that restricts the take. Interference with overland flow water is allowed.

Aspects of a stormwater and drainage project may involve taking or interfering with water for which a water entitlement is required. A water entitlement means a water allocation, interim water allocation or water licence.

The Water Act also provides for water authorities to be established to carry out water activities which may include stormwater drainage; flood prevention; floodwater control, etc. Under this Act stormwater drainage means a drain, channel, pipe, chamber, structure, outfall or other work used to receive, store, transport or treat stormwater.

The Water Act provides for a regulation to declare drainage and embankment areas for the purposes of the SPA. Works within a drainage and embankment area may be assessable or self-assessable for the SPA. A number of areas (generally rural) have been declared in Schedule 9 of the *Water Regulation 2002*. Operational work in the area controlling the flow of water into or out of a watercourse, lake or spring is assessable development for which a development approval is required.

Under the Water Act it is an offence to destroy vegetation, excavate or place fill in a watercourse, lake or spring without a permit under section 269. However, a permit is not required where such actions are lawfully authorised under a development permit for taking or interfering with water from a watercourse, lake or spring, or from a dam constructed on a watercourse or lake; or the interfering with overland flow water in a drainage and embankment area.

3.12.8   Water Supply (Safety and Reliability) Act 2008

The *Water Supply (Safety and Reliability) Act 2008* (Water Supply Act) provides a framework for regulating water and sewerage infrastructure and services in Queensland.

The Water Supply Act is not generally intended to regulate stormwater infrastructure. However, its section 193 prohibits a person from discharging a prohibited substance into a sewerage system. Stormwater is amongst the list of prohibited substances described in schedule 1 of the Water Supply Act.

Detention/retention basins may also be captured by the dam safety provisions of the Water Supply Act.

3.13   Other legal considerations

3.13.1   Native title

Native title has been recognised under Australian law since the early 1990s when the High Court delivered its judgement in the case of *Mabo v the State of Queensland*. The court's decision recognising native title triggered a legislative response in the form of the *Native Title Act 1993* (Native Title Act).

Under the Native Title Act any person (including a local government) that undertakes an activity which involves the doing of a 'future act' must comply with certain procedures under the legislation.

A future act is an act which affects native title (i.e. an act which extinguishes native title rights and interests or is inconsistent with their continued existence, enjoyment or exercise). Drainage and stormwater works may involve future acts if they are carried out on land over which native title subsists.

Future acts can include both physical construction activities (such as the construction of stormwater or drainage works) and non-physical activities (such as the granting of tenure and other statutory approvals required for the works).

In many locations, native title will not be relevant for a project because it will be possible to demonstrate that any native title over the site has been historically extinguished. This is the case for properties if the following has happened *on or before 23 December 1996*:

- freehold title was validly granted

  or

- if the property is not freehold – public works were established.

Native title is also extinguished in relation to roads dedicated before 23 December 1996, except where the dedication was made between 01 January 1994 and 23 December 1996 and the area dedicated had not previously been a freehold estate, a lease (but not a mining lease), or a public work.

As the existence of native title rights and the status of activities as future acts or otherwise can be a complex legal question, legal advice should always be sought in relation to the likelihood of a future act being undertaken as part of particular works.

Native title compliance may need to be considered where a drainage or stormwater project (or its upstream or downstream effects) involves unallocated State land, reserve land or other types of non-freehold land, or land that became freehold after 23 December 1996. The Native Title Act specifies the level of consultation that must be carried out for the various types of future act. A future act may require that, for example:

- a right to negotiate (RTN) process to be complied with (limited to mining and some compulsory acquisitions)
- an Indigenous Land Use Agreement (ILUA) be entered into

  or

- a notification process to be complied with.

The RTN process requires the State and the project's proponent to negotiate in good faith with the claimants, with a view to obtaining their agreement to the doing of the 'future act'.

ILUAs are voluntary agreements and must ultimately be registered by the National Native Title Tribunal. An ILUA provides for compensation measures for the temporary or permanent effects of a project on their native title rights.

Careful consideration needs to be given to various practical issues before a decision to develop an ILUA is made—including timing issues, cost issues, issues relating to overlapping native title claims and compliance with technical requirements in the Native Title Act.

Sometimes a project or activity can proceed after the proponent completes a notification process involving the relevant native title party.

Legal advice is recommended to ascertain the appropriate action required to comply with the level of consultation mandated by the Native Title Act for a particular future act.

3.13.2   Aboriginal cultural heritage

The *Aboriginal Cultural Heritage Act 2003* (ACHA) requires all persons to comply with the cultural heritage duty of care, which is a duty to ensure that in carrying out any activity, the person takes all reasonable and practicable measures to ensure the activity does not harm Aboriginal cultural heritage.

Aboriginal cultural heritage includes both objects and areas (of land or waters) which are culturally or historically significant to indigenous people. All persons (including local governments) must satisfy the statutory cultural heritage duty of care in relation to all activities which could cause harm to such objects or areas. Such activities can include any clearing, excavation or other disturbance of land associated with a drainage or stormwater project.

The onus of compliance with the duty of care is on the proponent to take such measures irrespective of the tenure of the land and irrespective of any other statutory approvals or permissions they may need to obtain.

The legislation sets out a series of statutory compliance options (including satisfying certain cultural heritage duty of care guidelines, preparing cultural heritage agreements and developing Cultural Heritage Management Plans (CHMPs)). These will deem the proponent to have complied with the duty of care and other cultural heritage protection provisions in the legislation. A CHMP is mandatory where a project requires an Environmental Impact Statement (EIS) under any legislation.

Specific legal advice may need to be sought on which of these compliance options is applicable or most appropriate in any particular case.

4.   Catchment hydrology

4.1   Hydrologic methods

The choice of hydrologic method must be appropriate to the type of catchment and the required degree of accuracy. Simplified hydrologic methods such as the Rational Method should **not** be used whenever a full design hydrograph is required for flood mapping or to assess flood storage issues. Instead the more reliable runoff-routing techniques presented in publications such as Australian Rainfall & Runoff (ARR) should be adopted.

Unless otherwise directed, a method that generates a hydrograph must be adopted for the design of those components of the drainage system which are volume dependent, such as detention basins. A detailed description of these methods is not included in this Manual.

The Rational Method provides a simplistic methodology for assessing the design peak flow rate to enable the determination of the sizes of drainage systems within urban catchments less than 500 hectares (5 km^2) in area, or rural catchments less than 25 km^2. Unfortunately the Rational Method has significant limitations, and it is the responsibility of the designer to be familiar with these limitations and to know when an alternative methodology is required.

A brief description of some commonly used hydrologic methods is given below:

4.1.1   The Rational Method

The Rational Method provides a simple means for the assessment of the peak discharge rate for design storms, but does not provide a reliable basis for the determination of runoff volume, hydrograph shape, or peak discharge rates from historical (real) storms.

Use of the Rational Method is generally not suitable for the following applications:

- analysis of historical storms
- design of detention basins
- catchments of unusual shape—refer to section 4.7
- catchments with significant, isolated areas of vastly different hydrologic characteristics, such as a catchment with an upper forested sub-catchment and a lower urbanised sub-catchment
- catchments with significant floodplain storage, detention basins, or catchments with wide spread use of on-site detention systems
- urban catchments with an area greater than 500 hectares
- catchments with a time of concentration greater than 30 minutes where a high degree of reliability is required in the hydrologic analysis.

4.1.2   Synthetic unit hydrograph procedure

The Clarke-Johnstone Synthetic Unit Hydrograph procedure is described in Australian Rainfall & Runoff (ARR, 1998) and involves the construction of a time-area diagram for the catchment, the routing of this through a linear storage and the convolution of the resulting unit hydrograph with the hyetograph to obtain a hydrograph at the point under consideration.

4.1.3   Runoff-routing models (RORB, RAFTS, WBNM and URBS)

RORB, RAFTS, WBNM and URBS are computer based runoff routing models for calculating flood hydrographs from rainfall, catchment and channel inputs. RORB is more frequently used for rural and sparsely developed catchments. RAFTS, WBNM and URBS have been widely used for both rural and urban catchments.

These models use the concept of 'critical storm duration' as opposed to the concept of 'time of concentration' used within the Rational Method. The critical storm duration for a given catchment may be similar in duration to the time of concentration, but the two terms are different and should not be confused. The critical storm duration is determined by testing the model for a range of storm durations.

Calibration of these models with actual flow data is recommended, particularly for urban areas. Where this is not possible, guidance on suitable model parameters for rural catchments is given by Weeks (1986) and McMahon and Muller (1986). These model parameters should be used in urban catchment with caution. Suggested procedures for accounting for the degree of urbanisation are provided in section 4.10. Reference should also be given to the development of regional methods as discussed in 4.1.5.

Alternatively, model results may be 'compared' with the output from other runoff-routing models. For small catchments less than 500 ha, it is common to 'compare' the results to a Rational Method peak discharge. This comparison should be to the satisfaction of the relevant regulating authority. A statement should be prepared providing justification for any differences between the models used.

Runoff-routing models such as RORB, RAFTS, WBNM and URBS can produce erroneous results when flows are extracted from the models at node locations that have just a few contributing sub-areas. This is because there may be insufficient sub-catchments to achieve a suitable balance between the calibrated rainfall runoff and flood routing components of the model. It should be noted, however, that this problem does not always occur. Ideally these models should have at least 5 sub-catchments upstream of the point of interest. Alternatively, refer to the User Guide for the computer program for guidance.

RAFTS features an automatic subdivision of each nominated sub-catchment into ten sub-areas which is thought to significantly reduce the risk of erroneous results when stream flows are extracted from the model at nodes that have just a few contributing sub-catchments.

4.1.4   Time-area runoff routing (e.g. DRAINS and PC-DRAIN)

DRAINS and PC-DRAIN are computer-based models that incorporate the routing of the time-area relationship developed for the sub-catchments under consideration.

DRAINS was developed from the TRRL Method, ILLUDAS and later, ILSAX. It is suitable for use in urban catchments, but requires calibration with available flow data. Where this is not available it is recommended that the obtained hydrograph be 'compared' with the peak discharge derived for the same catchment using the Rational Method—noting the issues raised in 4.1.1..

4.1.5   Regional flood frequency analysis

Project 5 of Engineers Australia's Australian Rainfall and Runoff Review is working on the development of regional flood methods for ungauged rural streams. This project is expected to develop appropriate regional equations for small to medium sized (8 to 1000 km^2) rural catchments (< 10% urban) for both coastal and semi-arid regions of Queensland. Stormwater designers should take appropriate steps to obtain the latest information from this project.

4.2 Hydrologic assessment

4.2.1 Hydrologic assessment of catchments not fully developed

Traditional drainage standards require design discharge rates to be based on a fully developed catchment in accordance with the current Planning Scheme or Strategic Plan. Unless otherwise directed by the local government, the design discharge rate has traditionally assumed no flow attenuation within future upstream developments.
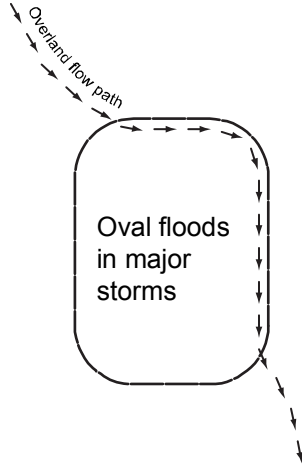
The main benefit of this practice was that it minimised the need for stormwater detention/retention systems within developing catchments. The disadvantages associated with this approach are:

* accelerated downstream watercourse erosion if runoff from the upstream development is not regulated to avoid increases in discharge
* the high cost of trunk drainage systems.

Even with current stormwater practices it may not always be appropriate to assume future upstream developments will adequately attenuate flows. For example, parts of the upper catchment may have been approved for development under an old Planning Scheme where flow attenuation was not required. Developers need to obtain guidance from the local government as to what flow conditions should be assumed for the fully developed upstream catchment.

4.2.2 Examples of catchments where application of the Rational Method is generally not recommended

The following section provides guidelines on the hydrologic assessment of catchments which contain features that are likely to significantly limit the applicability of the Rational Method. These catchment conditions are assumed to exist upstream of the location where a design discharge is being determined.

Oval floods in major storms	**Catchment 1:** **Overland flow path passing through a low gradient oval or park that provides significant detention storage during major storm events**
	• Use of the Rational Method to calculate peak flows downstream of the oval/park is **not** recommended. Note; it is inappropriate to use the very low flow velocities passing through the flooded oval to determine a time of concentration downstream of the oval. • Peak flow should be determined using a runoff-routing model that adequately accounts for flood storage, i.e. the oval/park may need to be modelled as a detention system.

Catchment 2:

Catchments where travel time for the minor drainage system is significantly different from that of the major drainage (overland flow) system

- If the Rational Method is used, then the time of concentration should be based on the shortest travel time, otherwise use a runoff-routing model.

- If the assessment of peak discharge is a critical design issue, then an appropriate runoff-routing model should be used.

Catchment 3:

Relief drainage works incorporating split pipe flows

- The assumed flow rate in each pipe should **not** be based on pipe gradient, but on an appropriate hydraulic gradeline analysis.

- Alternatively, use an appropriate pipe network or time-area runoff routing model to analyse the drainage system (preferred method).

Catchment 4:

The upstream catchment is zoned for urban usage, but is currently undeveloped

- Design flow rates should be based on ultimate development of the catchment based on the current Planning Scheme or Strategic Plan, whichever results in higher flows.

- It should be assumed that future upstream development would alter existing flow conditions, unless otherwise agreed in principle by the local government or specified through Planning Scheme requirements.

- Refer to discussion in section 4.2.1.

OSD tank Stormwater pipe	**Catchment 5:** **Catchments containing significant on-site stormwater detention (OSD)**
	• Use of the Rational Method to calculate minor storm flow rates is likely to be inappropriate. • The Rational Method may be used if applied in a conservative manner (i.e. OSD systems are ignored). • Typically the OSD systems should be ignored when analysing major storm events. • For those areas with on-site detention, the local government may agree to the adoption of a runoff coefficient (*C*) based on an appropriate pre-development land use. • Alternatively, use a runoff-routing model with appropriate allowances made for the OSD systems.

Basin	**Catchment 6:** **Sub-catchments containing one or more large lakes, wetlands or detention/retention basins**
	• Hydrological analysis should be performed using an appropriate runoff-routing model. • Use of the Rational Method to calculate peak flow rates downstream of the water storage is not recommended.

Dam	**Catchment 7:** **Catchments containing a major water supply dam or weir**
	• Use of the Rational Method is not recommended. • General practice is to assume the dam/weir is full at the start of the design storm.

Dam	**Catchment 8:** **Catchments with an upper rural area containing a farm dam**
	• It should be assumed that the farm dam may one-day be removed (even if the area stays rural) therefore, design flows downstream of the dam should consider both the dam and no dam condition. • In effect, downstream residents should not place an unreasonable onus on upstream residents for the flood protection of their (downstream) properties. Consequently downstream landowners should adequately plan for the changing hydrologic conditions resulting from the 'normal use' of upstream properties. • General practice is to assume existing dams are full at the start of the design storm.

	Catchment 9: **Urban catchments with an area greater than 500 ha.**
	• Use of the Rational Method is not recommended. • Catchments should be analysed using an appropriate runoff-routing model. • Runoff-routing models of ungauged urban catchments can be compared to the Rational Method, but only at locations where the upstream catchment areas is less than 500 ha.
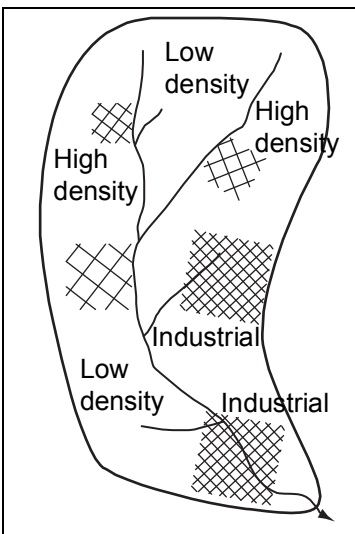
	Catchment 10: **Catchments developed using the principles of Water Sensitive Urban Design**
	• Applicability of the Rational Method will depend on the degree of on-site detention (refer to Catchment 5). • The Rational Method may be appropriate for the determination of peak discharge rates for major design storms for catchment less than 500 ha. • Alternatively, use an appropriate runoff-routing model. The above discussion does not apply to water quality modelling of WSUD systems.
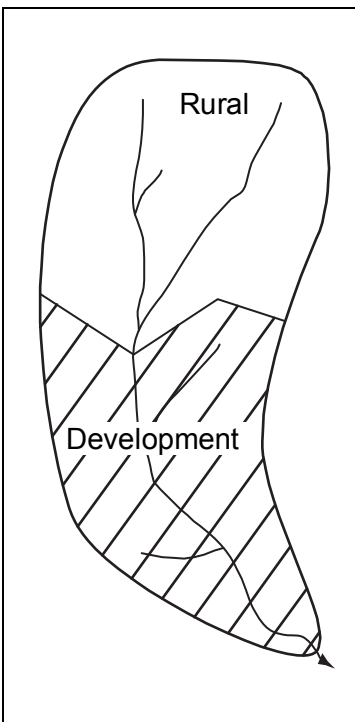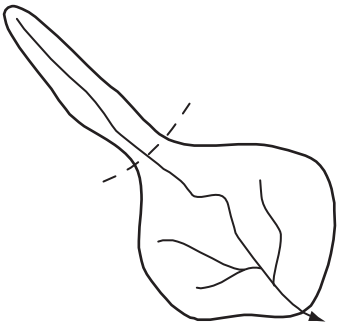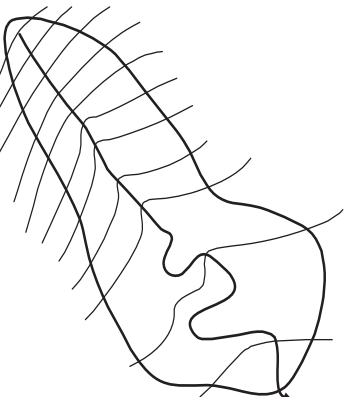
	Catchment 11: **Partially urbanised, ungauged catchments**
	• Use of the Rational Method may produce highly erroneous results. • Catchments should be analysed using an appropriate runoff-routing model or regional flood methods. • Results from runoff-routing models of ungauged catchments may be 'compared' with results from a Rational Method analysis for catchments less than 500 ha using the following procedure: (i) 'compare' the results assuming a fully undeveloped catchment (i.e. assuming the lower catchment is undeveloped) (ii) 'compare' the results assuming a fully developed catchment (i.e. assuming the upper catchment is developed) (iii) adjust the parameters used in the runoff-routing model based on past experience of similar catchments and the results of steps (i) and (ii). Note; output from the runoff-routing model should not be calibrated to precisely match the Rational Method unless there is reasonable and logical justification.

	Catchment 12:
	Irregular shaped catchments
	• Non-critical design discharge rates may be determined using the Rational Method with appropriate adjustments made to the time of concentration (refer to section 4.7).
	• In critical locations or where an accurate estimation of design discharge is required, use an appropriate runoff-routing model.

	Catchment 13:
	Catchments with a significant change in catchment slope or stream slope
	• Use of the Rational Method may produce highly erroneous results. In some cases, an estimate of design discharge rates may be determined using the procedures presented in section 4.7.
	• Catchments should be analysed using an appropriate runoff-routing model.

4.3   The Rational Method

In its general form (using the non-standard units of Q (m³/s), I (m/s) and A (m²)) the Rational Formula is:

$$Q = C \cdot I \cdot A \qquad (4.1)$$

For design purposes, the units of the key variables are changed to their more common form (Q (m³/s), I (mm/hr) and A (ha)) and the formula becomes:

$$Q_y = (C_y \cdot {}^{t}I_y \cdot A)/360 \qquad (4.2)$$

where:

Q_y = peak flow rate (m³/s) for annual exceedence probability (AEP) of 1 in 'y' years

C_y = coefficient of discharge (dimensionless) for AEP of 1 in 'y' years

A = area of catchment (ha)

${}^{t}I_y$ = average rainfall intensity (mm/h) for a design duration of 't' hours and an AEP of 1 in 'y' years

t = the nominal design storm duration as defined by the time of concentration (t_c)

The value '360' is a conversion factor to suit the units used.

Calculation of the flow at the various inlets and junctions along the drainage line is carried out from the top of the system progressively downstream.

The total peak flow at any point is **not** the sum of the calculated sub-area flows contributing at that point, but is **dependent on the time of concentration** at that point. The actual flow being the

product of the sum of the *C.A* values of the contributing sub-catchments, multiplied by ${}^{t}I_y$ appropriate for time of concentration at that point.

$$Q_{peak} = (2.78 \times 10^{-3}).[\Sigma(C.A)].\ {}^{t}I_y\ (m^3/s) \tag{4.3}$$

The time of concentration (t_c) is defined as the time for flow to travel from the most remote part of the catchment to the outlet, or the time taken from the start of rainfall until all of the catchment is simultaneously contributing flow to the outlet.

The Rational Method should not be used to analyse historical (real) storms. For additional explanation of the Rational Method refer to Books 4 and 8 of Australian Rainfall & Runoff (1998).

4.4   Catchment area

The boundaries of catchment areas may be determined from contour maps, council records, aerial photographs and field inspections. When selecting the catchment area the following issues and guidelines should be considered:

- Where the contributing catchment includes existing subdivided areas, the location of existing drainage works needs to be determined, either by field inspection, council records, or from 'As Constructed' drainage plans.

- In urbanised catchments, ridgelines should not automatically be adopted as catchment boundaries because pipe drainage systems may collect and carry stormwater across these natural catchment boundaries.

- The catchment area should take into account likely future road layouts and road drainage patterns if the contributing catchment includes areas subject to future development.

- In older urban areas where existing roads may have a high crown, significant quantities of stormwater runoff may be redirected by the crown of the road (Figure 4.1). When determining the catchment area, appropriate consideration should be given to the likelihood that the road will one-day be resurfaced and re-profiled, causing stormwater to return to its 'natural' flow path (Figure 4.2). In such cases, the design of new drainage works must adopt a conservative catchment area.
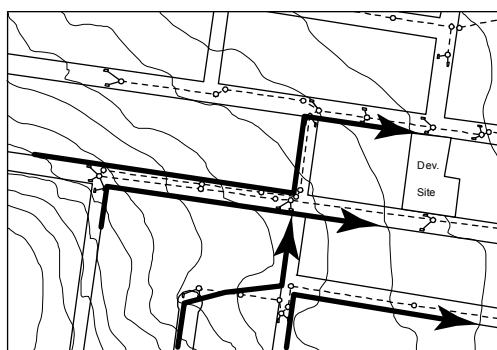
Figure 4.1 – Kerb flow diverted by road crown
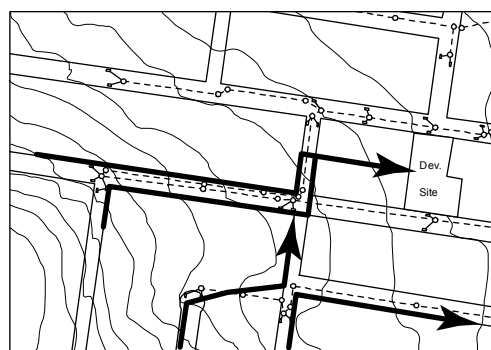
Figure 4.2 – Surface flow following re-profiling of the road crown

- When assessing catchment boundaries, allowance should be made of the possible piping of runoff against the natural ground slope (e.g. the discharge of roof water drainage to the street even though the property site below the road elevation). This may be especially significant in industrial and commercial areas where factory roofs and surrounding car parks may drain in opposite directions.

- Roads, fences and pathways may significantly alter catchment boundaries. Property fencing and sound-control fencing can either block or significantly alter the direction of surface runoff.

- The effective catchment area of the minor drainage system may be different from the catchment area of the major drainage system. In some cases the piped drainage system may discharge to a location different from that of the overland flow.

- In small urban catchments, the effective catchment boundary may be governed by the location of allotment boundaries as shown in Figure 4.3.

Figure 4.3 – Catchment boundaries of the natural catchment (dotted line) and the actual drainage catchment (solid line)

4.5   Coefficient of discharge

The coefficient of discharge, *'C'* is a coefficient used within the Rational Method. The value of *C* is linked, in a complex manner, to the infiltration characteristics of the catchment and impacts of other runoff 'losses'. It should **not** be confused with the *volumetric runoff coefficient 'C$_V$'*, which is a direct ratio of total runoff to total rainfall.

The coefficient of discharge must account for the future development of the catchment as depicted in the Planning Scheme or zoning maps for the relevant local government, but should not be less than the value determined for the catchment under existing conditions.

It is recommended that the coefficient of discharge should be calculated using the method presented in Book 8 of ARR (1998), with the exception of 100% pervious surface. This method is summarised in the following steps:

STEP 1   Determine the fraction impervious (f_i) for the catchment under study from Table 4.5.1.

STEP 2   Determine the 1 hour rainfall intensity ($^1I_{10}$) for the 10 year ARI (10% AEP) at the locality – refer to section 4.8.

STEP 3   Determine the frequency factor (F_y) for the required design storm from Table 4.5.2.

STEP 4   Determine the 10 year discharge coefficient (C_{10}) value from tables 4.5.3 and 4.5.4.

STEP 5   Multiply the C_{10} value by the frequency factor (F_y) to determine the coefficient of runoff for the design storm (C_y).

$$C_y = F_y \cdot C_{10} \qquad\qquad (4.4)$$

Note: In certain circumstances the resulting value of C_y will be greater than 1.0. In accordance with the recommendations of ARR (1998), a limiting value of $C_y = 1.0$ should be adopted for urban areas.

There is little evidence to support an allowance for either slope or soil type in fully developed (non WSUD) urban areas. If there are significant local effects, and reliable data is available, then adjustments for soil type may be incorporated within the calculations at the discretion of the designer in consultation with the relevant local authority.

The relationships presented in Book 8 of ARR (1998) and adopted in this Manual apply to areas that are essentially homogeneous, or where the pervious and impervious portions are so intermixed that an average is appropriate. In cases where separable portions of a catchment are significantly different, they should be divided into sub-catchments and different values of *C* applied.

Notwithstanding the above notes and limitations, it is the responsibility of the designer to ensure each sub-catchment flow is determined using a suitable coefficient of discharge. The local government may set specific *C*-values to be used within their area.

Table 4.5.1 – Fraction impervious vs. development category

Development category	Fraction impervious (f_i)
Central business district	1.00
Commercial, local business, neighbouring facilities, service industry, general industry, home industry	0.90
Significant paved areas e.g. roads and car parks	0.90
Urban residential – high density	0.70 to 0.90
Urban residential – low density (including roads)	0.45 to 0.85
Urban residential – low density (excluding roads)	0.40 to 0.75
Rural residential	0.10 to 0.20
Open space and parks etc.	0.00

Notes (Table 4.5.1):

1. Designer should determine the actual fraction impervious for each development. Local governments may specify default values.

2. Typically for urban residential high density developments:

    townhouse type development $f_i = 0.7$

    multi-unit dwellings > 20 dwellings per hectare $f_i = 0.85$

    high-rise residential development $f_i = 0.9$

3. In urban residential low density areas f_i will vary depending upon road width, allotment size, house size and extent of paths, driveways etc.

4. Refer to Table 7.3.3 for the definition of development categories.

Table 4.5.2 – Table of frequency factors

AEP (%)	ARI (years)	Frequency factor (F_y)
63%	1	0.80
39%	2	0.85
18%	5	0.95
10%	10	1.00
5%	20	1.05
2%	50	1.15
1%	100	1.20

Table 4.5.3 – Table of C_{10} values

Intensity (mm/hr) $^1I_{10}$	Fraction impervious f_i						
	0.00	**0.20**	**0.40**	**0.60**	**0.80**	**0.90**	**1.00**
39-44	Refer to Table 4.5.4	0.44	0.55	0.67	0.78	0.84	0.90
45-49		0.49	0.60	0.70	0.80	0.85	0.90
50-54		0.55	0.64	0.72	0.81	0.86	0.90
55-59		0.60	0.68	0.75	0.83	0.86	0.90
60-64		0.65	0.72	0.78	0.84	0.87	0.90
65-69		0.71	0.76	0.80	0.85	0.88	0.90
70-90		0.74	0.78	0.82	0.86	0.88	0.90

Refer to notes over page.

Table 4.5.4 – C_{10} values for zero fraction impervious [1]

Land description	Dense bushland			Medium density bush, or Good grass cover, or High density pasture, or Zero tillage cropping			Light cover bushland, or Poor grass cover, or Low density pasture, or Low cover bare fallows		
Intensity (mm/hr) $^1I_{10}$	Soil permeability			Soil permeability			Soil permeability		
	High	Med	Low	High	Med	Low	High	Med	Low
39–44	0.08	0.24	0.32	0.16	0.32	0.40	0.24	0.40	0.48
45–49	0.10	0.29	0.39	0.20	0.39	0.49	0.29	0.49	0.59
50–54	0.12	0.35	0.46	0.23	0.46	0.58	0.35	0.58	0.69
55–59	0.13	0.40	0.53	0.27	0.53	0.66	0.40	0.66	0.70
60–64	0.15	0.44	0.59	0.30	0.59	0.70	0.44	0.70	0.70
65–69	0.17	0.50	0.66	0.33	0.66	0.70	0.50	0.70	0.70
70–90	0.18	0.53	0.70	0.35	0.70	0.70	0.53	0.70	0.70

Refer to note over page.

Notes (Table 4.5.3):

$^1 I_{10}$ = One hour rainfall intensity for a 1 in 10 year ARI (10% AEP)

C_{10} = Coefficient of discharge for a 1 in 10 year ARI (10% AEP)

f_i = Fraction impervious

Note (Table 4.5.4):

[1] Developed from Department of Natural Resources and Mines (2004). These coefficients are not suitable for soils compacted by construction activities.

4.6 Time of concentration (Rational Method)

4.6.1 General

The time of concentration (t_c) of a catchment is defined as the time required from the start of a design storm for surface runoff to collect and flow from the most remote part of the catchment to its outlet. Its significance is in the assumption that for a given design storm frequency, peak flow at the catchment outlet will result from a storm of duration equal to the time of concentration. In reality this is not always the case and it is the task of the designer to be aware of the correct determination and application of the time of concentration.

It is noted that the time of concentration as used in the Rational Method is **not** the same as the *'critical storm duration'* or *'time to peak'* as determined from runoff-routing models, such as RAFTS, RORB and WBNM. It is therefore inappropriate to adopt the critical storm duration determined from a runoff-routing model and apply it as the time of concentration within a Rational Method analysis.

In certain circumstances, partial area effects need to be considered for a catchment and these are discussed in section 4.7.

In determining the time of concentration, the designer should adopt the appropriate catchment conditions in accordance with the required analysis. Flow conditions should be based on a fully developed catchment in accordance with the allowable land use shown in the relevant Strategic Plan, or as directed by the local authority.

The following discussion is relevant to the application of the Rational Method.

In a typical urban drainage system, a designer will need to calculate time of concentration for two purposes:

- To allow calculation of the runoff from sub-catchments in order to determine the position and size of inlets required to satisfy criteria such as flow-width (in the case of a minor storm) or roadway discharge capacity (in the case of a major storm). This time of concentration is known as the 'inlet time'.

- To size a pipe or channel draining a number of sub-catchments based upon the total area of the sub-catchments contributing to the upstream end of the drain and the time of concentration of that area.

4.6.2 Minimum time of concentration

Although travel time from individual elements of a system may be as short as two minutes, the total nominal flow travel time to be adopted from any catchment to its point of entry into the drainage network should not be less than 5 minutes.

4.6.3 Methodology of various urban catchments

By its nature the Rational Method is a very simple hydrologic model that depends on its original development and calibration to achieve reasonable flow estimation values for catchments of typical shape and surface condition. This equation addresses variations in rainfall loss, surface storage, and the rate (speed) of surface runoff in a very simplistic manner.

Within the Rational Method both the 'runoff coefficient' and the 'time of concentration' are adjusted to account for typical variations in catchment conditions as described below:

- The runoff coefficient (C) takes account of variations in catchment porosity, and hence the selection of this coefficient is usually related to the *fraction impervious* (f_i).

- The runoff coefficient (C) also takes account of variations in rainfall losses relative to the rainfall intensity/volume, and thus the coefficient is adjusted using the *frequency factor* (F_y).

- The average rainfall intensity (I) takes account of variations in the rainfall intensity for different storms through the use of *time of concentration (t_c)*.

- Indirectly, the nominated average rainfall intensity *(I)* may also need to take account of typical variations in channel/floodplain storage through variations in the methodology used to calculate the time of concentration. An example of this is the Stream Velocity Method (section 4.6.12(c)) which calculates a time of concentration not necessarily representative of the actual flow travel time, but instead an 'assumed' travel time, determined through stream gauging and model calibration.

To apply the Rational Method in an appropriate and consistent manner, five different methodologies for determination of the time of concentration are presented below for different types of drainage catchments. Those catchment types being:

(a) Predominantly piped or channelised urban catchments less than 500 ha with the top of the catchment being urbanised.

(b) Predominantly piped or channelised urban catchments less than 500 ha with the top of the catchment being bushland or a grassed park.

(c) Bushland catchments too small to allow the formation of a creek with defined bed and banks.

(d) Urban creeks with a catchment area less than 500 ha.

(e) Rural catchments less than 500 ha.

A summary of the typical components that make up the determination of the 'time of concentration' are presented in Table 4.6.1. This table provides only an indication of the flow travel components that typically make up the total travel time. Unusual drainage catchments will need to be assessed on a case-by-case basis. A detailed discussion of each catchment condition follows this table.

Table 4.6.1 – Summary of typical components of time of concentration

Catchment conditions	Standard inlet time	Overland sheet flow	Concentrated overland flow	Kerb flow time	Pipe flow time	Channel flow time	Creek flow travel time
(a) Urban piped catchment (< 500 ha) with urban development at the top of catchment	Yes			Yes	Yes	Yes	
(b) Urban piped catchment (< 500 ha) with park/bush at top of catchment		Yes	Yes	Yes	Yes	Yes	
(c) Small, non-piped catchment (< 500 ha) with no formal creek		Yes	Yes			Yes	
(d) Urban creek (< 500 ha) with <u>no</u> floodplain storage	As above for the appropriate catchment conditions					Yes	
(d) Urban creek (< 500 ha) with *significant* floodplain storage							Yes
(e) Rural creek catchment (< 500 ha)							Yes

(a)   Predominantly piped or channelised urban catchments less than 500 ha with the top of the catchment being urbanised

Components of time of concentration:

- *Standard inlet time*—from section 4.6.4. Alternatively calculate travel time from roof to kerb using section 4.6.5.

- **Note:** The 'standard inlet time' includes the travel time along a typical length of kerb/channel from near the top of the catchment to the first pipe or channel inlet. If the actual length of kerb/channel travel is unusually long, then an additional travel time must be added to the standard inlet time (step (ii) below). If a gully/field inlet does not exist near the top of catchment, then use sections 4.6.5 and/or 4.6.6 to determine the initial travel time to the start of the kerb/channel; then add the travel time along the kerb/channel.

- *Kerb flow time*—from figures 4.9 and 4.10 only if the length of kerb exceeds that which would normally exist at the top of a catchment.

- *Pipe flow time*—using actual flow velocities determined from a pipe network analysis or Manning's equation. Alternatively, if the pipe flow time is not critical, an average pipe flow velocity of 2 m/s and 3 m/s may be adopted for low gradient and medium to steep gradient pipelines respectively.

- *Creek and/or channel flow time*—using the 'actual' stream flow velocity determined from numerical modelling or Manning's equation (**not** values from Table 4.6.6). Alternatively, if the expected travel time in the creek is not critical, an average flow velocity of 1.5 m/s may be adopted (not applicable to constructed channels).

(b)   Predominantly piped or channelised urban catchments less than 500 ha with the top of the catchment being bushland or a grassed park

Components of time of concentration:

• *Sheet flow travel time*—estimate the length of sheet runoff at top of catchment from field observations noting the maximums presented in Table 4.6.4, then estimate the sheet flow travel time as per section 4.6.6.

• *Concentrated flow overland travel time*—determine the distance of concentrated overland flow from the end of the 'sheet flow' runoff to the nearest kerb, pipe inlet, open channel or creek. Then determine the travel time for this concentrated overland flow based on the calculated flow velocity.

• *Kerb flow time*—as per figures 4.9 and 4.10.

• *Pipe flow time*—using **actual** flow velocities determined from a pipe network analysis or Manning's equation. Alternatively, if the pipe flow time is not critical, an average pipe flow velocity of 2 m/s and 3 m/s may be adopted for low gradient and medium to steep gradient pipelines respectively.

• *Creek and/or channel flow time*—using the 'actual' stream flow velocity determined from numerical modelling or Manning's equation (**not** values from Table 4.6.6). Alternatively, if the expected travel time in the creek is not critical, an average flow velocity of 1.5 m/s may be adopted (not applicable to constructed channels).

(c)   Bushland catchments too small to allow the formation of a creek with defined bed and banks

Time of concentration determined using the procedure listed in (b) above.

(d)   Urban creeks with a catchment area less than 500 ha

Time of concentration for an urban catchment containing a watercourse with defined bed and banks may be determined as for rural catchments (section 4.6.12) provided the following conditions apply:

• channel/floodplain storage along the watercourse—for the catchment condition being analysed— is not significantly reduced from the natural (i.e. pre-urbanisation) conditions

• less than 20% of the catchment drains to a pipe network.

If the above conditions do not apply, then the time of concentration should be based on the procedures outlined in (a) or (b) above as appropriate for the catchment conditions.

Technical note 4.6.1

Use of the Rational Method is generally not recommended for urban catchments greater than 500 ha, or rural catchment greater than 25 km^2.

Hydrologic analysis of urban catchments greater than 500 ha should be performed using a combination of suitable runoff-routing modelling and dynamic hydraulic modelling. Designers should refer to the latest recommendations of Australian Rainfall and Runoff.

(e)   Rural catchments less than 500 ha

Recommended procedures for the determination of the time of concentration for rural catchments as outlined in section 4.6.12.

4.6.4   Standard inlet time

Use of standard inlet times for developed catchments is recommended because of the uncertainty related to the calculation of time of overland flow. The standard inlet time is defined as the travel time from the top of the catchment to a location where the first gully or field inlet would normally be expected to exist, as depicted in Figure 4.4.
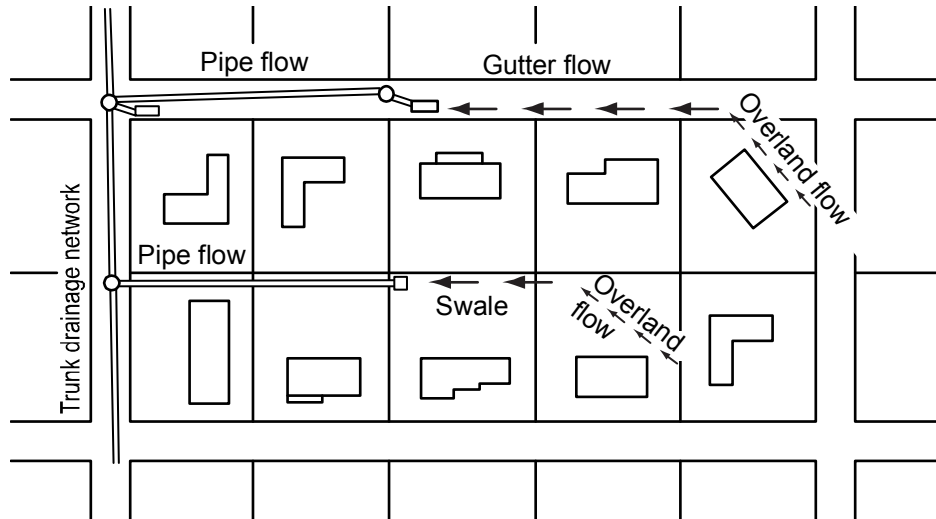
Figure 4.4 – Application of standard inlet time

Recommended standard inlet times are presented in Table 4.6.2. These inlet times are considered appropriate for traditional (i.e. non WSUD) low density residential areas where the top of the catchment is low density residential, but not a park or bushland.

If the top of the catchment consists of high density residential, then the local government should be consulted for inlet times appropriate for the catchment. In such cases it is recommended that the standard inlet time should not exceed 10 minutes unless demonstrated otherwise by the designer.

If the hydrologic analysis is being performed on a small urban development located at the top of the catchment, then use of a standard inlet time may not be appropriate because the inlet time may be significantly greater than the actual travel time for the drain being designed e.g. in the design of minor drains upstream of the first gully inlet.

If the first gully or field inlet is located further down the catchment slope than would normally be expected, then the standard inlet time shall only account for the travel time down to the location where the first gully or field inlet would normally have been located.

If the urban drainage system does not incorporate pipe drainage (i.e. no gully or field inlet exists) then the standard inlet time shall extend down the catchment to a location where a gully inlet would normally be located in a traditional kerb-and-channel drainage system.

A standard inlet time should **not** be adopted in sub-catchments where detailed overland flow and kerb/channel flow calculations are justified.

Table 4.6.2 – Recommended standard inlet times

Location	Inlet time (minutes)
Road surfaces and paved areas	5
Urban residential areas where average slope [1] of land at top of catchment is greater than 15%	5
Urban residential areas where average slope [1] of land at top of catchment is greater than 10% and up to 15%	8
Urban residential areas where average slope [1] of land at top of catchment is greater than 6% and up to 10%	10
Urban residential areas where average slope [1] of land at top of catchment is greater than 3% and up to 6%	13
Urban residential areas where average slope [1] at top of catchment is up to 3%	15

Note (Table 4.6.2):

[1]   The average slopes referred to in this table are the slopes along the predominant flow path for the catchment in its developed state.

A local government may determine that the use of standard inlet times shall not apply within their area and may direct designers to use alternative methods.

In certain circumstances the use of standard inlet times may result in times of concentration unacceptably short for the catchment under consideration, such as airports, or large flat car parks. In these cases the designer should utilise other methods (e.g. Friend's equation or the Kinematic Wave equation) to determine the time of initial overland flow (refer to section 4.6.6). Inlet times calculated by these methods should only be adopted for design if the sheet flow length criteria discussed in section 4.6.6 are met, and if due consideration is given to the type and continuity of the surface where overland flow is occurring.

Notwithstanding the above, it is recommended that a **maximum** inlet time of 20 minutes be adopted for urban and residential catchments, including playing fields and park areas.

4.6.5   Roof to main system connection

In cases where use of a standard inlet time is not considered appropriate, the following roof to main system flow travel times are recommended:

Table 4.6.3 – Recommended roof drainage system travel times

Development category	Time to point A (minutes)
Rural residential, residential low-density For the roof, downpipes and pipe connection system from the building to the kerb and channel or a rear-of-allotment drainage system (Figure 4.5(a)).	5
Residential medium and high-density, commercial, industrial and central business district For the roof and downpipe collection pipe to the connection point to the internal allotment drainage system abutting the building (Figure 4.5(b)).	5

Note: The flow time from point A (Figure 4.5) through the internal allotment pipe system to the kerb and channel, street underground system or rear of allotment system for the more intense developments noted should be calculated separately.
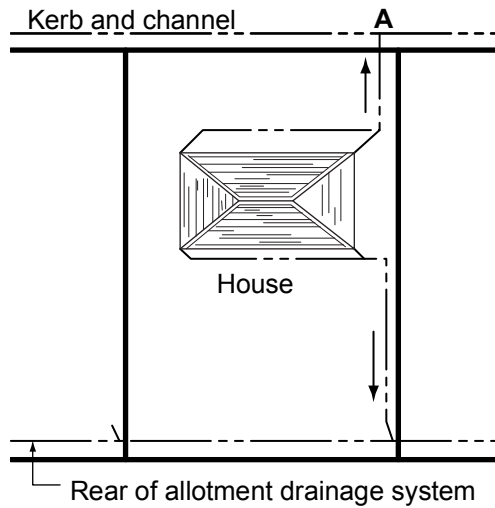
Figure 4.5 (a) – Typical roof drainage systems (residential)
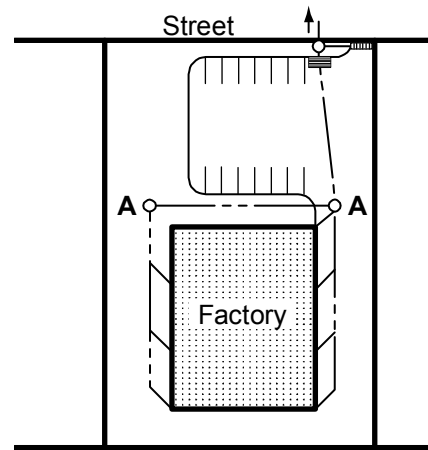
Figure 4.5 (b) – Typical roof drainage systems (industrial)

Note (Figure 4.5): Point A is referred to in Table 4.6.3

4.6.6 Overland flow

(a) General

Overland flow at the top of a catchment will initially travel as 'sheet flow', after which it will move down the catchment as minor 'concentrated flow'. Travel times for the sheet flow and concentrated flow components need to be determined separately.

The sheet flow travel time is defined as the travel time from the top of a catchment to the point where stormwater runoff begins to concentrate against fences, walls, gardens, or is intercepted by a minor channel, gully or piped drainage. This concentration of flow may also occur in the middle of vegetated areas as the stormwater concentrates in minor drainage depressions.

The time required for water to flow over a homogeneous surface such as lawns and gardens is a function of the surface roughness and slope. There are a number of methods available for the determination of sheet flow travel times and a local government may direct which of these methods shall be applied. Two such methods are presented in this section.

Irrespective of which method of calculation is adopted, it is the designer's responsibility to determine the effective length of this sheet flow.

In urban areas, the length of overland sheet flow will typically be 20 to 50 metres, with 50 metres being the recommended maximum. In rural residential areas the length of overland sheet flow should be limited to 200 m (Argue, 1986), however the actual length is typically between 50 and 200 m, where after the flow will be 'concentrated' in small rills, channels, or tracks.

(b)   Design Steps

To determine the *overland flow travel time* the following steps should be applied:

- Where practical, inspect the catchment to determine the length of initial overland sheet flow, or for new developments measure the length of overland flow from the design plans.

- Where it is not practical to inspect the catchment, determine the likely length of overland sheet flow based on Table 4.6.4.

- Determine the sheet flow travel time using either the Friend's equation (equation 4.5 – preferred method) or the Kinematic Wave equation (equation 4.6).

- Determine or measure the remaining distance of assumed concentrated overland flow from the end of the adopted sheet flow to the nearest kerb, channel, or pipe inlet.

- Determine the concentrated flow travel time using either Figure 4.8 or Manning's equation.

Table 4.6.4 – Recommended maximum length of overland sheet flow

Surface condition	Assumed maximum flow length (m)
Steep (say >10%) grassland (Horton's n = 0.045)	20
Steep (say >10%) bushland (Horton's n = 0.035)	50
Medium gradient (approx. 5%) bushland or grassland	100
Flat (0–1%) bushland or grassland	200

(c)   Friend's equation/nomograph for overland sheet flow time (preferred method for overland flow calculation)

The formula shown below and attributed to Friend (1954) may be used for determination of overland sheet flow times. This was derived from previous work in the form of a nomograph for shallow sheet flow over a plane surface (Figure 4.6). This assessment process is recommended in preference to that presented in (d) below. It is noted that values for Horton's *'n'* are similar to those for Manning's *'n'* for similar surfaces.
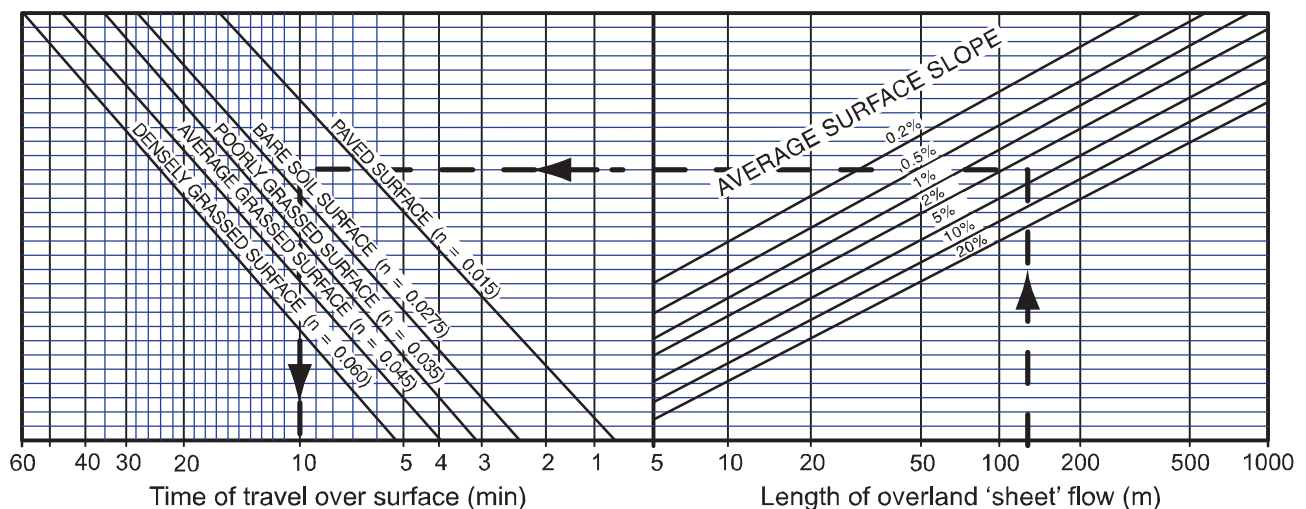
Figure 4.6 – Overland sheet flow times (shallow sheet flow only) (source: ARR, 1977)

Friend's equation:

$$t = (107n\, L^{0.333})/S^{0.2} \qquad (4.5)$$

where:

t = overland sheet flow travel time (min)

L = overland sheet flow path length (m)

n = Horton's surface roughness factor

S = slope of surface (%)

(d) Kinematic wave equation for overland sheet flow time

The kinematic wave equation for overland travel time developed by Ragan & Duru (1972) may also be used; however, it should only be applied to planes of sheet flow that are homogenous in slope and roughness. Thus, travel times need to be determined separately for areas of different slope or roughness.

As shown by McCuen (1984) it cannot be applied to large heterogeneous catchments. The Kinematic Wave equation is best applied to large paved areas such as car parks and airports.

$$t = 6.94\, (L.n^*)^{0.6} /(I^{0.4}.S^{0.3}) \qquad (4.6)$$

where:

t = overland travel time (min)

L = overland sheet flow path length (m)

n^* = surface roughness/retardance coefficient

I = rainfall intensity (mm/hr)

S = slope of surface (m/m)

Typical values for n* are presented below:

(i) As quoted by Argue (1986) p. 28.

- o Paved surfaces = 0.015
- o Lawns = 0.25
- o Thickly grassed surfaces = 0.50

(ii) As derived from ARR (1998), Book 8, Table 1.4.

Table 4.6.5 – Surface roughness or retardance factors

Surface type	Horton's roughness coefficient n*
Concrete or Asphalt	0.010 – 0.013
Bare Sand	0.010 – 0.016
Gravelled Surface	0.012 – 0.030
Bare Clay-Loam Soil (eroded)	0.012 – 0.033
Sparse Vegetation	0.053 – 0.130
Short Grass Paddock	0.100 – 0.200
Lawns	0.170 – 0.480

Notes (Table 4.6.5):

1. The surface roughness/retardance coefficient n* is similar but not identical to Manning's n value for surface roughness.

2. For further details of this procedure reference should be made to technical note 3, Book 8, ARR (1998).

3. Experience both locally and as quoted by McCuen (1984) indicates that the kinematic wave equation tends to result in excessively long overland sheet flow travel times.
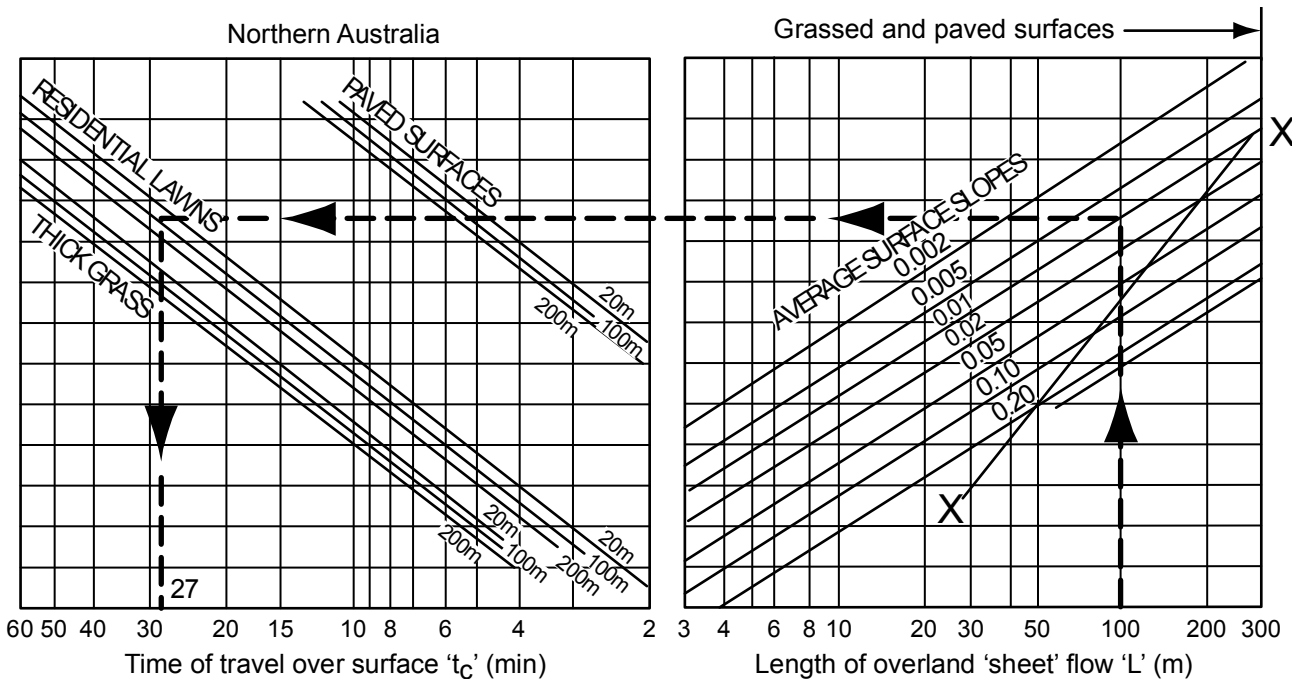
Figure 4.7 – Overland sheet flow times using kinematic wave equation (Source: Argue, 1986)

Technical notes for Figure 4.7

Based on rainfall intensity = 125 mm/hr.

The boundary X-X defines the practical limit of 'sheet flow' path length on grass or unpaved surfaces. For example, for 0.20 grassed slopes = 50 m, or for 0.05 grassed slopes = 120 m.

Pervious surface flow distances exceeding these limits should be treated as natural channel flow. Flow velocity should be determined using the Manning's equation based on expected operating conditions.

4.6.7   Initial estimate of kerb, pipe and channel flow time

An initial (trial) estimate of flow time can be determined from Figure 4.8. The chart may be used directly to determine approximate travel times along a range of rigid channel types and, with the application of multiplier Δ for a range of loose-boundary channel forms.

Technical notes for Figure 4.8

Flow travel time (approximate) may be obtained directly from this chart for:

kerb-and-gutter channels

- stormwater pipes
- allotment channels of all types (surface and underground)
- drainage easement channels (surface and underground)

Multiplier Δ, should be applied to values obtained from the chart as per:

- grassed swales, well maintained and without driveway crossings, Δ = 4
- blade-cut earth table drains, well maintained and no driveway crossings, Δ = 2
- natural channels, Δ = 3

Once a trial flow rate has been determined, the travel time determined from Figure 4.8 will need to be checked using either figures 4.9 or 4.10.
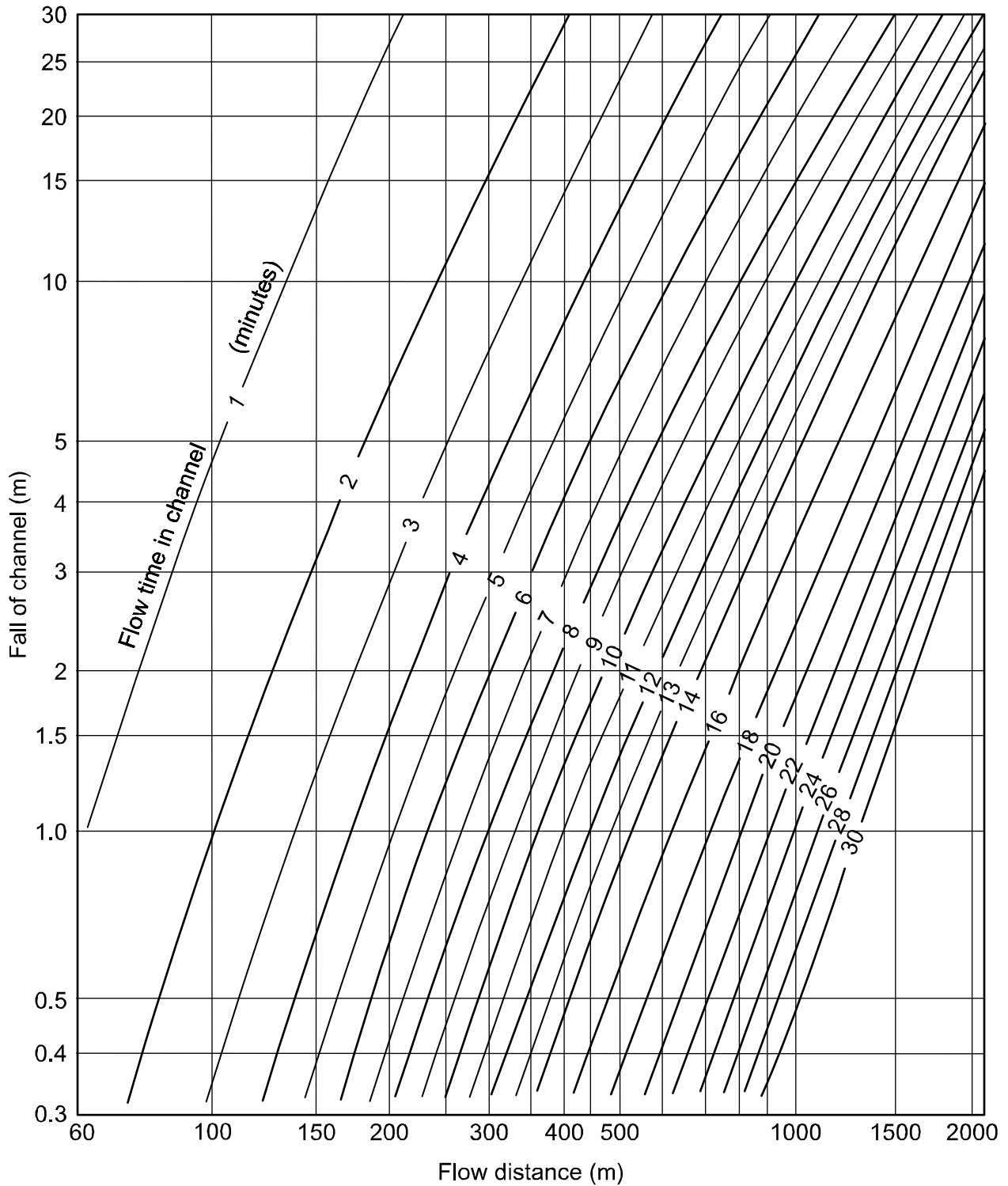
Figure 4.8 – Flow travel time in pipes and channels (Source: Argue, 1986)

4.6.8 Kerb flow

Time of flow in kerb and channel should be determined by dividing the length of kerb and channel flow by the average velocity of the flow.

The average velocity of the flow may be determined in either of two ways:

* Izzard's equation—refer to Technical Note 4, Book 8, ARR (1998). Reference is also made to section 7.4.2 (d) of this Manual for a more detailed explanation of Izzard's equation. Figure 4.10 provides a quick solution to Izzard's equation—accurate enough for travel time calculations.

* Using Figure 4.9.
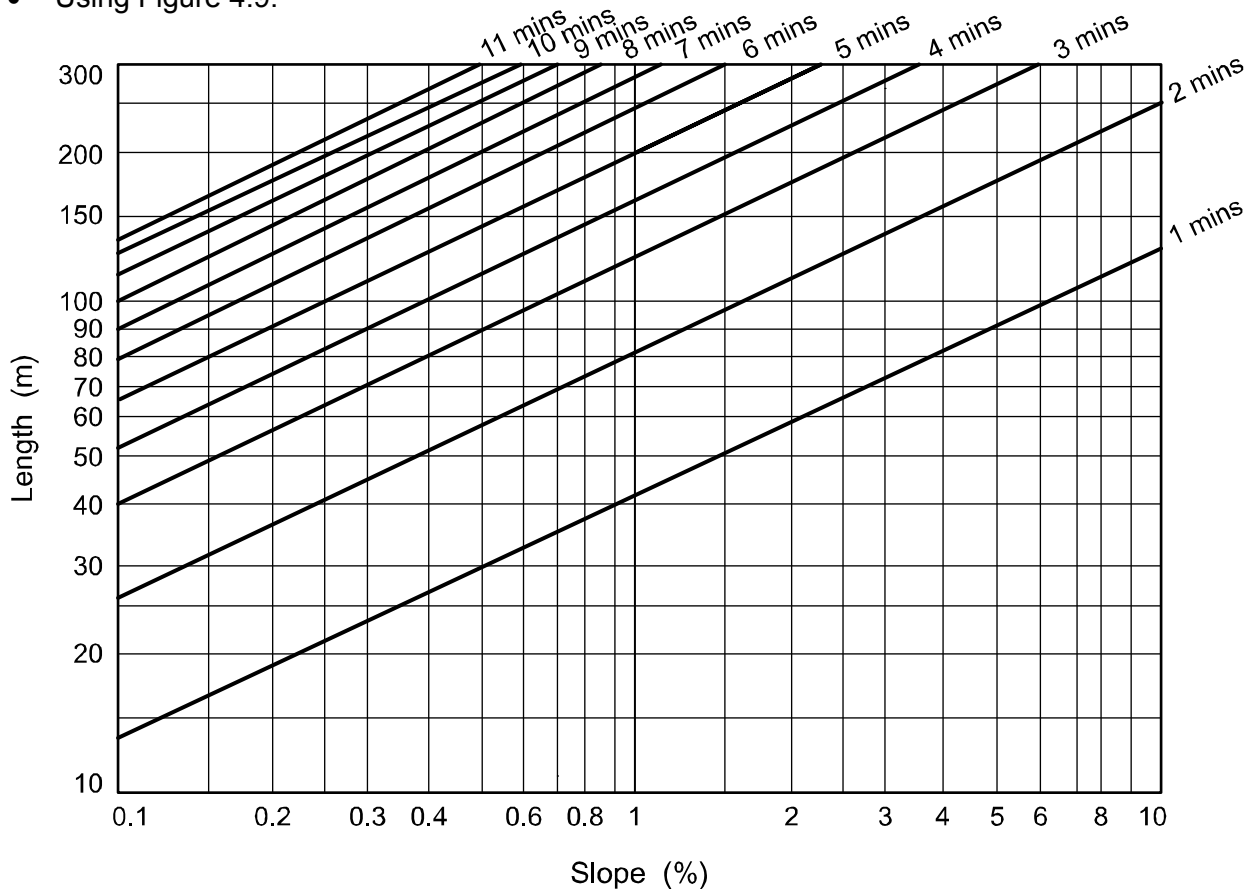
Figure 4.9 – Kerb and channel flow time using Manning's equation

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Technical notes for Figure 4.9

Formula: $t = 0.025\, L\, /\, S^{0.5}$ (minutes)

where:

t = time of gutter flow in minutes

L = length of gutter flow in metres

S = slope of gutter (%)

Example Length of gutter flow = 100m

Average slope of gutter = 3%

Thus, time of travel = 1.5 minutes.
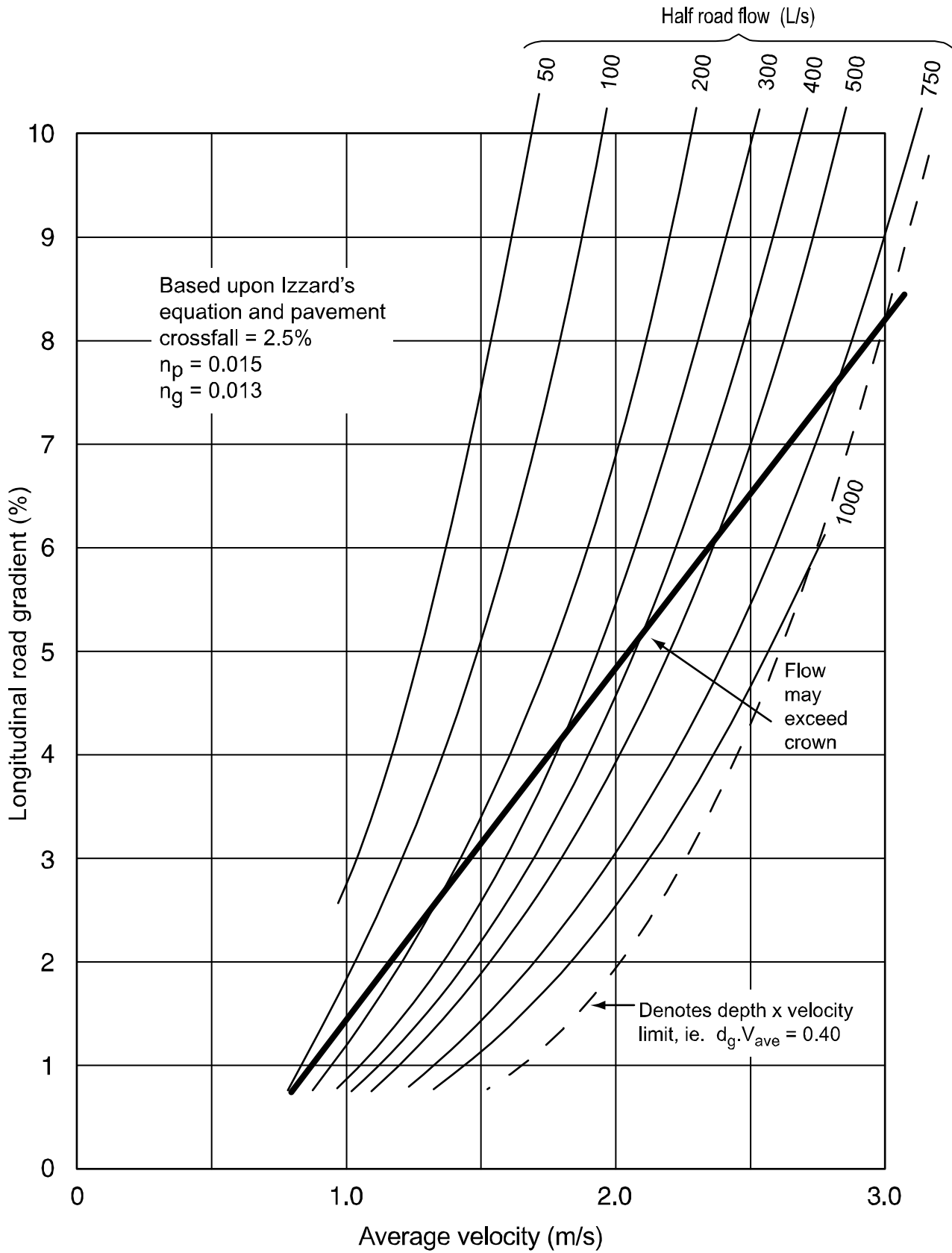
Figure 4.10 – Kerb and channel flow velocity using Izzard's equation

4.6.9   Pipe flow

Wherever practical, pipe travel times should be based on calculated pipe velocities either using a Pipe Flow Chart (e.g. n = 0.013 for concrete pipes), uniform flow calculations using Manning's equation (equation 4.7), or results from a calibrated numerical drainage model.

An initial (trial) assessment of the pipe flow travel time can be determined using Figure 4.8.

Alternatively, if the travel time within the pipe is small compared to the overall time of concentration, then an average pipe velocity of 2 m/s and 3 m/s may be adopted for low gradient and medium to steep gradient pipelines respectively.

4.6.10   Channel flow

The time stormwater takes to flow along an open channel may be determined by dividing the length of the channel by the average velocity of the flow. The average velocity of the flow is calculated using the hydraulic characteristics of the open channel.

Manning's equation is suitable for this purpose:

$$V = (1/n).R^{2/3}. S^{1/2} \tag{4.7}$$

From which
$$t = L/(60.V) = n . L / (60. R^{2/3}. S^{1/2}) \tag{4.8}$$

where:
- V = average velocity (m/s)
- n = Manning's roughness coefficient
- R = hydraulic radius (m)
- S = friction slope (m/m)
- L = length of reach (m)
- t = travel time (min)

Where an open channel has varying roughness or depth across its width it may be necessary to segment the flow and determine the average flow velocity, to determine the flow time.

Flow travel times along grassed swales can vary significantly depending on flow depth and swale roughness. The effective swale roughness should be determined from vegetation retardance charts (Department of Main Roads, 2002). For a grass length of 50 to 150 mm, typical Manning's roughness values may be interpolated from Table 9.3.4.

4.6.11   Time of concentration for rural catchments

Detailed information on the development of regional flood methods for ungauged rural catchments was not available at the time of release of this edition of QUDM (refer to section 4.1.5). In the near future it is expected that regional flood frequency analysis will supersede the application of the Rational Method for small to medium rural catchments.

In the application of the Rational Method in rural catchments, the time of concentration can be found by using either the Bransby-Williams' equation, modified Friend's equation or the stream velocity method. The local authority should be consulted for the acceptability of particular methods.

(a) Bransby-Williams' equation

$$t_c = 58 L / (A^{0.1} . S_e^{0.2})$$ (4.9)

where:

t_c	=	the time of concentration (min)
L	=	length (km) of flow path from catchment divide to outlet
A	=	catchment area (ha)
S_e	=	equal-area slope of stream flow path (%)

(b) Modified Friend's equation (maximum catchment area of 25 km²)

$$t_c = 800 L / (Ch . A^{0.1} . S_e^{0.4})$$ (4.10)

where:

t_c	=	time of concentration (min)
L	=	Length (km) of flow path from catchment divide to outlet
C_h	=	Chezy's coefficient at the site = $(1/n)R^{1/6}$
R	=	hydraulic radius = $0.75R_S$ where stream slope is fairly uniform
	=	$0.65R_S$ where stream slope varies appreciably along the stream
R_S	=	hydraulic radius at the initially assumed flood level at the site
n	=	average Manning roughness coefficient for the entire stream length
A	=	catchment area (ha)
Se	=	equal-area slope of stream flow path (%)

The calculation of hydraulic radius is based upon the peak level of the design flood at the site in question. If later hydraulic calculations show this level to be in error by more than 0.3–0.6 m, the value should be recalculated.
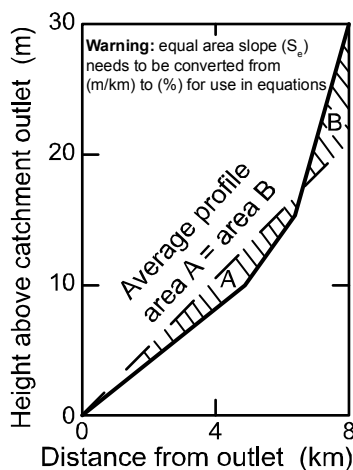
Figure 4.11 – Derivation of the equal-area slope (S_e) of main stream

Technical note 4.6.1 Use of Bransby-Williams and modified Friend's equations

Because an initial overland flow component is incorporated into the Bransby-Williams and modified Friend's equations, the addition of an overland flow travel time or standard inlet time is **not** required.

The equations as presented within this Manual are different from their original presentation within the 1992 edition of this Manual, as well as other publications such as ARR (1998). This is because the units have been changed such that both equations now utilise the same units for time, area and equal-area slope. The adopted coefficients within each equation have been appropriately adjusted (though rounded down from the exact unit conversion) for use of the revised units.

Also note: Figure 4.11 demonstrates equal-area slope in units of m/km which needs to be converted to percentage units by dividing by 10.

(c) Stream velocity method

As the catchment area increases, the relative influence of minor surface storage and in-stream channel storage on the peak discharge typically increases. To account for the flow-attenuating effects of channel/floodplain storage, the adopted or 'assumed' stream velocity needs to be less than the actual stream velocity, especially for low gradient streams where channel and floodplain storage is expected to be significant.

It is noted that for steep gradient or channelised streams with little or no floodplain storage the assumed stream velocity should be close to the expected actual stream velocity.

Table 4.6.6 – Assumed average stream velocities for rural catchment areas <500 ha [1]

Type of country	Average slope of catchment surface (%) [2]	Assumed stream velocity (m/s) [3]
Flat	0 to 1.5	0.3
Rolling	1.5 to 4	0.7
Hilly	4 to 8	0.9
Steep	8 to 15	1.5
Very steep rocky mountains	> 15	3.0

Notes (Table 4.6.6):

[1]   Source: Book 4, Australian Rainfall & Runoff (1998).

[2]   Catchment slope is not the same as stream slope or equal area slope.

[3]   These are assumed average stream velocities that need to be adopted in order to determine an appropriate time of concentration for use in the Rational Method.

4.7   The partial area effect

In general, the appropriate time of concentration (t_c) for calculation of the flow at any point is the longest time of travel to that point. However, in some situations, the maximum flow may occur when only part of the upstream catchment is contributing. Thus the product of a lesser $C.A$ and a higher ${}^t I_y$ (resulting from a lower t_c) may produce a greater peak discharge than that if the whole upstream catchment is considered. This is known as the 'partial area effect'.

Usually the above effect results from the existence of a sub-catchment of relatively small $C.A$ but a considerably longer than average t_c. This can result from differences within a catchment of surface slope, or from catchment shape. Typical cases include a playing field or open space within a residential area, or an elongated catchment. Figure 4.12 shows various examples.

It is important to note that particular sub-catchments may not produce partial area effects when considered individually, but when combined at some downstream point with other sub-catchments, the peak discharge may result when only parts of these sub-catchments are contributing.

The onus is on the designer to be aware of the possibility of the partial area effect and to check as necessary to ensure that an appropriate peak discharge is obtained.

There are two generally accepted Rational Method based procedures for the calculation of peak flow rates from partial areas as presented below; however, it is generally recommended that the hydrologic assessment of catchments with unusual or widely varying surface features should be undertaken with an appropriate numerical runoff-routing model.
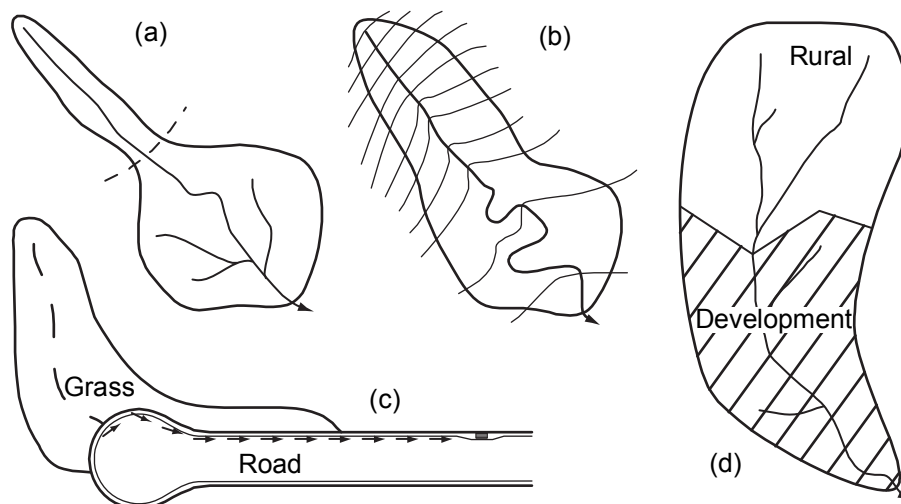
Figure 4.12 – Examples of catchments that may be subject to partial area effects

(a)   Simplified Procedure

A simplified procedure is given in section 4.5 of Argue (1986) based upon a comparison between the full area discharge and the partial area peak discharge for the time of concentration of the impervious areas of the critical sub-catchment. Care must be exercised as this procedure can underestimate the peak discharge.

The method involves the use of a time of concentration (t_i) corresponding to the flow travel time from the most remote, directly connected, impervious area of the catchment to the point under consideration. Thus, the calculated peak discharge is that from the impervious portion of the catchment plus that from the pervious part of the catchment which has begun to contribute up to time t_i since the storm began.

Thus,
$$C.A = C_i.A_i + \left[\frac{t_i}{t_c}.C_p.A_p\right]$$
(4.11)

Care must be used in applying this equation to catchments of irregular shape, and a case by case assessment is recommended.

(b)   Isochronal Method

This is a trial and error method that is applicable where it is possible to identify those sub-catchments likely to have long response times relative to the balance of the catchment.

Isochrones are lines drawn on a catchment plan passing through points which have equal travel times to the catchment outlet. Isochrones are drawn for the critical sub-catchments and these are used to assess the contributing area for a range of travel times from which the highest peak discharge is selected.

Depending upon the complexity of the sub-catchments it may be necessary to distinguish between pervious and impervious areas both in respect of travel time to the outlet and their effect upon the equivalent impervious area under consideration.

4.8 Intensity-frequency-duration data

Intensity-frequency-duration data is required as input to the hydrologic model used for design. There are a number of means by which this data can be obtained, including:

- Local authorities may issue IFD curves and/or tables and direct that these be used within specified regions within their local authority area.

- IFD data may be generated using the procedures given in ARR (1998) Book 2. Book 2 provides both algebraic and graphical procedures that allow the user to determine either complete or selected IFD design rainfall information for any location in Australia. The procedures enable the determination of rainfall intensities for durations of 5 minutes to 72 hours and ARIs from 1 year to 100 years. Book 2 also describes procedures for extrapolation to ARIs up to 500 years.

- IFD Curves for specific locations can be obtained from the Bureau of Meteorology. The Bureau will also provide tabulated data, a polynomial equation and coefficients for this equation. The equation can be used to generate a more detailed IFD table.

- Kennedy and Minty (1992) provides a procedure for estimating rainfall intensity data for durations shorter than the five-minute limit presented within the Bureau of Meteorology data.

Technical note 4.8.1 Consideration of climate change

State of Queensland 2010 report *Increasing Queensland's resilience to inland flooding in a changing climate* provides the following recommendations with respect to considerations of climate change:

- Local governments should factor a 5 per cent increase in rainfall intensity per degree of global warming into the 1 per cent (Q100), 0.5 per cent (Q200) and 0.2 per cent (Q500) AEP flood events recommended in SPP 1/03 for the location and design of new development.

- The following temperatures and timeframes should be used for the purposes of applying the climate change factor: 2°C by 2050, 3°C by 2070 and 4°C by 2100.

The report indicates that the Queensland Government will review and update the above climate change advice when a national position on how to factor climate change into flood studies is finalised as part of the current review of AR&R.

The algebraic and graphical procedures are presented in ARR (1998) Book 2 as a series of eight steps which guide the user to obtain a complete matrix of rainfall intensities for selected durations and ARIs.

The determination of design rainfall intensities using the above steps and the maps of ARR (1998) Volume 2 can be summarised as:

- Select the region of Queensland for the required location using the index to maps.

- Read the log-normal design rainfalls for the basic ARIs of 2 and 50 years and durations of 1, 12 and 72 hours for the required location from MAPs 1 to 6.

- Read the appropriate skewness from the regionalised skewness map (MAPS 7b and 7c).

- Read the short duration geographical factors F2 and F50 from MAPs 8 and 9 and calculate the 6 minute duration log-normal rainfall intensities for ARIs of 2 and 50 years.

- Convert the log-normal rainfalls from (ii) and (iv) to log-Pearson Type III distribution estimates using algebraic or graphical procedures.

- To determine rainfall intensities for other durations and ARIs, use algebraic or graphical interpolation and extrapolation techniques.

The ARR Volume 2 (1998) maps applicable to Queensland are presented in Table 4.8.1.

Table 4.8.1 – Design rainfall intensity-frequency-duration maps for Queensland

Map No.	Description	Region covered
1.1 to 6.1	Design rainfall isopleths	North coast
1.2 to 6.2	Design rainfall isopleths	North central coast
1.3 to 6.3	Design rainfall isopleths	Central coast
1.4 to 6.4	Design rainfall isopleths	South central coast
1.5 to 6.5	Design rainfall isopleths	South east
1.13 to 6.13	Design rainfall isopleths	West, north west and far north
1.14 to 6.14	Design Rainfall Isopleths	South west
7b	Regional map of average coefficient of skewness	North and west
7c	Regional map of average coefficient of skewness	South west and south east
8	Contours of F2 for determining 6 minute rainfall intensities from 60 minute intensities for a 39% AEP (2 year ARI).	Whole state
9	Contours of F50 for determining 6 minute rainfall intensities from 60 minute intensities for a 2% AEP (50 year ARI).	Whole state

Computer programs are available to generate IFD tables using the above input data from ARR.

4.9   Estimation of runoff value

4.9.1   General

In stormwater design, the estimation of runoff volume is often as important as the estimation of peak discharge. Runoff volume is used for a variety of purposes, including:

- sizing temporary and permanent sedimentation basins
- sizing stormwater detention/retention basins
- designing various urban stormwater treatment systems.

In some cases it will be necessary to determine the 'average annual runoff volume' from a drainage catchment, while in other cases it will be necessary to determine the runoff volume from just a 'single storm'. Table 4.9.1 summarises the typical applications of these two forms of volumetric runoff.

Table 4.9.1 – Application of runoff volume estimation to stormwater design

Design activity	Annual runoff or single storm	Application
Temporary construction site sediment basins	Single storm event	• Sizing temporary construction site (wet) basins • Performance analysis of a basin following an actual storm event
Permanent sedimentation basins	Either average annual or single storm event	• Use of a specific volumetric runoff in the design of permanent sedimentation basins depends on the adopted design procedure
Stormwater detention and retention basins	Single storm event	• Analysis of the hydraulic performance of a detention basin
Urban stormwater design	Average annual runoff volume	• Design of new land developments to minimise changes in runoff volume so that the risk of downstream creek erosion is minimised
	Single storm event	• Sizing a stormwater treatment device for a specified design storm

Estimating the volume of runoff from a single storm requires different procedures to those used to determine the average annual volumetric runoff coefficient.

4.9.2 Use of the volumetric runoff volume

The volumetric runoff coefficient (C_V) is defined as the ratio of the volume of stormwater runoff to the volume of rainfall that produced the runoff. The determination of this parameter is a necessary part of determining runoff volume.

It should be noted that the volumetric runoff coefficient is **not** the same as the Rational Method coefficient of discharge (C).

The **'average annual volumetric runoff coefficient'** may be further defined as: the ratio of the average annual volume of stormwater runoff released from a specific catchment, to the average annual volume of rainfall released onto that catchment.

For a **'single storm event'**, the volumetric runoff coefficient may be further defined as: the ratio of the volume of stormwater runoff resulting from a single storm, to the volume of rainfall released by that storm over the specified catchment area.

The volumetric runoff coefficient for a single storm event will almost certainly be different from the average annual volumetric runoff coefficient. Thus, if reference is made to a specific volumetric runoff coefficient value within a report or design guideline, then it is important to acknowledge whether the coefficient refers to a single storm event, or to an annual average.

4.9.3 Estimation of annual average runoff volume

The average annual runoff volume may be determined from continuous catchment modelling (preferred method), or through the use of a calibrated regional volumetric runoff coefficient.

The average annual volumetric runoff coefficient for a given catchment will depend on the following factors:

• soil permeability

- local hydrology
- percentage of directly connected impervious area
- percentage of indirectly connected impervious surface area
- degree of stormwater harvesting, including the use of rainwater tanks.

Local hydrology can also affect the volumetric runoff coefficient. In tropical regions, high intensity storms represent a greater percentage of total annual rainfall. This can increase the runoff coefficient relative to those value expected within temperate regions.

An estimation of the average annual volumetric runoff coefficient may be obtained using one of the following methods:

- analysis of long-term stream gauging and rainfall records (preferred option)
- continuous water balance modelling using a calibrated catchment yield model (second option)
- use of an annual average volumetric runoff coefficient from an adjacent catchment with similar soil, topographic and climatic conditions (third option).

Local governments are encouraged to establish low-flow gauging stations within their region to assist in the development of local data for model calibration.

Guidelines on continuous event modelling may be found in Chapter 14 of *Australia Runoff Quality* (ARQ, 2005) and post-1998 versions of Australian Rainfall & Runoff.

4.9.4 Estimation of runoff volume from a single storm

An estimation of runoff volume from a single (isolated) storm event may be obtained using one of the following methods:

- calibrated runoff–routing model (preferred method)
- use of the single storm event volumetric runoff coefficient (Table 4.9.2)
- direct extraction of estimated rainfall losses from a given rainfall hyetograph
- estimation of runoff volume based on the Rational Method peak discharge (for use only during the preliminary design phase).

It is noted that the actual runoff volume will be dependent on a number of variables including soil type, depth of soil, land slope, type and density of vegetation cover, and the degree soil moisture at the start of the storm event (i.e. the lasting effects of previous rainfall).

(a) Single event volumetric runoff coefficient

The volumetric runoff coefficient for a single storm event may be estimated using the U.S. Soil Conservation Service (1986) procedures. Volumetric runoff coefficients developed from these procedures are presented in Table 4.9.2.

When using the coefficients presented in Table 4.9.2 the following issues should be noted:

- The coefficients apply to the pervious surfaces only; therefore, an adjustment must be applied to determine a coefficient for urbanised catchments, as presented in equation 4.12.
- The coefficients where originally developed for relatively flat agricultural land; therefore, these coefficients are likely to under-estimate the runoff volume from steep catchments.

Table 4.9.2 – Typical single storm event volumetric runoff coefficients for various Soil Hydrologic Groups

Rainfall (mm)	Soil Hydrologic Group			
	Group A Sand	Group B Sandy loam	Group C Loamy clay	Group D Clay
10	0.02	0.10	0.09	0.20
20	0.02	0.14	0.27	0.43
30	0.08	0.24	0.42	0.56
40	0.16	0.34	0.52	0.63
50	0.22	0.42	0.58	0.69
60	0.28	0.48	0.63	0.74
70	0.33	0.53	0.67	0.77
80	0.36	0.57	0.70	0.79
90	0.41	0.60	0.73	0.81
100	0.45	0.63	0.75	0.83

Source: US Soil Conservation Service (1986)

Group A soils: soil with very high infiltration capacity. Usually consist of deep (> 1 m), well-drained sandy loams, sands or gravels.

Group B soils: soil with moderate to high infiltration capacity. Usually consist of moderately deep (>0.5 m), well-drained medium loamy texture sandy loams, loams or clay loam soils.

Group C soils: soil with a low to moderate infiltration capacity. Usually consist of moderately fine clay loams, or loamy clays, or more porous soils that are impeded by poor surface conditions, shallow depth or a low porosity subsoil horizon.

Group D soils: soil with a low porosity. Usually consists of fine-texture clays, soils with poor structure, surface-sealing (dispersive/sodic) soils, or expansive clays. Included in this group would be soils with a permanent high watertable.

Landcom (2004) provides typical infiltration rates for the various Soil Hydrological Groups (A, B, C, and D) as presented in Table 4.9.3.

Table 4.9.3 – Typical infiltrations rates for various Soil Hydrological Groups [1]

Soil Hydrological Group	Typical infiltration rate (mm/hr)		K_{sat} (mm/hr) [2]
	Saturated	Dry soil	
A	25	>250	>120
B	13	200	10–120
C	6	125	1–10
D	3	75	<1

Notes (Table 4.9.3):

[1]   Sourced from Landcom, 2004.

[2] K_{sat} = Saturated hydraulic conductivity

$$C_{V(composite)} = \frac{C_{V(pervious)} \cdot (A - A_{(imp.)}) + A_{(imp.)}}{A} \tag{4.12}$$

where:

$C_{V(composite)}$ = Composite volumetric runoff coefficient

$C_{V(pervious)}$ = Volumetric runoff coefficient for pervious surface (Table 4.9.2)

A = Total catchment area

$A_{(imp.)}$ = Area of directly connected impervious surface, plus a percentage of the indirectly connected impervious surface area (assume 50% unless otherwise directed)

If the coefficient is being determined for the design of a temporary construction site sediment basin established within a clayey or loamy soil catchment, then a volumetric runoff coefficient of 1.0 is recommended for all compacted soils and areas exposed to heavy construction traffic (unless otherwise directed within an recognised sediment basin design procedure). Otherwise, use values from Table 4.9.2, or adopt a value of 0.5 for pervious surfaces if the soil texture is not known.

(b)   Analysis of rainfall hyetograph

If adequate information is known about the effective loss rates (e.g. initial loss and continuing loss rate) for the catchment's pervious and impervious areas, then a single storm event volumetric runoff coefficient can be estimated directly from a given rainfall hyetograph. However, it should be noted that 'design storms' are not typical of a real storm—they are at best a representation of a possible design storm burst likely to be found within a real storm. Thus, extreme care must be taken in the selection of an appropriate initial loss value.

Unless otherwise nominated by the local authority, the adopted initial loss rate should reasonably reflect the vegetation density, groundcover/mulch density and soil porosity. Guidelines on the determination of storm losses are provided in Book 2 of ARR (1998).

(c)   Estimation of runoff volume using the Rational Method

A preliminary (**not** final design) estimation of the runoff volume may be determined directly from the calculated peak discharge for the nominal design storm using equation 4.13. This volume must be used with caution.

$$V_i = (4/3)\, t_c\, Q_i \tag{4.13}$$

where:

V_i = runoff volume for the nominated storm event (m^3)

t_c = time of concentration used to calculate Q_i (s)

Q_i = peak discharge for the runoff hydrograph (m^3/s)

4.10   Methods for assessing the effects of urbanization on hydrologic models

Generally, the effects of urbanisation on runoff from a [drainage] basin include higher volume, higher peak discharge and shorter time of concentration. These changes are associated with the increased imperviousness and more efficient drainage that are characteristics of constructed drainage systems, (Hoggan, 1989).

Whilst it is not possible to provide firm recommendations in this Manual in respect of the effects of urbanisation, it is suggested that designers consider the following alternatives as applicable, and where appropriate refer to the sources.

(a)   Unit hydrograph methods

After Rao, Delleur and Sarma (1972).

$$Catchment\ lag\ =\ \frac{1.275.A^{0.458}.D^{0.371}}{P^{0.267}.(1+U)^{1.662}}$$ (4.14)

where:

A = catchment area (km^2)
P = depth of rainfall excess (mm)
D = duration of rainfall excess (hr)
U = degree of urbanisation (fraction)

Catchment lag is defined as the average time required for all parts of a catchment to contribute to the discharge at the outlet and includes allowance for both catchment storage and channel (or transmission) storage. A catchment where no account is taken of catchment storage effects has a lag time (or catchment lag) of t_c/2. Including the effects of catchment storage gives a lag time of approximately 1.33 t_c.

(b)   Runoff routing methods – RAFTS

After Aitken (1975)

$$B\ =\ \frac{0.285.A^{0.52}.S^{-0.50}}{(1+U)^{1.97}}$$ (4.15)

where:

B = routing parameter for RAFTS Model
S = modified equal area slope (%) (equivalent m = 0.715)

(c)   Runoff routing models – RORB

(i) After Laurenson and Mein (1990)

For use with RORB Model

$$k_{ri}\ =\ \frac{F_i\ .\ L_i}{d_{av}}$$ (4.16)

where:

k_{ri} = relative delay time of storage i
L_i = reach length represented by storage i (km)
F_i = a factor depending upon the type of reach

For a natural channel reach, F_i = 1.0

For a lined or piped reach, $F_i = 1/(9S_c^{0.5})$

where:

S_c = slope of the channel reach (%)

(ii) After Brisbane City Council, (Carroll, 1990)

For use with RORB Model (m = 0.8)

$$k_c = \frac{1.2\, d_{av}}{(1 + U)^{2.0}}$$ (4.17)

where:

k_c = empirical coefficient

d_{av} = average distance of flow in the channel network of sub-area inflows (km)

(d)   Runoff routing models—WBNM

After Boyd, Bufill & Knee (1993), Boyd & Milevski (1996) and Boyd, Rigby & VanDrie (1999). Calculates separate hydrographs from pervious and impervious areas. Different rainfall losses are specified for the two surfaces, and the hydrographs are combined at the subarea outlet. Runoff from pervious areas uses the standard WBNM lag equation:

$$Pervious\ Lag = LagParam \cdot A_{per}^{0.57} \cdot Q^{-0.23}$$ (4.18)

where: *LagParam* is the lag parameter for natural catchments, based on recorded flood data, with a recommended value of 1.6.

Runoff from impervious areas uses a modified equation, based on recorded flood data from urban catchments:

$$Impervious\ Lag = ImpLagFactor \cdot LagParam \cdot A_{imp}^{0.25}$$ (4.19)

where: *ImpLagFactor* reduces the lag time for runoff from impervious surfaces, with a recommended value of 0.10.

The above equations apply to runoff from the pervious and impervious surfaces of the subarea. If the stream channel is itself modified, with increased flow velocities and hence reduced lag times, a reduced lag time can be applied to the watercourse:

$$Stream\ Channel\ Lag = StreamLagFactor \cdot LagParam \cdot A^{0.57} \cdot Q^{-0.23}$$ (4.20)

where: *StreamLagFactor* reduces the lag time in the stream channel, depending on the flow velocity. For example, if the channel remains in essentially natural condition, *StreamLagFactor* has a value of 1.0, whereas concrete lining—which may increase flow velocities 3 times—would have a *StreamLagFactor* of 0.33.

All three equations are built into the model, and the user only has to specify values of *LagParam*, *ImpLagFactor* and *StreamLagFactor*.

(e)   Other

Mein and Goyen (1988) provides a useful summary of the effects of urbanisation.

5.   Detention/retention systems

5.1   General

In the absence of adequate controls, urban development can increase both storm runoff volumes and peak discharge rates. Such increases can aggravate downstream flooding, initiate creek erosion and cause stress to in-stream ecosystems.

One of the main objectives of an urban drainage system is to limit property flooding to acceptable levels. The use of stormwater detention/retention systems is one means of achieving this objective; however, the preferred response is to minimise any potential changes in stormwater runoff as a result of urban development.

Another objective of the urban drainage system is to minimise the degradation of downstream environmental values. The main concerns here relate to the potential for increased creek erosion and the stressing of in-stream aquatic ecosystems. Stormwater detention/retention systems have the potential to both increase and decrease these threats depending on their design and location within the waterway catchment. It is for this reason that great care must be taken in their design and their interaction with the greater-catchment hydrology.

In the context of this chapter, detention/retention systems include traditional detention basins, on-site detention (OSD), extended detention systems and stormwater retention devices, all of which have the effect of reducing and delaying peak flow rates. A definition of each of these systems is contained within the Glossary (Chapter 13).

5.2   Planning issues

While helping to reduce many of the adverse impacts of urbanisation, detention and retention systems can also introduce problems that designers and regulators should be aware of.

Some of the potential problems that may be associated with the use of stormwater detention/retention systems are outlined in Table 5.2.1.

If a total catchment model is being used to investigate the design and operation of a stormwater detention/retention system for a single land development, then the following issues need to be given appropriate consideration:

- It is inappropriate to consider the impact of a single development in isolation from the cumulative effects of full catchment development.

- The cumulative effects of stormwater detention/retention should be determined by modelling the hydraulic conditions that would exist if all future land developments were conducted in accordance with the current Planning Scheme.

- Consideration also needs to be given to the likely impacts of the development that would occur under existing catchment conditions.

- The potential adverse impacts of waterway flooding needs to be considered over all reaches of a waterway where flood waters are likely to adversely affect either the 'value' or 'potential use' of the land.

Table 5.2.1 – Potential problems resulting from the use of detention/retention systems

Problem	Likely causes	Management options
Aggravation of coincident flood peaks This action can cause increases in flood levels within the lower reaches of a waterway even though all upstream developments have not increased peak discharges from their sites.	This is often associated with the existence of several basins within a drainage catchment, or basins located within the lower reaches of a waterway.	In some cases it may be desirable to avoid the use of detention basins within the lower third of a catchment—unless supported by full catchment modelling. If stormwater detention is required within the lower third of the catchment, then it may be necessary to utilise extended detention systems.
Increases in flood levels well downstream of several basins	The cause of this problem is in part related to the above issue, but also to the effects of increases in the volume of runoff from upstream developments.	This problem is best managed through the adoption of Water Sensitive Urban Design principles, specifically those measures that avoid increases in the volume of runoff from **'major'** storms.
Increased potential for creek erosion immediately downstream of basins	The initiation and extent of creek erosion is not related solely to flow velocity, but also to the frequency and duration of bankful flows. Thus, an increase in runoff volume can increase the potential for creek erosion even if stream velocities remain unchanged.	The best management option is to avoid changes to the velocity, volume, duration and frequency of near bankful flows within the watercourse.
Stress to aquatic ecosystems downstream of basins	These stresses can be caused by increases in both high and low flows, but are more commonly associated with increases in the frequency and duration of low flows (i.e. less than the 1 in 1 year storm flow).	This problem is best managed through the adoption of Water Sensitive Urban Design principles, specifically those measures that avoid increases in the volume of runoff from **'minor'** storms.
Damage to vegetation within basins and potential maintenance mowing problems	These problems can be caused by extended periods of basin inundation resulting from overlapping storms.	Basins in tropical regions may require the installation of enhanced subsoil drainage systems.
Potential salt intrusion of excavated or low-lying basins	Vegetation problems can be caused by the movement of groundwater salts. This problem is typically limited to the dryer temperate climates.	This is best managed through appropriate soil surveys and groundwater studies, and the construction of shallow basins.
Safety risks	Safety risks can be associated with inadequate basin egress, excessive water depth, and hydraulic pressures associated with the outlet structure.	Basins should be designed to allow egress in all directions, and barriers should be placed in front of outlet systems to prevent close contact by humans.

Many of the above problems can be avoided through detailed catchment planning. Preference should always be given to the use of total catchment modelling to determine the preferred location and operational requirements of stormwater detention/retention systems. Such modelling would usually be carried out in association with a Flood Study, Stormwater Management Plan, or Master Drainage Study.

The types of stormwater detention/retention applied throughout a drainage catchment can vary significantly depending on:

- where the development is located within the catchment
- the locations of current flood risks within the catchment
- the current or potential environmental values of the waterways located downstream of the development
- the type of receiving waters into which the catchment drains.

Table 5.2.2 provides a guide to the placement of different types of detention/retention systems throughout a drainage catchment.

Table 5.2.2 – General guide to the placement of detention systems within a catchment

Location within catchment	Downstream waterway conditions	Type of basin
Lower third of catchment	Channelised watercourse with little ecological value, or developments that discharge into a river or large water body	It may be desirable to avoid the use of stormwater detention if no flood risks exist downstream.
	Minor watercourse (e.g. creek) with existing or potential ecological values	Utilisation of retention systems wherever practicable Use of extended detention systems if retention systems fail to achieve the required flood protection
Central catchment area	All cases	Utilisation of retention systems wherever practicable to minimise flood risks and adverse impacts on aquatic ecosystems
Upper third of catchment	Flood risks exist only within the lower reaches of the catchment.	Utilisation of retention systems wherever practicable The critical design storm is usually related to the critical storm duration for the downstream flood risk area.
	Flood risks exist at various locations downstream of the development.	Utilisation of retention systems wherever practicable The critical storm durations are likely to range from 1 to 3 hours (inclusive).

5.3 Functions of detention/retention systems

Stormwater detention and retention systems perform a variety of functions depending on their design. A short description of these functions is provided below, with a summary provided in Table 5.3.1.

5.3.1 Detention and retention systems

(a) Discharge control

On-site detention and regional detention systems may be designed to restrict peak outflows for selected design storms to either pre-development conditions, or to the maximum capacity of the existing downstream drainage network.

Outflow restrictions linked to the downstream drainage may relate to the hydraulic capacity of the drainage systems, or to safety issues associated with an overland flow path.

(b) Flood control

Both detention and retention systems can be used to alleviate flooding concerns resulting from past development activities, or from changing community attitudes to what is considered an acceptable flood risk.

Traditional detention systems delay stormwater runoff for a few hours, or fractions of an hour, while *extended detention systems* can be used to store and discharge part of the total runoff over a period of 1 to 2 days. Extended detention systems can be effective for the management of new developments located within the lower half of a catchment where traditional detention basins may aggravate downstream flooding due to the effects of coincident flood wave peaks.

(c) Erosion control

The operation of stormwater detention systems within a catchment can have both positive and negative impacts on downstream channel erosion. It should be noted that channel erosion within vegetated waterways is not solely governed by the peak discharge of major floods. Instead, it is the frequency and duration of near-bankful flows that primarily governs channel erosion within these waterways.

In general, a development that causes an increase in the peak flood discharge without causing a significant increase in the volume of runoff is likely to cause fewer adverse impacts on a downstream watercourse than a development that increases the volume of runoff without increasing peak discharge.

An increase in the volume of runoff is likely to cause a significant increase in the **duration** of near-bankful flows, while an increase in the impervious surface area (especially directly connected impervious surface areas) is likely to cause an increase in the frequency of stream flows.

Unlike some 'retention' systems, 'detention' systems generally cannot be used to compensate for changes in runoff volume. Thus, in circumstances where urbanisation has increased the volume of runoff, the use of stormwater detention systems may contribute to an increase in the potential for downstream creek erosion.

Therefore, in most circumstances, detention systems need to operate in coordination with appropriate runoff-reducing measures (i.e. WSUD) if the risks of increasing downstream channel erosion are to be minimised.

(d) Pollution control

Most detention basins provide little if any measurable water quality benefit, especially if an impervious low-flow drainage system is constructed through or below the open basin. Permanent sedimentation basins, however, can provide both stormwater detention and stormwater quality treatment (i.e. settlement of sediment and particulates).

Extended detention systems can provide water quality benefits through extended sedimentation and solar treatment. In some circumstances, filter basins and sand filters can be designed to operate as extended detention systems, thus providing both stormwater detention and stormwater treatment benefits.

Most retention systems incorporate stormwater quality treatment measures, such as a pond or wetland, or they may actually be the treatment measure, such as an infiltration trench or basin.

5.3.2 Retention systems

(a) Rainwater harvesting

Household rainwater tanks effectively operate as stormwater retention systems. In some cases the tanks may consist of two zones, one zone for stormwater detention (which freely drains after each storm), and one zone for rainwater harvesting. This latter case is generally not desirable as there is the risk that such systems will be modified to maximise rainwater harvesting at the expense of stormwater detention.

Under certain geological conditions, stormwater captured in retention basins may be injected into underground aquifers as a water storage measure. Argue (2004) provides guidelines on such practices.

The use of retention systems for stormwater harvesting and the design of rainwater tanks will not be discussed within this chapter. Designers should refer to the relevant local government guidelines.

(b) Control of runoff volume

Stormwater retention systems can be designed to reduce the 'total annual runoff volume', and/or reduce the runoff volume from a specified design storm. Reducing the total annual runoff volume provides water quality benefits, especially in circumstances where the stormwater ultimately flows to a large, semi-confined water body such as a lake, river, estuary or bay. On the other hand, reducing the runoff volume from a specific storm event can be beneficial for the control of erosion and flooding in minor watercourses such as creeks.

One of the main benefits of controlling runoff volume is the protection of aquatic ecosystems and habitats. Research (refer to ARQ 2005) has shown a strong correlation between the percentage impervious surface area of urban developments and the depletion of aquatic ecosystem health. It is suggested that this correlation is primarily linked to the effects of increases in stormwater runoff from minor storms. The adverse effects of introduced impervious surface area can be reduced by increasing the percentage of *indirectly-connected* impervious surface areas, and retaining runoff

from impervious areas to a depth equivalent to the soil's 'initial loss' depth (usually around 15 to 25 mm).

Stormwater retained within these systems may be made available for secondary (non-potable) purposes through a stormwater harvesting system, or removed from the surface drainage system through infiltration and/or evaporation.

5.3.3   Summary of functions

A summary of the possible functions of detention and retention systems is provided in Table 5.3.1.

Table 5.3.1 – Summary of detention/retention system functions

		Discharge control	Flood control	Volume control	Scour control	Stormwater harvesting	Pollution control
Detention systems	**On-site detention**	Yes	Yes				
	Detention basins	Yes	Yes		[1]		[1]
	Extended detention basins [2]	Yes	Yes		[1]		Yes
	Filter basins	[1]	[1]				Yes
Retention systems	**Rainwater tanks**	[3]		[4]		Yes	
	Retention basins	Yes	Yes	Yes	[1]	Yes	Yes
	Infiltration trenches	Yes	Yes	Yes	[1]		Yes
	Infiltration basins	Yes	Yes	Yes	[1]	[1]	Yes

Notes (Table 5.3.1):

[1]   Not the normal function of this type of system, however, this function may be achieved if modifications are made to the design.

[2]   The most commonly used terminology is extended detention basin, however, the concept of extended detention may also apply to the design of retention basins.

[3]   Generally rainwater tanks cannot be used for on-site discharge control.

[4]   When wide spread across a catchment, rainwater tanks can contribute to runoff volume control through activities such as water reuse, garden watering and groundwater infiltration.

5.4   Design standards

5.4.1   General

Design standards depend on the required functions of the detention/retention system. **If the detention/retention system is required to satisfy more than one function, e.g. flood control and the control of creek erosion, then appropriate consideration must be given to achieving all design requirements.**

In all cases, detention/retention systems must not cause unacceptable increases in flood levels upstream or downstream of the system. An unacceptable increase in flooding would include any change in flood characteristics on surrounding properties that could cause damage to, or adversely affect either the 'value' or 'potential use' of the land, or cause problems resulting from changes in flow velocity or the distribution of flow velocity within that land.

5.4.2   On-site detention systems

There are generally three design standards set by regulating authorities, they are:

- A specified minimum site storage requirement (SSR) and permissible site discharge (PSD) relative to either the site area, land use, or the change in impervious area.

- A permissible site discharge for the specified design storm frequency with no minimum storage volume specified.

- A requirement not to exceed pre-development peak discharge rates for a range of design storm frequencies.

The first two design criteria are often adopted by local governments following the development of a regional flood control strategy, Master Drainage Plan, or Stormwater Management Plan.

Most small on-site detention systems incorporate underground tanks. When appropriate soil and groundwater conditions exist, some underground tanks can be converted into infiltration systems. Above-ground stormwater detention tanks are rarely used on single residential properties because of the risk of the tanks being converted solely to rainwater tanks.

5.4.3   Flood control systems

Traditionally detention basins have been designed to ensure no increase in post-development peak discharge immediately downstream of the basin for specified storm events such as 1, 2, 10, 18, 39 and 63% AEPs (100, 50, 10, 2 and 1 year ARIs). Satisfying this criterion however will not necessarily guarantee that there will be no adverse impacts on flood levels well downstream of the development. The full impacts of a stormwater detention system can only be assessed by modelling the full catchment, including all flood prone areas downstream of the detention system.

An increase in downstream flooding may occur for one or more of the following reasons:

- Changes in the speed of the flood wave passing down the catchment and the resulting risk of coincident flooding.

- Changes in the volume of stormwater runoff from new land developments and the impact this has on the shape of the basin's discharge hydrograph.

An increase in runoff volume is an inevitable result of traditional urban development. Thus, discharge rates within the rising and/or falling limb of a detention basin outflow hydrograph may be significantly higher than the corresponding pre-development discharge rates. If several detention basins are located within a given catchment, then these increased discharge hydrographs may overlap causing an increase in flood flows and flood levels downstream of the basins.

One of the benefits of adopting Water Sensitive Urban Design (WSUD) is that it reduces the potential for increases in runoff volume, thus reducing the potential for increases in downstream flows and flooding.

(a)   Greenfield and infill developments

In cases where the design requirements of detention/retention systems have **not** been determined from an appropriate total catchment study, the recommended sizing of such a flood control system shall be based on achieving the following minimum requirements:

- No increase in flood levels on land adjoining the basin and/or the development where such an increase would cause damage to, or adversely affect, either the value or potential use of the land.

- No increase in peak discharges immediately downstream of the development for a selected range of storm durations, for a selected range of AEPs up to the Defined Flood Event.

Technical note 5.4.1

The second dot point (above) indicates that the peak discharge for each of the selected storm durations shall not increase even if that storm duration does not produce the highest peak discharge for the given AEP.

It is recommended that the selected storm durations tested should include the 1-hour storm, 3-hour storm, and a storm of duration at least three times the critical storm duration of the detention or retention basin.

Exceptions to this rule may be considered by the local government only if a storm of a given duration does not inundate floor levels or adversely affect the potential use of land upstream or downstream of the basin.

In cases where the design requirements for detention/retention systems **have** been determined from an appropriate total catchment study, the recommended modelling of such flood control systems shall be based on achieving the following minimum requirements:

- No increase in flood levels on land adjoining the basin and/or the development where such an increase would cause damage to, or adversely affect, either the value or potential use of the land.

- No increase in peak flood level and/or discharge at any location downstream of any basin where existing land owners/users may be adversely affected by such an increase. This requirement shall apply to a full range of storm durations and frequencies up to the Defined Flood Event where such storms result in flooding that either inundates floor levels or adversely affects the potential use of the land (refer to technical note 5.4.1).

(b)   Control of existing flooding problems

Flood control detention/retention basins constructed to alleviate existing flooding problems should be designed to achieve one or more of the following outcomes:

- Maximum flood attenuation benefits from the available land area (i.e. where storage volume is limited by site constraints). This option usually requires the basin's low-flow outlet to be sized to make maximum use of the safe hydraulic capacity of the downstream drainage system. The local authority should be consulted when determining the maximum allowable discharge rate into the downstream drainage system.

- All requirements listed above in (a) for greenfield developments.

5.4.4   Control of accelerated channel erosion

If one of the primary objectives of a stormwater system is to minimise the risk of accelerated channel erosion, then consideration must be given to those measures that will minimise changes to:

- the frequency and duration of near-bankful flows

- the peak discharge of stream flows greater than or equal to the bankful flow rate.

This may be achieved through various combinations of the following:

- adopting the principles of Water Sensitive Urban Design

- minimising changes in impervious surface area, particularly on highly porous soils

- decreasing the percentage of directly connected impervious surfaces

- maximising stormwater infiltration
- using rainwater harvesting to minimise changes to runoff volume
- utilising stormwater retention rather than detention systems.

In this context, the primary aim is **not** to reduce changes in the 'annual runoff volume', but to reduce changes in the runoff volume of those storms that are likely to contribute to near-bankful flows. Thus the focus is likely to be on storms with an AEP between 1 in 2 years and 1 in 10 years.

It is noted that this requirement is different from that used in the management of stormwater quality and the protection of in-stream ecology where the primary aim is to reduce changes in the annual runoff volume and the total water cycle.

It should also be noted that an increase in the frequency and duration of low flows within a waterway (i.e. flows less than the 1 in 2 year AEP) may increase the stress on in-stream aquatic ecology and habitats. Thus the only way to minimise the risk of both accelerated channel erosion, and a decline in aquatic habitats, is to minimise changes to the natural water cycle, including the frequency, duration, velocity, volume, and peak discharge of all runoff events.

5.5 Flood-routing

5.5.1 Basin sizing

The final sizing of the basin should be completed with the aid of a computer model. The selected model must accurately simulate the hydraulic behaviour of the basin outlet, especially when partial full pipe flow or tailwater submergence occurs.

To account for the effects of urbanisation upon the flood hydrograph the procedures contained in section 4.10 are recommended.

Technical note 5.5.1

As an alternative to the use of a computer model, the final sizing can be undertaken by manual flow routing based on a direct solution of the storage equation:

$$(I_1 + I_2) + (2S_1/T - Q_1) = (2S_2/T + Q_2) \qquad (5.1)$$

where:

I	=	the inflow rate
S	=	the volume in storage
Q	=	the outflow rate
T	=	the routing time step
$1, 2$		denote the start, finish of the routing step

Equation 5.1 requires the shape of the inflow hydrograph to be determined. Full details of the procedure are given in Book 5, ARR (1998).

Whichever technique is used for final basin sizing, the routing time-step or increment must be short enough relative to the storm duration to ensure that the peak storage requirements will be accurately determined.

The design of the basin and its outlet structures must be based on a range of storm durations and appropriate temporal patterns in order to identify the critical hydraulic dimensions. If the basin is required to prevent an increase in flooding at a given location, then the performance of the basin needs be checked for a storm of duration equal to the critical storm duration at this location. If the basin is required to prevent an increase in flooding at **all** locations downstream of the basin, then

the performance of the basin needs be checked for a range of storm durations up to the critical storm duration of the most downstream location.

*Note: It is **not** sufficient to simply determine which storm duration produces the largest peak discharge from the basin. Even though a storm of greater duration than the basin's critical storm duration produces a lower peak discharge, it may require a greater detention volume to prevent an increase in the peak discharge of such a storm.*

5.5.2 Temporal patterns

The design of the low-level outlet can normally be based on the average temporal patterns given in the latest version of ARR. Design of the high-level outlet and the embankment crest height should account for the fact that the temporal patterns given in ARR are only the averages of the many storm bursts that can actually occur. It should also be noted that these temporal patterns do not represent full storms, but just the worst burst within a longer storm.

Designers should confirm with the relevant regulating authority the types of temporal patterns to be used. It is recommended that the response of the basin should also be checked using real storms, even if such storms have an AEP significantly different from the design storm.

If data from a real storm with an AEP similar to the specified design storm is not available, then the size of the basin should be checked using the following three alternative temporal patterns, in addition to the average pattern:

- A pattern in which the peak intensity is located midway between the start of the storm and the peak of the average pattern.

- A pattern in which the peak intensity is located midway between the end of the storm and the peak of the average pattern.

- A pattern recorded during a major storm at a rainfall gauging station near the site, if available.
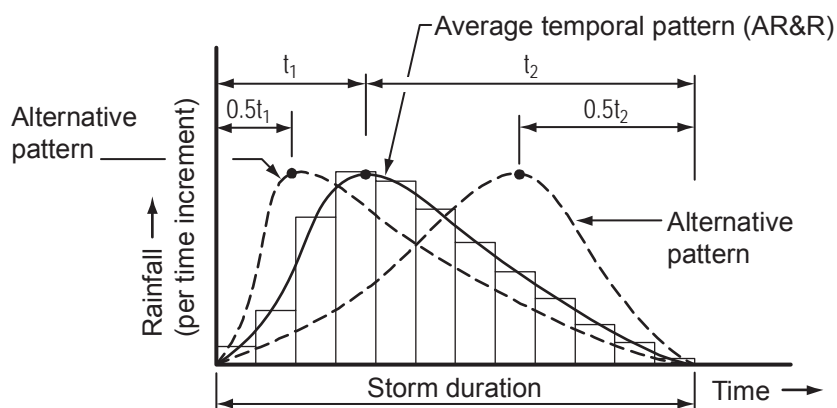
Figure 5.1 – Additional temporal patterns for use in design of embankments and high-level outlets

5.5.3 Allowance for existing channel storage

When a hydrologic analysis is performed on a detention/retention basin located within a waterway, it is important to ensure that:

- The flood mitigation effects of the existing channel storage are not duplicated within both the channel routing component of the model (i.e. routing from node to node) and the detention storage routing (i.e. flood routing through a basin at the downstream node).

- Appropriate consideration is given to the potential effects of lead-up rainfall prior to the storm burst as normally occurs in real storms.

The first issue (above) may be addressed by reducing the modelled basin storage by the measured natural channel storage. Alternatively, a new node may be inserted at the upstream influence of the basin (i.e. limit of the basin's backwater effects), with the flood routing coefficient adjusted so that there is no flood attenuation between the upstream and downstream basin nodes (i.e. for Muskingum routing, x = 0.5).

The second issue (above) may be addressed by modelling the basin using real storm data to assess likely storage levels prior to a storm burst.

5.6 Basin freeboard

Recommendations on the selection of freeboard are provided in Table 5.6.1.

Table 5.6.1 – Guidelines for basin freeboard requirements

Situation	ARI (years)	Maximum depth or level
Basin formed by road embankment (a)	20	Bottom of pavement box
(b)	50	0.3 m below edge of shoulder
Basin formed by railway embankment	50	Underside of ballast
Large basins with separate high level spillway	100	Embankment crest with freeboard ≥ 10% of the 1% AEP storage depth and with minimum freeboard = 0.3 m [1]

Note (Table 5.6.1):

[1]   Freeboard must fully contain the potential wave height if the resulting overtopping is likely to represent a safety risk to the embankment or undesirable erosion. U.S. Army Corps of Engineers (1984) provides guidelines on the estimation of wave height.

5.7 Basin floor drainage

Design of the low-flow drainage system through the basin will depend on numerous factors including the required dry-weather function of the basin, the need for water quality treatment of the low flows, and safety and maintenance issues. General guidelines for the design of low-flow channels are provided in section 9.8 of this Manual.

The design of the basin floor should take appropriate consideration of the following recommendations:

- Minimum cross gradient of 1 in 80 for grassed basins to allow efficient surface drainage (this is based on the recommended minimum cross fall for school ovals).

- Minimum cross gradient of 1 in 100 for vegetated basins (i.e. deep-rooted plants such as trees and shrubs). Minimum cross gradient does not apply if the basin floor is a natural drainage surface.

- Minimum invert level for mowable grassed areas at least 300 to 500 mm above the invert of an adjacent stream (i.e. on-line basin). The range 300 to 500 mm depends on the soil drainage properties and the degree of sedimentation likely to occur within the stream channel.

If field inlets are used to help drain the basin floor, then adequate scour protection needs to be placed around the inlet as discussed in section 7.5.4(c).

It is noted that safety issues may require an inlet screen of sufficient size to limit flow velocities through the screen to a maximum of 1 m/s. Minimum dimensions of dome inlet safety screens are presented in section 12.5.8.

5.8 Low-level basin outlet structures

5.8.1 Types of basin outlets

Low-level outlet structures generally consist of orifice plates, pipes or culverts placed at a low level in the basin to cater for the discharge of normal outflows.

Recommendations for the design of outlet structures are given by the American Society of Civil Engineers (ASCE, 1985). Hydraulic relationships for various outlet structures are provided in the User Manuals for software packages such as DRAINS, RAFTS and RORB.

The storage-discharge curve used in the flood-routing analysis must accurately reflect expected hydraulic conditions including allowances for part-full pipe flow, inlet/outlet control where appropriate, partial blockages and the effects of external catchments on the hydraulic grade line.

Low-level outlet structures for small basins (Figure 5.2) will generally consist of a single orifice or pipe. In some cases a pump will be installed with capacity designed to match the outflow limitation only at the AEP at which the high-level outlet just begins to operate. Where a pump is allowed by the local government, a stand by power supply may be required.

Figure 5.2 – Typical basin outlets for small basins

Low-level outlet structures for **large** detention basins will more often be required to limit the outflows over a range of intermediate AEPs down to the AEP for the Design Flood. In such cases, the low-level outlet structure may comprise either a single-level outlet sometimes preceded by a weir, or a multi-level outlet.

A weir located immediately upstream of a single-level outlet may have an orifice of smaller diameter than the main basin outlet to attenuate the outflows from smaller storms and to aid the free draining of the basin. During higher inflows the weir will overtop. A multi-level outlet will have a range of pipes or culverts set at different levels, possibly of different sizes to achieve the required attenuation throughout the AEP range.

If the basin outlet is directly connected to a downstream piped drainage network, then this system should be checked for undesirable surcharge. A full HGL analysis may be required by the regulating authority.

5.8.2   Protection of basin outlet

The intake to a detention basin outlet should be protected against expected debris blockages and designed to minimise the safety risk to a person trapped against the outlet structure. The level of protection will vary depending on the consequences of failure caused by blockage of the intake and the potential frequency of blockage.

Consideration should also be given to the consequences of a fully blocked low-level outlet.

Protection can be achieved by the installation of a trash rack, bar screen and/or a fence. These should be designed to shed debris and to assist egress by persons trapped in the basin generally in accordance with the recommendations of Weisman (1989) and section 12.5 of this Manual. Trash racks comprising inclined vertical bars (inclined in the direction of flow) and spaced horizontal support bars are preferred.

Design criteria for intake structures are given in Table 5.8.1.

Table 5.8.1 – Criteria for basin outlet structures

Item	Criterion
Spacing of vertical bars	125 mm (max)
Inclined spacing of horizontal supports	600 mm (max) [1]
Net clear opening area	≥ 3 times the calculated outlet area [2]
Limiting velocity through trash rack [3]	0.6 m/s (not readily accessible) 1.5 m/s (accessible)

Notes (Table 5.8.1):

[1]   The maximum (inclined) spacing of horizontal supports aims to allow a trapped person to climb up the screen to safety.

[2]   The calculated outlet area may depend upon the level of the outlet relative to the water surface. Where the outlet is contained in a drop structure, the outlet area used for determination of the net clear opening for the intake may need to be adjusted to account for the level difference.

[3]   The limiting velocity through the trash rack should be related to the accessibility of the intake structure for cleaning purposes.

Detailed procedures for determining the hydraulic losses through trash racks are given in Chow (1959) and U.S. Bureau of Reclamation (1987) otherwise refer to section 12.5.6 of this Manual.

5.8.3   Pipe protection

Outlet pipes should have spigot and socket rubber-ring joints and lifting holes should be securely sealed. Pipe and culvert bedding should be carefully specified to minimise its permeability. Cut-off walls or seepage collars must be installed where appropriate, to control seepage and prevent piping failure adjacent to the outlet pipe.

Appropriate measures, such as internal sealing of pipe joints and lifting holes, and bolting down of access chamber lids, should be applied to any existing downstream systems which could be pressurised by the discharge from the outlet. Alternatively, surcharge chambers may need to be incorporated into the outlet pipe to limit the internal pressure.

5.8.4 Outfall protection

Where the outlet from a basin is to a free outfall, this should be located, where possible, within a well-defined natural depression or watercourse. The outlet should also be located a suitable distance upstream of the downstream property boundary to ensure that the downstream properties will not be adversely affected by the velocity or the concentration of the outflow.

Adequate protection must be provided both downstream and immediately upstream of the outlet, where appropriate, to prevent scour.

5.9 High-level outlet structures

5.9.1 Extreme flood event

The designer may select the storage level at which the high-level outlet will begin to discharge; however, care must be taken to ensure that flooding of upstream properties is not worsened.

The spillway and embankment should be designed both hydraulically and structurally to permit the safe discharge of floods in excess of the Design Flood. The AEP of the Extreme Flood for which the performance of the basin should be checked, needs to be determined with appropriate consideration of the likely consequences of failure, and in consultation with the local government. DEWS (2013) provides a basis for determining the AEP of the Extreme Flood based upon consideration of the incremental hazard associated with failure. Designers should refer to DEWS (2013) and DERM (2010a). Reference may also be made to ANCOLD (2000a) and (2000b).

Table 5.9.1 shows the range of AEPs applicable.

Table 5.9.1 – Recommendations for extreme flood [1]

Incremental flood hazard category [2]	Extreme flood AEP (%)
Extreme	PMF [3,4]
High A	PMP Design Flood [3,5]
High B	10,000 to PMP Design Flood or 1,000,000
High C	10,000 to PMP Design Flood or 100,000
Significant	1,000 to 10,000
Low to very Low	1% to 0.1%

Notes:

[1]   Sourced from ANCOLD (2000a)

[2]   Refer to Table 5.9.2.

[3]   Pre-flood reservoir level to be taken as the maximum normal operating level of the reservoir.

[4]   PMF refers to Probable Maximum Flood

[5]   PMP Design Flood refers to flood hydrograph generated by the Probable Maximum Precipitation

Table 5.9.2 – Hazard categories for referable dams [1]

Incremental population at risk	Severity of damage and loss			
	Negligible	**Minor**	**Medium**	**Major**
2 to 10	Low [2]	Significant [6]	Significant [6]	High C [7]
11 to 100		Significant [3,6]	High C [7]	High B [7]
101 to 1000	[2]	[3]	High A [7]	High A [7]
> 1000			[4]	Extreme [7]

Notes:

[1]   Sourced from DEWS (2013) (modified from ANCOLD, 2000b.)

[2]   It is unlikely that the severity of damage and loss will be "Negligible where one or more houses are damaged.

[3]   Minor damage and loss would be unlikely when PAR exceeds 10.

[4]   Medium damage and loss would be unlikely when the PAR exceeds 1000.

[5]   Not used.

[6]   Change to High C where there is the potential for one or more lives being lost.

[7]   Refer to ANCOLD (2000b) – sections 2.7 and 1.6 for explanation of the range of High Hazard Categories.

5.9.2   Spillway design

The high-level outlet, usually formed by a spillway, must be designed to safely convey extreme outflows from the basin. The design flow should consider the potential for full or partial blockage of any outlet structures. Wherever practical, design of the spillway should assume full blockage of the low-flow outlet.

Where possible, the spillway should be cut into virgin ground at the side of the embankment, or otherwise located to minimise the possibility of embankment failure.

In some circumstances the high-level outlet may be constructed as a glory-hole inlet (with bar screen and anti-vortex device as required) leading to a pipe or a culvert through the embankment.

The spillway chute may be protected by riprap, concrete, paving, or other suitable coverings. A grass or reinforced grass cover may be adequate where spillway slopes are flatter than 1 on 6 (1V:6H). Care should be taken to maintain a healthy, continuous grass cover on grass spillways. Trees, shrubs, watering tap outlets, or any other fixed structure that may cause turbulence or eddy-induced erosion must not be located within a grassed spillway chute. Design information for grassed spillways is described by the U.S. Soil Conservation Service (1979).

5.10   Embankments

Detention basins are intermittent water-retaining storages for which the embankments do not need to be as rigorously designed as dams unless they are particularly high or have special soil problems. Retention basins are designed to have a permanent or semi-permanent water storage component—these structures need particular design measures if the retention depth is significant.

The embankment design of all detention and retention basins should be undertaken, or at least reviewed by a suitably experienced Geotechnical specialist.

The sides of grassed embankments, including any inner basin grassed slopes, should generally be flatter than 1 on 6 and never steeper than 1 on 4 for reasons of mower access. The top-width should be at least three (3) metres. Steeper slopes may be used on embankments or basins lined with structural facings or low-maintenance ground covers, but steps must be provided at appropriate intervals if the steepness of the slope could impede the egress of a person from the basin during a flood.

5.11   Public safety issues

While detention basins are generally less hazardous than drainage channels with respect to water velocity, they are typically much deeper. The safety hazards associated with detention/retention basins are, however, often less obvious to the public. Safety hazards associated with submerged outlet structures can be significant—consequently, measures usually need to be made to prevent the public approaching these structures while the basins are in operation.

The hazards associated with off-stream basins (i.e. basins not directly connected to a watercourse) are likely to be less obvious than those associated with on-stream basins, thus greater consideration may need to be given to safe egress from off-stream basins.

The side slopes of basins should preferably be 1 on 6 or flatter to allow easy egress up the likely wet surface. Areas with slopes steeper than 1 on 4 will require steps and a handrail to assist egress. These recommendations especially apply to basins that incorporate dual use activities such as passive or active recreation.

The provision of exclusion fencing around open water stormwater detention/retention systems should be considered a last resort. Wherever practical, the first preference should be to minimise the safety risk through appropriate design.

Where suitable land is available, designers should aim to restrict basin depths to 1.2 m at the 1 in 20 year AEP level and, if possible, for a greater recurrence interval. In cases where this is neither practical nor economical, and the provision of a detention basin is considered to be better on safety grounds than other alternatives, greater depths may be acceptable. Not withstanding this, designers are responsible for:

- investigating the overall safety risks associated with the basin
- design of the basin and the surrounding landscape in a manner that minimises safety risks
- satisfying any safety requirements specified by the local government.

Suitable safety provisions, such as fences and warning signs, should be provided for deeper basins. Ultimately, the owner of the detention/retention basin is required to accept the ongoing responsibility for maintaining the above safety standards.

Depth indicators should be installed within the basin and in the channel downstream of the embankment for basins with a storage depth of greater than one (1) metre. The indicator within the basin should have its zero level relative to the lowest point in the basin floor.

Special attention should be paid to basin outlets to ensure that persons trapped in the basin's water are not drawn into the basin's outlet system. Rails, fences, anti-vortex devices, trash racks or grates should be provided where necessary. Outlet systems should be located well away from the water's edge of the flooded basin such that a person wading along the edge of the basin cannot be drawn into the basin's outlet. This usually requires the outlet system to be located well away from the embankments.

5.12　Statutory requirements

Works constructed within a watercourse generally require approval under the *Water Act 2000* and need to satisfy all legal requirements of this Act. Reference should be made to this Act for definition of the term watercourse.

Under the *Water Supply (Safety and Reliability) Act 2008* (Water Supply Act) and under common law, responsibility for the safety of a dam rests with the dam owner. Dam owners may be liable for loss and damage caused by the failure of a dam or the escape of water from a dam. Consequently, dam owners need to be committed to dam safety and have an effective dam safety management program. A dam safety management program is intended to minimise the risk of a dam failing and to protect life and property from the effects of such a failure should one occur.

In addition the embankment for a detention basin may be a 'Referable Dam' requiring the approval of the Chief Executive Officer of the State agency responsible for administering the Water Supply Act.

A dam is referable if:

- a failure impact assessment is required to be carried out under the Water Supply Act
- that assessment states that the dam has or will have a Category 1 or Category 2 failure impact rating
- the Chief Executive has, under the Water Supply Act, accepted the assessment.

In addition, some dams may be made referable by:

- a regulation made under the Water Supply Act, or
- the transitional provisions in the *Water Act 2000*.

A failure impact assessment is required when a dam is or will be:

- more than 10 metres in height and have a storage capacity of more than 1500 megalitres
- more than 10 metres in height and have a storage capacity of more than 750 megalitres, and a catchment area that is more than 3 times the surface area of the dam at full supply level.

Additionally, the Chief Executive may give a dam owner a notice to have a dam failure impact assessed (regardless of its size), if the Chief Executive reasonably believes the dam will have a Category 1 or Category 2 failure impact rating.

Referable dams are classified according to categories which are based on the population at risk if the dam fails.

- dams with a Category 1 failure impact rating have between 2 and 100 people at risk
- dams with a Category 2 failure impact rating have over 100 people at risk.

If less than 2 people are at risk by the dam failing then the dam is not referable under the Water Supply Act.

6. Computer models

6.1 Introduction

As with all computer software, designers are expected to be familiar with the underlying concepts used, the limitations of those concepts and the capabilities/limitations of the programs themselves. Further guidance on the use of numerical models is provided in *Australian Rainfall & Runoff* (ARR) and *Australian Runoff Quality* (ARQ).

Designers should be aware of the need for model calibration and the limitations which should be placed upon results where such calibration is not available. Sensitivity analysis is recommended so that the sensitivity of the program's performance in any given situation can be measured against variation in uncertain parameters.

6.2 Computer models

The use of computer modelling for flood assessments and drainage design is now standard industry practice in all but minor drainage systems; however, manual calculation procedures for the estimation of flow and the sizing of drainage components remain an important part of the checking and calibration process.

In broad terms, computer models of relevance to this Manual can be split into three categories, being hydrologic, hydraulic and water quality. The latter is dealt with in detail in the ARQ (Engineers Australia, 2005) and is mentioned here only briefly for completeness.

6.2.1 Hydrologic models

In broad terms, there are two types of hydrologic models, being:

- individual rainfall event simulation
- continuous, long-term simulation of runoff characteristics.

Continuous long-term simulation models are becoming more widely used in understanding the total hydrologic cycle, including effects on volumetric runoff, base-flow in streams and seasonal variability, and the effects of development and infrastructure on the hydrologic cycle. They are also used as part of catchment pollutant yield simulations and associated stormwater management.

Individual rainfall event simulations are aimed primarily at assessing the effects of severe to extreme flood events due to specific rainfall events, usually of durations less than a day, for all but large river systems.

Generally, dynamic analysis—taking account of the shape and volume of the flood hydrograph—is required (except for minor drainage systems) to ensure that the true effects of flooding and development impacts, such as loss of floodplain storage and the timing of the flood wave, are properly understood.

6.2.2 Hydraulic models

With the rapid increase in computational ability of microcomputers, the use of dynamic flow models has become routine and full two-dimensional surface linked to one-dimensional sub-surface models has also become more widespread.

Hydraulic models fall into the following general categories:

- peak flow steady state/backwater (both pipe and surface/open channel) one-dimensional (1D)

- dynamic (full hydrograph) 1D models (both pipe and surface/open channel flow)
- 2D dynamic (surface flow)
- 1D/2D dynamic (combined surface and pipe flow).

There are many specialist 1D peak flow, steady state models available that take account of pressure flow, pipe, pit and inlet losses, pit bypass and inlet and outlet losses. In general, these models are designed for road and trunk drainage systems of localised catchments, where design flows are less than 15 m^3/s.

For large open drain and creek systems, where flow paths are well defined and contained, dynamic 1D modelling is recommended. Steady-state analysis may only be applicable where storage/attenuation and flood peak timing is not critical.

For floodplains or urban flooding situations with complex flow patterns, dynamic 2D modelling is recommended. 1D/2D modelling is also preferred for complex urban flow situations with significant sub-surface flow networks, particularly where there is the potential for significant overland flow that may not follow the road and pipe systems.

6.2.3   Water quality models

Available water quality models are generally either catchment pollutant yield models—which use continuous hydrologic simulation—or in-channel/water body process models. Examples of the former are MUSIC and XP-AQUALM, and of the latter are MIKE-11 WQ, MIKE-21 WQ, SOBEK and Delft 3D. More details are provided in ARQ (Engineers Australia, 2005).

6.3   Reporting of numerical model outcomes

Designers who use numerical models to design and/or support their design, have a duty of care to provide regulating authorities with sufficient information about the model and its outcomes to allow the regulating authority to adequately review the model's suitability and output.

In effect, the designer has two tasks; one, to operate the model appropriately and therefore obtain an appropriate model output; and two, to demonstrate that the model set-up and output are appropriate for the site conditions. It is noted that the latter task cannot be achieved if the regulating authority, their representative, or a third-party reviewer are either not familiar with the model, or are not supplied with sufficient information to review the model and its output.

It is noted that most problems/errors occur with the application of a numerical model rather than the initial development of the software program. If an in-house software model is used in the design of a drainage system, then it is not sufficient to simply indicate to the regulating authority that the software has been calibrated, or that the software is similar to another commercially available program.

As a minimum, when a numerical model is used in the design of a stormwater system, then the following information should be supplied to a regulating authority:

- name and version of software package
- full details of the modelling assumptions
- review of model calibration
- copy of the model's error listing output file
- copies of input data should be made available to the local government (i.e. supplied on request).

7. Urban drainage

7.1 Planning issues

The following discussion outlines key planning issues that need to be considered when designing an urban development and its stormwater drainage network.

7.1.1 Space allocation

At the earliest stages of development planning it is important to allow adequate space for the installation of both the stormwater conveyance and treatment systems. Preliminary subdivision and development layouts should be drafted with appropriate consideration of required space allocations for stormwater systems, including:

• location of overland flow paths

• width requirements for both constructed drainage channels and the protection of existing waterway corridors

• stormwater detention/retention and treatment systems.

In some circumstances, the width of an existing drainage corridor may not satisfy the requirements of current Best Management Practice. For example, an existing overland flow easement on an undeveloped property may have been sized for the requirements of a concrete-lined channel based on an old drainage standard. Such an easement would unlikely be sufficient for the construction of a vegetated open channel based on current standards. Wherever practical, stormwater designers should not limit the width and location of drainage systems to existing drainage easements if such actions limit or prevent the application of current best practice stormwater management principles.

7.1.2 Water Sensitive Urban Design

Stormwater designers are encouraged to incorporate the principles of Water Sensitive Urban Design when planning an urban drainage system (refer to section 11.3.2). The form and layout of an urban drainage system are influenced by a number of key issues, including:

• the preferred location of major overland flow paths

• the retention of natural drainage channels and waterways

• the preferred location of major stormwater detention/retention and treatment systems.

7.1.3 Locating major overland flow paths

The location and design of major overland flow paths is often recognised as the most important part of the drainage system. Failure to adequately plan for major overland flow paths can result in unnecessary property flooding as well as delays in development approval.

The location and anticipated width of major overland flow paths should be identified and mapped during the planning phase of land developments. Wherever practical, major overland flow paths should be maintained along their natural flow paths. These overland flow paths need to be contained within a drainage easement, with the land either managed by a body corporate or government body.

In this context, a 'major overland flow path' is defined as an overland flow path that:

- drains water from more than one property
- has no suitable flow bypass
- has a water depth in excess of 75 mm during the major design storm

  or

- is an overland flow path recognised as significant by the local government.

Locating major overland flow paths through residential properties is strongly discouraged within traditional urban landscapes, especially in greenfield developments. Designers need to consider the following issues:

- Major overland flow paths should be the first components of a drainage system defined for an urban development.

- Special care must be taken to minimise conflicts between overland flow paths and noise control barriers.

- Wherever practical, overland flow paths should follow the natural drainage paths of the catchment.

- Wherever practical, the spacing/density of overland flow paths within the developed landscape should be similar to the spacing/density of the natural gully lines.

- Diverting major overland flows away from their natural flow path may result in significant property damage during storms in excess of the design major storm, or when unexpected debris blockage of the drainage system occurs.

- It cannot be assumed that an overland flow path passing under a residential property fence will be maintained in proper working order at all times. Blockages can occur and should be addressed within the design. Such flow paths may be blocked by garden beds, garden mulch and/or post-development fencing modifications made for the purpose of containing domestic pets.

- Overland flow paths within residential properties may also transport excessive quantities of organic matter, including grass clippings and garden mulch. Such material may result in debris blockages of downstream drainage systems and waterway pollution.

- Overland flow paths should not pass through waste collections compounds where such actions are likely to result in wheelie bins or industrial waste collection skips floating away during severe storms. Such items are increasingly contributing to flood debris and the blockage of drainage structures such as waterway culverts.

- Further to the above, overland flow paths should also not become a source of hazardous materials during major storms. Not only can such materials harm receiving waters (a matter regulated by the *Environmental Protection Act 1994*) but they can also represent a safety risk to emergency personnel conducting inspections of flooded properties or carrying out flood rescues.

Not all of the above issues may be applicable to rural residential areas. Also, designers should ensure that wherever practical, the operation of overland flow paths will not compromise emergency/maintenance access to essential equipment and infrastructure.

7.1.4 Provision of piped drainage systems

Water Sensitive Urban Design does not exclude the use of piped drainage systems, rather it focuses on limiting their use, and minimising the direct connection of impervious drainage surfaces to piped drainage.

Consideration should be given to the piping of minor flows in the following circumstances:

- when it is unsafe, impractical, or otherwise undesirable to carry minor storm flows within an open channel or overland flow path

- when flow passage within an open drain or overland flow path exceeds the design standards of the flow path (e.g. depth*velocity product, flow width, channel capacity or allowable flow velocity)

- where a piped drainage system is required in association with a swale or overland flow path to provide a discharge point for a sub-surface drainage system.

Local authorities should give consideration to the adoption of a maximum desirable catchment area (appropriate for their region) for piped drainage systems.

7.1.5 Provision of grassed and vegetated drainage channels

The application of grassed channels is generally limited by design requirements such as the allowable flow velocity, depth*velocity product, or maximum desirable bed width (refer to section 9.2).

Consideration should be given to the incorporation of the principles of Natural Channel Design for the design of constructed drainage channels in the following circumstances:

- when the channel is required to have a natural appearance

- when it is necessary to incorporate aquatic or terrestrial habitat, or when the channel forms part of a fauna corridor

- when rehabilitating a natural drainage channel or waterway within a heavily modified catchment.

For further discussion on vegetated channels and Natural Channel Design, refer to section 9.6 of this Manual.

7.1.6 Retention of natural drainage channels and waterways

Consideration should be given to the 'retention' of existing natural channels in the following circumstances (also refer to section 9.2):

- waterways identified as important within a Waterway Corridor Plan, Catchment Management Plan, or similar strategic plan

- waterways defined as fish corridors by Queensland Fisheries (DAFF)

- natural waterways with well-defined bed and banks, and associated floodplain/s or riparian corridors.

7.1.7 Drainage schemes within potential acid sulfate soil regions

Guidelines for the planning of drainage systems located within potential acid sulfate soils are presented in section 9.7.9.

7.2   The major/minor drainage system

7.2.1   General

Design of the drainage system should be in accordance with the Major/Minor drainage concept discussed in Argue (1986) and Australian Rainfall & Runoff. This design concept recognises the dual requirements of the drainage system to provide for convenience and the protection of life and property for all storms up to the nominated major storm event. In addition, consideration should also be given to the consequences of events in excess of the major storm.

The appropriate Annual Exceedence Probabilities (AEPs) for design are detailed in section 7.3 and are applicable to normal design situations. The local government may direct that certain developments, or sections of developments, be designed for greater or lesser immunity than those outlined.

In a system designed in accordance with the Major/Minor drainage concept, the flow under both minor and major storm conditions is conveyed partly by the minor surface drain or underground pipe system, and partly by the major surface flow components of the system. As a consequence, it would not be reasonable to say that an underground system has been designed to convey the peak discharge from a storm of given AEP. Rather the system as a whole will convey the flows under both minor and major storm conditions.

Designers should note that constraints on the safe management of the major system discharge may require an increase in the capacity of the minor system beyond that required by the design discharge for the minor system alone.

7.2.2   Minor drainage system

The minor drainage system includes kerbs and channels, roadside channels, drainage swales, inlets, underground drainage, junction pits, access chambers and outlet structures designed to fully contain and convey the discharge from the minor storm.

This arrangement may also include:

- Field or kerb inlet pits installed to collect surface runoff from within allotments, as well as the roof-water drainage provisions for buildings.

- Cross drainage under minor roads where delay or inconvenience during major flows is acceptable. This also includes low-flow pipes or box culverts installed under floodways.

- Low-flow pipes installed under drainage reserves or park areas.

The recommended 'performance objectives' of the minor drainage system design are:

- Operation of the drainage system during a minor storm does not cause unacceptable safety risks.

- Drainage infrastructures do not present a safety risk to the public.

- Operation of the drainage system during a minor storm allows convenient and flood-free movement of vehicles and pedestrians.

- Operation of the drainage system during the nominated minor storm does not cause unacceptable flood damage.

- Operation of the drainage system during a minor storm allows the normal use of the land soon after cessation of rainfall.

- The minor drainage system appropriately integrates into natural and built environments.

- The drainage system is designed to minimise adverse impacts upon the values of receiving waters caused by the velocity of stormwater discharges.

- The drainage system is designed to minimise adverse impacts upon the values of receiving waters caused by changes to the natural water cycle with respect to the volume, frequency and duration of runoff.

- The drainage system is designed to minimise adverse impacts upon the values of receiving waters caused by the quality of stormwater discharges.

- The value of stormwater as a potential water source is appropriately realised.

- Sufficient information is supplied to demonstrate the adequacy of numerical models.

- The cost of constructing the minor drainage system is affordable for the asset provider.

- The cost of maintaining the minor drainage system is affordable for the asset manager.

7.2.3   Major drainage system

The major drainage system is that part of the overall drainage system designed to convey a specified major storm flow. This system may comprise:

- open space floodway channels, road reserves, pavement expanses and other flow paths designed to carry flows in excess of the capacity of the minor drainage system

- natural or constructed waterways, detention/retention basins and other major water bodies

- major underground piped systems installed where overland flow is impractical, unacceptable, or incapable of carrying the required discharge.

The recommended performance objectives of the major drainage system design are:

- Operation of the drainage system during the nominated major storm does not cause unacceptable safety risks.

- Operation of the drainage system during the nominated major storm does not cause unacceptable flood damage.

- Operation of the drainage system during the nominated major storm allows safe movement of emergency vehicles.

- To the maximum degree practicable, the principles of Water Sensitive Urban Design are integrated into the planning and design of major drainage structures.

- Operation of the drainage system during the nominated major storm does not cause adverse, long-term changes to the normal use of the land.

- The drainage system appropriately integrates into the natural and built environments.

- Operation of the stormwater system does not cause unnecessary soil erosion.

- The drainage system is designed to minimise adverse impacts upon the values of receiving waters caused by changes to the natural water cycle with respect to the volume, frequency, and duration of runoff.

- Adequate space is provided for the conveyance of major storm flows and for the maintenance of stormwater infrastructure.

- Major overland flow paths are retained along their natural alignment.

- The cost of constructing the major drainage system is affordable for the asset provider.

- The cost of maintaining the major drainage system is affordable for the asset manager.

- The drainage system is designed to maximise its resilience to flood damage.

Local governments may adopt a 'Defined Flood Event' for waterway flooding in accordance with State Planning Policies. Currently, these state planning documents strongly recommend that the 1% AEP (100 year ARI) is adopted as the Defined Flood Event. It is noted that the nominated major system design AEP for such things as overland flow paths may be different from the Defined Flood Event.

The design of the major drainage system should:

- Account for the flow conveyed in the underground minor drainage system and for the consequences of malfunctions or blockages in that system.

- Demonstrate that the inlet system for the minor drainage network can continue to operate under appropriate levels of debris blockage (refer to section 7.5.2) otherwise appropriate adjustments must be made to the design of the major drainage system to account for potential malfunctions or blockages in the minor drainage system.

The design of major underground drainage systems with no overland flow component is **strongly** discouraged, and should only be adopted where overland flow is either impractical or unacceptable. In circumstances where a major underground pipe system is used with no overland flow component, the designer shall prepare a report for the local government. As a minimum, this report shall discuss the following issues:

- analytical justification demonstrating that design flows can enter the underground drainage system under appropriate blockage conditions
- potential effects of flows in excess of the design flow including the consequences of the Probable Maximum Flood (also refer to sections 7.2.4 & 7.3.3)
- allowances made in the design for debris blockage of inlets
- potential effects of debris blockages in excess of that allowed for in the design.

When assessing the potential effects of debris blockage, consideration must be given to at least the following:

- potential risk to life either directly resulting from the blockage or resulting from rapid or unexpected changes in flow conditions resulting from the blockage
- potential floor level flooding
- adverse affects on the use of adjacent land
- the potential for unrepairable property damage (e.g. damage to historical sites, or severe erosion that threatens the structural integrity of major structures).

7.2.4   Operation of the drainage system during severe storms

Traditionally, drainage systems have been designed using the Major/Minor drainage concept, with limited consideration of storms in excess of this design storm. One of the potential adverse effects of this design philosophy is that the degree of storm/flood damage can dramatically escalate during severe storms that exceed the major storm design standard.

To manage this condition it is recommended that the stormwater design process be expanded to include the preparation of a 'Severe Storm Impact Statement'. These statements may vary from a single paragraph to a comprehensive report depending on the assessed hazards and consequences.

Recommended performance objectives have been prepared for drainage systems operating under storms that exceed the nominated major storm. The **intent** of these performance objectives may be summarised as:

- To provide an appropriate incentive and process to encourage and enable stormwater designers to consider the potential impacts of storms and floods in excess of the nominated design storm or flood.

- To avoid the circumstances where the nominated major storm AEP is increased above the 1 in 100 year event without due consideration of the benefit:cost impacts.

- To manage the cost of damage to private and community assets resulting from severe storms/floods to an acceptable level.

- To manage safety risks resulting from severe storms/floods to an acceptable level.

- To take appropriate steps to improve the resilience of the State to the occurrence of severe storms/floods.

The recommended **performance objectives** of the drainage system during a storm that exceeds the major storm, and/or where the site conditions (e.g. debris blockage) exceed those conditions assumed for the major storm case, are listed below:

- Operation of the drainage system during storms in excess of the major storm does not cause a rapid or unexpected increase in safety risks or flood damage.

- Operation of the drainage system during severe storms does not cause the unacceptable isolation of essential community infrastructure or residential areas.

- Operation of the drainage system during severe storms does not cause rapid or unexpected increases in the extent and/or cost of flood damage relative to natural flood conditions.

- Operation of the drainage system during severe storms does not cause, or increase the degree of, unrepairable property damage (e.g. damage to historical sites, or severe erosion that threatens the structural integrity of essential community infrastructure).

- Operation of the drainage system during severe storms does not result in unnecessary damage to private and community assets that could otherwise have been avoided through appropriate awareness training and/or warning signs.

- Operation of the drainage system during severe storms allows safe movement of emergency services vehicles.

- Operation of the drainage system during severe storms does not cause adverse, long-term changes to the normal use of the land.

- The cost of constructing the major drainage system remains affordable for the asset provider.

- The cost of maintaining the major drainage system remains affordable for the asset manager.

- The drainage system's resilience to flood inundation is maximised.

7.2.5   Preparation of a Severe Storm Impact Statement

In general terms, a Severe Storm Impact Statement is similar to an environmental impact statement in that the complexity and detail presented within the report needs to be commensurate with the assessed risks; in this case, any safety or flood hazard risks.

In is simplest form, the Severe Storm Impact Statement should address, in short sentences, how the drainage system will address the performance objectives listed in section 7.2.4. If the potential flood hazard resulting from a severe storm is considered high, then a more detailed statement will be required in response to each of the objectives.

In some cases these statements will need to be supported by an appropriate risk assessment investigation and/or numerical flood modelling. This however, does not imply that all new

stormwater drainage designs should undergo PMF modelling. The need for numerical modelling beyond the major storm discharge depends on the assessed risks.

The preparation of a Severe Storm Impact Statement may also be warranted during the design of structures that may interfere with the passage of stormwater. The following list provides examples of circumstances where a Severe Storm Impact Statement would likely be required.

- Proposed installation of noise control barriers over an overland flow path where the barrier could cause unacceptable flooding or flow diversion during severe storms.

- Installation of solid traffic-control barriers along the median of a roadway that crosses over a waterway or valley (i.e. where overland or overtopping flows would normally have occurred across the roadway).

- Subdivision of land upstream of a road or railway embankment where the occurrence of severe blockage of the culvert, or the occurrence of a discharge in excess of the major storm, would likely cause unacceptable flooding of the subdivided land.

- Design of a stormwater detention/retention basin where the potential impacts of an inflow in excess of the major storm (peak flow rate or volume) would likely cause unacceptable flooding.

- Design of a stormwater detention/retention basin where the failure of the structure, or its embankment, could potentially result in an increase in flood levels (relative to the no-failure case) of at least 300 mm through a residential or workplace building. In the latter case, such conditions are likely to make the structure a Referable Dam (refer to section 5.12).

- Rehabilitation or revegetation of an urban waterway where the potential impacts of a discharge in excess of the major storm would likely cause unacceptable flooding.

7.3   Design standards

7.3.1   Design AEPs

The Annual Exceedence Probability (AEP) as used in this Manual is the probability of exceedence of a given rainfall intensity or discharge within a period of one year. Throughout this Manual the AEP of the design flood/discharge is assumed to be the same as the AEP of the nominated design storm.

Tables 7.3.1 and 7.3.2 show **recommended** AEPs and the equivalent ARIs for minor and major rainfall events associated with a range of land uses and development categories. The final selection of the design AEP may be influenced by factors, such as:
- required level of service for hydraulic performance
- construction and operating costs
- maintenance requirements
- the need to reduce potential flood damage based on a risk assessment process
- safety
- aesthetics
- regional planning goals
- legal and statutory requirements
- convenience or nuisance reduction requirements.

A local authority may vary the design AEPs/ARIs shown in tables 7.3.1 and 7.3.2 to suit local conditions. However, it is recommended that the minor system AEP should not be reduced below 1 in 2 years (39% AEP) in respect of the Residential and Industrial development categories, or below 1 in 1 year (63% AEP) for Open Space, Parks, etc.

Since the release of the original 1992 edition of the Manual there has been significant discussion and debate over the choice between the 1% and 2% AEPs for the selection of the major storm. The outcomes of these discussions generally fall along the following lines:

- State Planning Policies recommend adoption of the 1% AEP (1 in 100 years) flood frequency for waterway flood management planning.

- Many of the organisations that had previously adopted a 2% AEP (1 in 50 years) major storm are moving towards, or have already adopted, a 1% AEP major storm in all circumstances.

- Within those organisations that adopt a combination of 1% or 2% AEP for the design of overland flow paths, the design standard is generally based on the following conditions:

  o the 2% AEP is commonly adopted for the design of drainage paths in circumstances where the surface/channel roughness conditions are known with a high degree of certainty, and these surface conditions are expected to be well maintained

  o the 1% AEP is commonly adopted for the design of drainage paths where it is difficult to predict actual flow conditions (e.g. complex 3D hydraulics) or where the surface roughness can be highly variable (e.g. vegetated channels).

- In most organisations, minimum fill levels and minimum habitable floor levels are set relative to the 1% AEP flood level even if a 2% AEP major storm has been adopted for design of the adjacent overland flow path.

When selecting design AEPs, local authorities should consider the required performance objectives of each system as outlined in section 7.2.

Examples where AEPs of a higher standard to those recommended in tables 7.3.1 and 7.3.2 include the following:

- Where runoff from an up-slope catchment is piped through private property and there has been no allowance for, nor opportunity to, protect the property from inundation by flows that exceed the desired standard of service of the pipeline.

- Where higher residential densities are likely as a result of long-term infill and population growth, and nuisance flooding may lead to more severe consequences.

- Where mixed residential and commercial development is proposed.

- Where a risk-based analysis identifies the potential for an unacceptable increase in the degree of flood damage during events that exceed the nominal design storm (refer to section 7.3.3).

AEP values presented in tables 7.3.1 and 7.3.2 are **recommended** values for the design of new works and the upgrading of existing systems. The design standard for 'relief' drainage (section 13.1) may or may not be consistent with tables 7.3.1 and 7.3.2 depending in part on cost-benefit analysis, site conditions, and site constraints.

Table 7.3.1 – Recommended design average recurrence intervals (ARI) and annual exceedence probabilities (AEP) for the minor system

Development category [1]		ARI (yrs)	AEP
Central business and commercial		10	10%
Industrial		2	39%
Urban residential high density – greater than 20 dwelling units/ha		10	10%
Urban residential low density – 6 to 20 dwelling units/ha		2	39%
Rural residential – 2 to 5 dwelling units/ha		2	39%
Open space – parks, etc.		1	63%
Major road	Kerb and channel flow	10 [2]	10%
	Cross drainage (culverts)	50 [3]	2%
Minor road	Kerb and channel flow	[4]	[4]
	Cross drainage (culverts)	10 [3]	10%

Notes (Table 7.3.1):

[1]   The terms used in this table are described in the Glossary (Chapter 13) and Table 7.3.3.

[2]   The design AEP for the minor drainage system in a major road shall be that indicated for the major road, not that for the development category of the adjacent area.

[3]   Refer to discussion in section 7.3.7.

[4]   Refer to relevant development category.

Table 7.3.2 – Recommended design average recurrence intervals (ARI) and annual exceedence probabilities (AEP) for the combined minor/major system

Development category [1]	ARI (yrs)	AEP
Reference flood for setting floor levels in hospitals, emergency services, flood evacuation buildings and Civil Defence HQ	500	0.2%
Reference flood for setting floor levels of emergency shelters, police facilities, museums, libraries, storage facilities for valuable records or item of historical or cultural significants, and housing for aged and those with impaired mobility; and the setting design levels for water and wastewater centres [2] and critical utility services infrastructure [2]	200	0.5%
Reference flood for setting habitable floor levels in residential buildings and floor levels in commercial/industrial buildings adjacent floodplains or overland flow paths	100	1%
Design storm for overland flow paths	50 or 100	2 or 1%

Notes (Table 7.3.2):

[1]   The terms used in this table are described in the Glossary (Chapter 13) and Table 7.3.3.

[2]   Refers to critical components of the system that are required to be flood-free in order to allow prompt and cost-effective recovery of services after a flood (e.g. electrical equipment).

[3]   Refer to relevant local authority for confirmation of design storm AEP. Fill, building and floor levels are usually set relative to the 1% AEP event even if the overland flow path design storm represents a 2% probability.

7.3.2 Selection of the major storm AEP based on risk assessment

As noted above, the nominated major storm AEP presented in Table 7.3.2 may need to be increased when designing critical infrastructure in circumstances where flows in excess of the 1% AEP discharge could result in unacceptable flood damage or safety risks.

In most cases these safety risks will be associated with the following conditions:

- floodwaters are redirected by a choked bridge or culvert through populated areas at flow velocities that would cause structural damage to buildings or possible loss of life
- structural failure of a road or rail embankment that could cause loss of life similar to a dam failure.

In either case, the assessment of an alternative major storm design standard should be based on procedures similar to those presented for a dam failure analysis.

7.3.3 Consideration of events in excess of the major storm

The likely effects of stormwater flows resulting from events in excess of the design storm should be considered and the consequences discussed with the local government (refer to section 7.2.4). The consideration of choice of extreme events should be based on recommendations of State Planning Policies, ANCOLD (1986), or the local authority as appropriate.

When assessing the potential effects of flows in excess of the design flow, consideration must be given to at least the following:

- floor level flooding
- adverse affects on the use of adjacent land
- potential unrepairable property damage (e.g. damage to historical sites, or severe erosion that threatens the structural integrity of major structures).

In cases where potential flow restrictions or diversions are introduced to an overland flow path, then the consequences of such restrictions or diversions shall be considered for flows in excess of the specified major storm. The regulating authority may require consideration of flows up to the PMF. If it is not practical to determine the PMF, then a nominal flow rate of four times the 1% AEP peak discharge may be considered acceptable. The assessed consequences shall be discussed with the relevant regulating authority.

There are no specific quantitative requirements for the performance of stormwater drainage systems operating under flows in excess of the nominal major storm. Each site must be assessed on a case-by-case basis. If the Severe Storm Impact Statement (section 7.2.5) identifies an unacceptable risk, then the local government may choose to adopt a more severe AEP for the major storm and/or adopt a higher design standard for minimum fill and/or floor levels.

7.3.4 Land use/development categories

In order to determine the desired standard of service (i.e. appropriate minor and major AEP values) it will be necessary to assess the development category for the catchment. Development categories are broadly defined in Table 7.3.3.

A local government may use different terminology to that presented in Table 7.3.3. It is the responsibility of the designer to check with the relevant local government to determine the actual development category which is applicable to the land use or zoning, or the potential land use or zoning for the catchments under consideration.

Table 7.3.3 – Development categories

Central business	A section of a city or town where the primary use is for business or retail activities and where buildings are commonly built up to the property boundaries, awnings overhang the footpaths and landscaping is minimal or non-existent. Central business areas are often encapsulated within the older parts of a city or town. This category would likely include Queensland Planning Zones: central, principal central, major centre and district centre.
Commercial	A building or group of buildings where primary uses include retail sales, business activities, health activities, hospitality functions, etc. It may include regional shopping centres, business centres, hospitals, medical facilities, food outlets, sports centres, car sales yards, entertainment facilities, nurseries and the like. This category would likely include Queensland Planning Zones: central, principal central, major centre, district centre, local centre, neighbourhood centre, community facilities.
Industrial	Areas where the primary activities carried out are manufacture, processing, trade sales or storage facilities, etc. (e.g. motor vehicle repairs, manufacture, wholesale, warehouses). This category would likely include Queensland Planning Zone: industry.
Urban residential high density	Residential areas which have greater than 20 dwelling units per hectare, including multi-unit residential and cluster housing. This category would likely include Queensland Planning Zone: apartment residential.
Urban residential low density	Residential areas which have over 5 and up to 20 dwelling units per hectare e.g. normal detached houses on residential allotments. This category would likely include Queensland Planning Zones: general residential, residential living, residential choice and tourist accommodation.
Rural residential	Rural residential areas which have between 2 and 5 dwelling units per hectare e.g. a house on 2000 m^2 to 5000 m^2 allotment. This category would likely include Queensland Planning Zone: rural residential.
Open space	Open areas primarily used for recreation or drainage including parks, golf courses, trunk drainage channels etc. This category would likely include Queensland Planning Zones: recreation and open space, sport and recreation, and open space.
Major road or minor road	Consult the relevant local authority for the appropriate road classification to be adopted i.e. major or minor. Guidance in this regard is given in section 7.4 and the Glossary (Chapter 13). Examples of major roads are: highways, arterial and sub-arterial roads and trunk collector roads. Examples of minor roads are: access places and access streets.

7.3.5 Essential community infrastructure

Reference should be made to Table 7.3.2 of this Manual for design immunity recommended for strategic facilities e.g. Hospitals, Civil Defence headquarters, Police, Fire and Ambulance.

When making planning decisions in regards to the setting of minimum fill or floor levels, the intent should be to take all reasonable measures to establish systems that are resilient to severe floods for the benefit of the State and the community.

7.3.6 Overland flow paths

The design of overland flow paths can be very complex with many of the flooding issues associated with floodplains also applying to overland flow paths. Design standards relating to overland flow paths include the minimum flow capacity specified in Table 7.3.2, and the maximum allowable depth*velocity product (d.V) specified in Table 7.3.5.

It is strongly recommended that **major** overland flow paths (i.e. those defined in section 7.1.3) are not located within private properties. If it is unavoidable, then an overland flow easement should be obtained over the flow path to allow local governments to control works within these flow paths that could adversely affect adjacent properties.

It is often necessary to build over **minor** overland flow paths, such as in the construction of property fencing, sound-control barriers and minor foot bridges. When designing such structures it is important to consider the consequences of flows in excess of the nominated major storm as discussed in sections 7.2.4 and 7.3.3.

Also, as discussed in section 7.1.3, it cannot be assumed that an overland flow path passing under a residential property fence will be maintained in proper working order at all times. Blockages can occur and should be addressed within the design. Such flow paths may be blocked by garden beds, garden mulch or by fencing modifications designed to contain domestic pets.

7.3.7 Cross drainage structures (culverts)

Culverts under roads should be designed to accept the full flow for the minor system AEP shown in Table 7.3.1. In addition, the designer must ensure adequate public safety controls (e.g. maximum flow depth and d.V product) exist for flows passing over the road surface, and that the nominated major storm flow does not cause unacceptable damage to adjacent properties.

If upstream properties are at a relatively low elevation, it may be necessary to install culverts of a greater capacity to that for the minor system's design storm to ensure adequate flood protection of upstream properties.

In urban areas, culverts and causeways are generally considered trafficable when the maximum flow depth within a trafficable lane does not exceed 200 mm and the depth*velocity product does not exceed 0.3 m^2/s (refer to tables 7.4.2 and 7.4.4).

The intent of the higher design standards presented in Table 7.3.1 (Note 4) for cross drainage is to reduce the safety risks to those vehicles passing along a roadway where system bypass flows would pass **transversely** across to the road alignment. Such conditions most commonly occur at watercourse crossings, but can also occur at road bends or changes in road camber where excess surface flows spill from one side of a road to the other.

In circumstances where stormwater flows across a road surface in such a manner that could cause traffic safety issues (e.g. aquaplaning) the road surface should be 'trafficable' during the nominated storm for cross drainage (i.e. 2% AEP for major roads and 10% AEP for minor roads). This means

the longitudinal piped drainage system may need to have an increased flow capacity in order to provide safe driving conditions on the full length of the roadway.

When determining the local design standard, local governments should consider the expectations of a particular type of roadway carrying traffic during major storms such as the 2% AEP. It should be noted that in many regions of Queensland it becomes very difficult to drive during storms that exceed a 10% AEP due to the limited visibility through vehicle windscreens. This means two things: firstly, vehicles are likely to be travelling slowly; secondly, drivers are less likely to observe, and appropriately respond to, unsafe drainage conditions on the roadway.

The above design standard would **not** apply to a drainage pipe that simply crosses from one side of a road to another in circumstances where bypass flows would not pass 'across' the road crown. In general, this is not a simple design issue and each circumstance should be considered on a case-by-case basis in consultation with the relevant road authority.

7.3.8   Flood evacuation routes

Guidance on the design of evacuation routes is provided in the State's Planning Policies (SPP 1/03 or its replacement).

It is noted that the trafficable operation of evacuation routes is often limited to the flooding of culvert and bridge crossings. Table 7.3.1 provides recommended design standards for cross drainage structures (e.g. culverts) on minor and major roads. This table should **not** be used to set design standard for flood evacuation routes.

7.3.9   Basements and non-habitable rooms of buildings

In the past, the focus of urban flood management has been on the protection of habitable rooms. In recent times however, the financial and emotional cost of the flooding of basements and non-habitable rooms has increased due to a number of factors, including:

- the increased value and importance of electrical equipment housed within basements
- the value and quality of materials used in the furnishing of non-habitable rooms, including the type of wall cladding
- the unauthorised conversion of non-habitable rooms into 'habitable rooms' such as entertainment centres and spare bedrooms.

According to the Building Code of Australia, a non-habitable room includes a bathroom, laundry, water closet, pantry, walk-in wardrobe, corridor, hallway, lobby, photographic darkroom, clothes drying room, and other spaces of a specialised nature occupied neither frequently nor for extended periods.

Councils can help to improve the State's resilience to floods by:

- setting minimum floor levels for non-habitable rooms in circumstances where the cost of flood damage to such rooms is likely to be significant
- restricting the types of equipment (such as generators) that can be housed within flood-prone basements in accordance with current best practice floodplain management principles.

7.3.10 Public car parks

When setting the flood immunity of public car parks, consideration should be given to:

- the likelihood of flood warnings
- the likelihood of users being aware of, or being able to respond to, any flood warnings (e.g. warning times, access to parked cars, egress of cars from dangerous waters)
- required flood warning signs
- the likely safety risks, flood risks and resulting damage associated with displaced cars becoming debris blockage within downstream stormwater/watercourse structures.

7.3.11 Areas of manufacture or storage of bulk hazardous materials

It is important that hazardous materials are not stored within overland flow paths where they could reasonably be expected to be displaced by stormwater runoff. Refer to the *Work Health and Safety Act 2011* and associated Regulation and Guidelines, the *Environmental Protection Act 1994* and the relevant building assessment provisions under the *Building Act 1975* for requirements related to the manufacture and storage of hazardous substances within overland flow paths.

7.3.12 Freeboard

The primary purpose of freeboard is to address issues such as uncertainties in flood level prediction, variations in structure blockages, variations in water level across the floodplain (e.g. superelevation), conversion of the water's kinetic energy (velocity head) into potential energy, and the effects of wave action.

Wave action can result from a number of forces depending on location. Along overland flow paths, wave action can result from vehicle movements along flooded roads, or the effects of standing waves caused by supercritical flows passing around obstructions. On water bodies, wave action can result from local winds or watercraft. In coastal regions, waves can be generated by local winds or distant storms.

In normal circumstances, freeboard should **not** be relied upon to provide additional protection beyond the nominal design flood event. Stormwater designers must acknowledge that the water surface of waterways and overland flows during major storms events is rarely smooth and level. In many circumstances, the effective protection of buildings from flood inundation resulting from a Defined Flood Event is only achieved through the adoption of a nominal freeboard above the 'theoretical' flood level.

That said, during the preparation of a Severe Storm Impact Statement it will be necessary to assess the likely flood inundation during storms in excess of the defined flood event. In such cases it is appropriate to assume the existence of freeboard will provide additional flood protection. Therefore, such an analysis would represent a 'theoretical' prediction of flood inundation. It is noted that the completion of such a flood study should **not** be used to imply or advertise that the assessed buildings or properties have a flood immunity greater than the Defined Flood Event.

General freeboard recommendations are provided in tables 7.3.5 and 7.4.3, and figures 7.3.1 and 7.3.2. This manual does not specifically address freeboard requirements for coastal regions where higher freeboards are often recommended. Freeboard requirements for open channel are discussed in Chapter 9 – *Open channel hydraulics*.

Local governments that choose a major design storm standard less that the 1% AEP (1 in 100 years) may choose to adopt higher freeboard requirements. Alternatively, the local government

may require additional hydraulic checks to ensure floor levels are at least above the anticipated 1% AEP peak water level.

Local governments should consider setting minimum floor levels in critical areas to minimise the risk of future building works being constructed below the anticipated 1% AEP peak water level (refer to the requirements specified within current State Planning Policies).

7.3.13 Risk-based freeboard requirements

Risk-based freeboard requirements are most commonly applied to the design of flood levees. A risk-based analysis of freeboard requirements may also be incorporated into a severe storm impact statement. Such an analysis may consider issues such as:

- the increase in debris blockage required within a stormwater system (e.g. culvert inlet) to elevate the major storm flood level to the top of nominated freeboard
- the increase in the major storm discharge required to elevate the major storm flood level to the top of nominated freeboard.

The outcomes from such an analysis may prompt the local government to set alternative freeboard requirements.

7.3.14 Easement widths

Easements for drainage purposes are generally obtained over stormwater pipes located within freehold land if the pipe diameter exceeds 300 mm. Drainage easements may also need to be obtained over overland flow paths that cross more than one property boundary where it may be necessary for the local government to manage the operation and/or maintenance of the overland flow path and any cross drainage structures, such as fencing and noise control barriers.

These easements need to be of such width, length and location to enable necessary works (e.g. construction, maintenance and site inspection) to be carried out.

Recommended drainage easement widths are discussed in section 3.8.

7.3.15 Flow depth and width limitations

The drainage system should be designed so that the flow depth, flow width and pedestrian/vehicle safety limitations are met for the required major and minor design storm conditions. These limitations are detailed in tables 7.3.4, 7.3.5 and 7.4.1 to 7.4.4, and figures 7.3.1, 7.3.2 and 7.4.1.

Accordingly the underground piped drainage system and the inlets etc. leading to it must be designed to accept that part of the flow which cannot be contained in surface flow paths such as roads, channels and overland flow paths operating under major and minor storm conditions respectively whilst complying with the flow depth/width limitations.

The flow depth and flow spread should be limited by whichever of the criteria in tables 7.3.4 and 7.3.5 is the most restrictive. These criteria are shown diagrammatically in figures 7.3.1 and 7.3.2 whilst the manner in which these criteria and those of section 7.4 restrict flow depth and width within road reserves are detailed in tables 7.4.1 to 7.4.4 and Figure 7.4.1.

Table 7.3.4 – Flow depth and width limitations for the minor storm

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Minor system design criteria:

(a) The underground drainage system together with associated inlets, access chambers, outlets, etc. shall be designed to convey the discharge for the design minor storm with road flow limited as detailed in (c) below.

(b) Field inlets shall be provided to collect allotment runoff as detailed in section 7.13.

(c) Road flows shall be restricted by:

- flow spread limitations on the road pavement and the positioning of kerb inlets as detailed in sections 7.4 and 7.5

- flow conditions limited by $d.V ≤ 0.3\ m^2/s$ for flow 'transverse' to the road alignment where the risk to life is reasonably foreseeable (also see Table 7.4.2).

(d) The total flow for the minor flood event shall be contained within the drainage easement or drainage reserve provided through a park or open space.

Table 7.3.5 – Flow depth and width limitations for the major storm

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Major system design criteria:

(a) Freeboard not less than 300 mm below floor level of an adjacent building where the building is located on ground that is above street level.

(b) Water surface not greater than 50 mm above top of kerb, where the floor level of an adjacent building is less than 350 mm above top of kerb and the fall across the footpath towards the kerb is greater than 100 mm. Otherwise the flow depth must be restricted to top of kerb in conjunction with a footpath profile that prevents flow from the roadway entering onto the adjacent property. Where no kerb is provided the above depths shall be measured from the theoretical top of kerb.

(c) The product of flow depth and velocity shall be limited by the formula:

$$d_g.V_{ave} ≤ 0.6\ m^2/s \qquad\qquad (7.1)$$

where:

d_g = maximum flow depth (e.g. at kerb invert) (m)

V_{ave} = average flow velocity within the flow path (m/s)

If the risk to life is reasonably foreseeable, then $d_g.V_{ave} ≤ 0.3\ m^2/s$ (refer to Table 7.4.4).

(d) The total overland flow for the major flood event shall be entirely contained within a road reserve, drainage reserve, park or open space and shall be limited to such depth to ensure a minimum 300 mm freeboard below the floor level of an adjacent building.

(e) Maximum flow depth of 300 mm and depth*velocity product, $d.V ≤ 0.3\ m^2/s$ in car parks where the flow depth is near uniform across its width.

(f) Where flow is contained in an open channel, freeboard in accordance with section 9.3.4.

(g) Such other limitations or relaxations as may be set by the local authority.

Road reserve

Major flood event contained within road reserve or lesser depth to maintain freeboard.

Freeboard 300 mm

Floor level

Underground pipe network

Building above top of kerb and channel

See detail 'A'
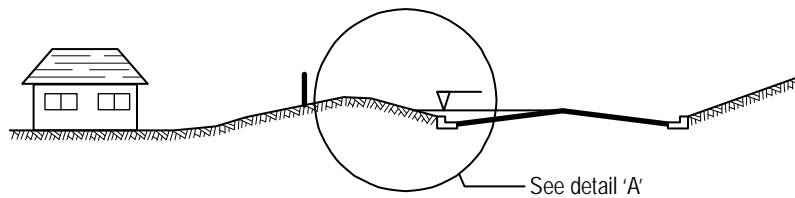
Building below top of kerb and channel

50 mm limitation above top of kerb (Major system)

Footpath

100 mm

Detail 'A'

See detail 'A'
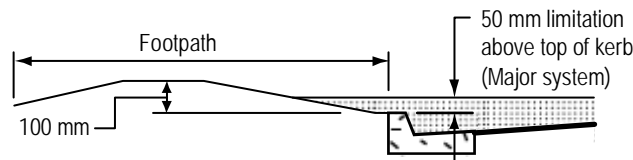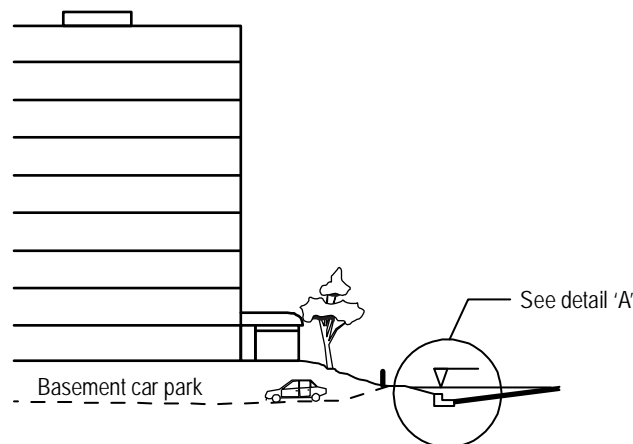
Basement car park

Multi-unit or commercial development with below ground car parking or building services

Figure 7.3.1 – Major storm flow design criteria
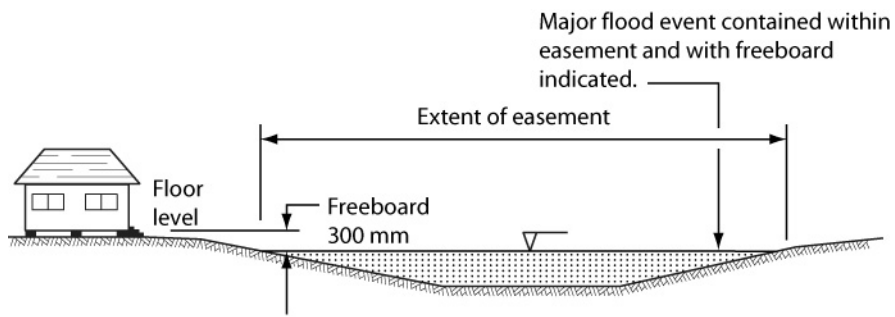
Easement width requirements for overland flow path
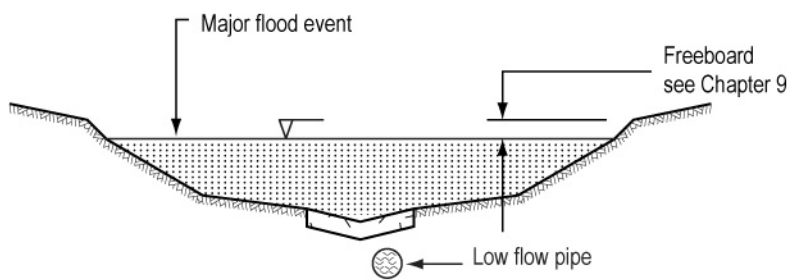
Channel with or without low-flow pipe
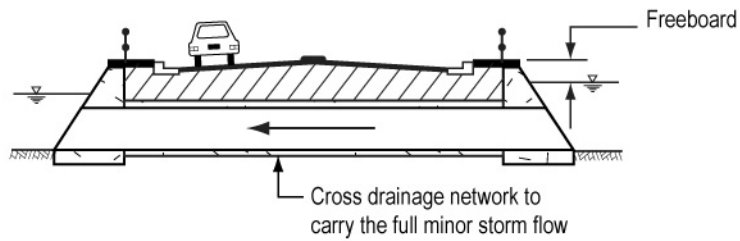
Cross drainage (minor storm conditions)
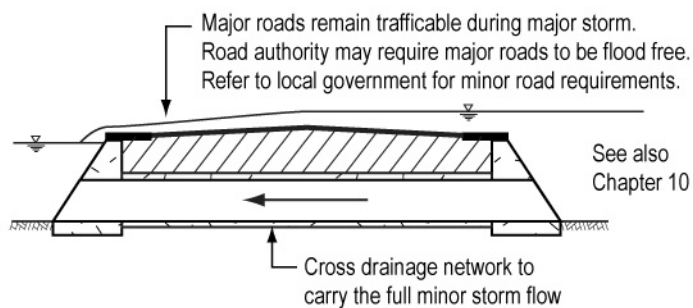
Cross drainage (major storm conditions)

Figure 7.3.2 – Major storm flow design criteria

7.4   Roadway flow limits and capacity

It is necessary for road flow capacity to be checked for both the minor and major design storms. Design criteria are provided in section 7.3. Additional criteria also apply and these are outlined in the following sections.

Note that in this section, and others, reference is generally made to roads with kerb and channel. This is not meant to preclude the use of grassed channels located at the verges, nor other edge treatments. The type of road edge treatment should be decided after consultation with the local authority.

7.4.1   Flow width (minor storm)

The flow width criteria for minor storms are related to the function of the road. Definitions of major and minor roads, for the purpose of this Manual, are contained in the Glossary (Chapter 13).

Note: It should be emphasised that flow width restrictions are dependent on the function of the road and its expected maximum traffic catchment. They are not necessarily a function of the road reserve or pavement width. Designers of drainage systems in existing areas are urged to clarify such issues with the local authority prior to design.

Relevant flow width limitations are contained in tables 7.4.1 and 7.4.2, and Figure 7.4.1. Flow width should be limited by whichever of the limitations in tables 7.4.1 and 7.4.2 is the more restrictive. The designer's attention is also drawn to the requirements of Table 7.3.1 in respect of design AEPs for kerb and channel flow.

(a) At bus stop

(b) At kerb return

(c) At deceleration lane
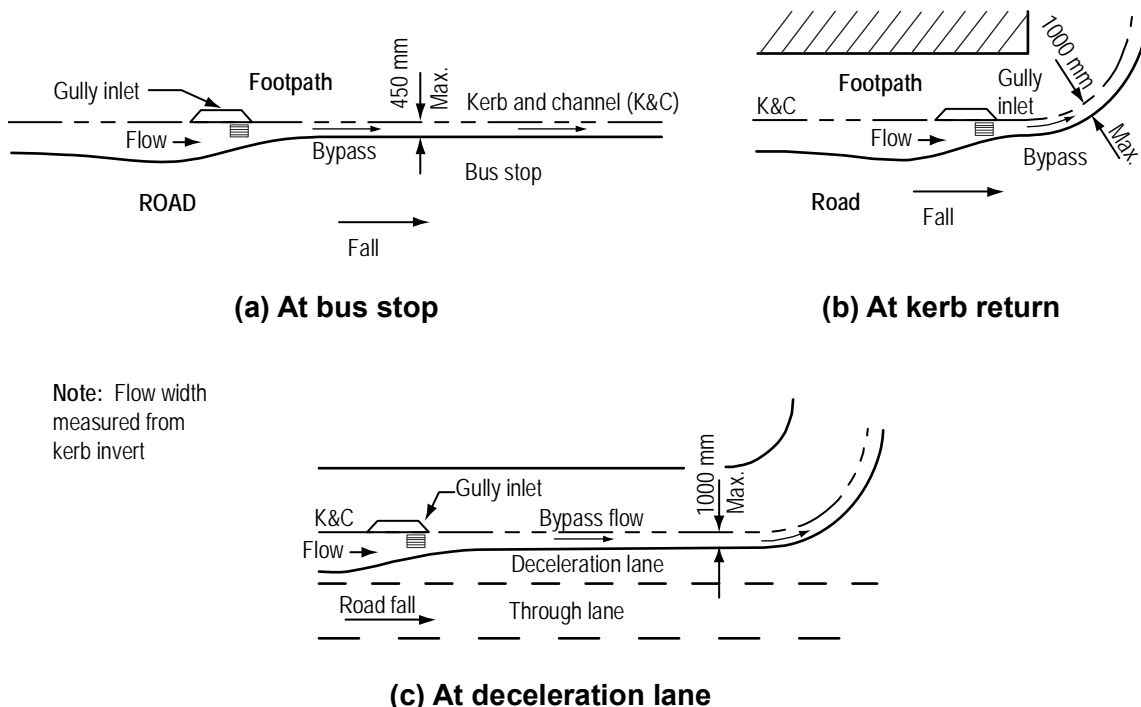
Figure 7.4.1 – Typical flow width criteria (minor storm)

Note (Figure 7.4.1): Flow width is measured from kerb face.

Table 7.4.1 – Roadway flow width [1] limitations during MINOR STORM for 'longitudinal' flow

Site condition	Major road	Minor road
Normal situation	Parking lane width (usually 2.5 m) or breakdown lane width [2]	(i) Full pavement width with zero depth at crown (ii) Where one way crossfall exists (i.e. no crown), flow width is limited to the high side of road pavement, but not above top of kerb on low side
Where parking lane may become an acceleration, deceleration or turn lane	1.0 m	Not applicable
Where road falls towards median	1.0 m	Not applicable
Pedestrian crossings or bus stops	0.45 m	0.45 m
At intersection kerb returns (including entrances to shopping centres and other major developments)	1.0 m [3] [4]	1.0 m [3] [4]
Vehicular safety	Flow depth and depth*velocity limits as per major storm	

Notes (Table 7.4.1):

[1] Widths are measured from channel invert for kerb and channel, and from kerb face for kerb only.

[2] It may be necessary to limit discharge to 0.03 m^3/s upstream of small radius bends (less than 15 m radius) to avoid flooding and traffic safety issues.

[3] Where flow is required to follow a kerb return at an intersection it may be necessary, where the longitudinal grade is steep, to check for the effect of flow superelevation upon flow spread. A procedure for the calculation of superelevation is given in section 9.3.6(c).

[4] When considering the 1.0 m flow spread limitation at a kerb return the effect of the reduced pavement crossfall beyond the tangent point should be examined.

Table 7.4.2 – Roadway flow depth and velocity limitations during MINOR STORM for 'transverse' flow

Site condition	Flow depth and width limits
Still water at road sag	• Maximum flow depth, $d_g \leq 300$ mm
Vehicle safety: Transverse flow limits (no risk to life) e.g. road intersection	• Maximum flow depth, $d_g \leq 300$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.3$ m^2/s
Vehicle safety: Transverse flow limits (risk to life) e.g. causeway	• Maximum flow depth, $d_g \leq 200$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.3$ m^2/s

Table 7.4.3 – Roadway flow depth and velocity limitations during MAJOR STORM for 'longitudinal' flow

Site condition	Flow depth and width limits
Where floor levels of adjacent buildings are above road level	• Total flow is contained within road reserve • Minimum freeboard of 300 mm to floor level of adjacent buildings
Where floor levels of existing adjacent buildings are below, or less than 300 mm above, the top of kerb; and there is **at least** 100 mm fall on footpath towards the kerb	• Maximum flow depth of 50 mm above top of kerb
Where floor levels of existing adjacent buildings are below, or less than 300 mm above, the top of kerb; and there is **less than** 100 mm fall on footpath towards the kerb	• Maximum flow depth at top of kerb
Vehicle safety: Flow conditions at kerb for flow along a road (no risk to life)	• Maximum flow depth, $d_g \leq 250$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.6$ m²/s
Vehicle safety: Flow conditions at kerb for flow along kerb (potential risk to life)	• Maximum flow depth, $d_g \leq 250$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.4$ m²/s

Table 7.4.4 – Roadway flow depth and velocity limitations during MAJOR STORM for 'transverse' flow

Site condition	Flow depth and width limits
Still water at road sag	• Maximum flow depth, $d_g \leq 300$ mm
Vehicle safety: Transverse flow limits (no risk to life) e.g. road intersection	• Maximum flow depth, $d_g \leq 300$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.45$ m²/s
Vehicle safety: Transverse flow limits (risk to life) e.g. causeway	• Maximum flow depth, $d_g \leq 200$ mm • Depth*velocity product, $d_g.V_{ave} \leq 0.3$ m²/s

7.4.2   General requirements

(a)   Pedestrian safety

The depth*velocity product is currently recommended as the best design measure for pedestrian safety within shallow-water overland flow paths. Recent reports (Engineers Australia, 2010, 2011a) highlight that for some people, notably small children and frail older persons; there are **no** depth or velocity limitations that can be considered safe in all circumstances.

The product of depth d_g and velocity V_{ave} in the kerb and channel should not exceed 0.6 m²/s (Engineers Australia, 2010) to reduce hazard for pedestrians within the roadway. However, where there is an obvious risk of serious injury or loss of life, the $d_g . V_{ave}$ product should be limited to a value of 0.4 m²/s. This is applicable to 'longitudinal flow' along the roadway for both major and minor design storms.

An *'obvious risk of serious injury or loss of life'* would include:

- Upstream of kerb inlets or any stormwater/pipe inlet with a clear opening greater than 90 to 125 mm (at the discretion of the local authority—refer to section 7.5.3(f)) where there is a risk to life resulting from small child entry into the downstream stormwater system.

- Overland flow paths passing through, or discharging into, flow conditions defined in section 12.2 for Contact Classes A to D.

No definitive depth*velocity limitations can be specified for stormwater flow within childcare centres or areas frequented by elderly persons such as hospitals and retirement villages. Local governments should treat all situations on a case-by-case basis. Children with a height*mass product less than 20 m.kg are generally of greatest risk.

(b) Management of supercritical flows along roadways

On steep slopes, surface flows passing down roadways can become supercritical. In such cases, there is a high potential for these surface flows to spill from the kerb and channel, and cross the road surface. Road designers should avoid sharp changes in road direction where such designs could cause surface flows to spill across the road and cause traffic safety issues, or cause stormwater to spill into adjacent properties.

Such issues are of particular concern where:

- the roadway passes down a ridge line and the adjacent properties and/or buildings are below the elevation of the road surface
- T-junctions placed at the base of, or on the side of, steep slopes
- circumstances where traffic calming devices are placed on steep roads and such features could cause road runoff to be deflected into adjacent driveways or could cause the formation of large standing waves.

(c) Major flows at T-Junctions

Care should be taken in the design of surface flows at road T-Junctions adjacent steep hill slopes. In cases where the surface water enters a T-Junction via a steep gradient roadway, the high-velocity, supercritical surface flow may fail to follow the desired flow path through the intersection. In the worst case scenario, the flow can pass across the road junction—causing a traffic safety hazard—before entering a down-slope property potentially causing flooding and property damage.

(d) Flow capacity calculation for roadways with kerb and channel

Roadway flow capacity may be calculated using Izzard's equation (refer to Technical note 4, Book 8, ARR-1998). The values outlined in Table 7.4.5 are recommended for Manning's roughness coefficient (n) and Flow Correction Factor (F).

Izzard's Equation provides a solution to flow determination in a triangular channel as follows:

$$Q = 0.375\ F.(Z/n).S^{0.5}.d^{2.667} \tag{7.2}$$

For composite flow, as in a half road where the pavement and channel have different roughness and crossfall, the equation becomes:

$$Q = 0.375\ F.\ [(Z_g/n_g).(d_g^{2.667} - d_p^{2.667}) + (Z_p/n_p).(d_p^{2.667} - d_c^{2.667})].S^{0.5} \tag{7.3}$$
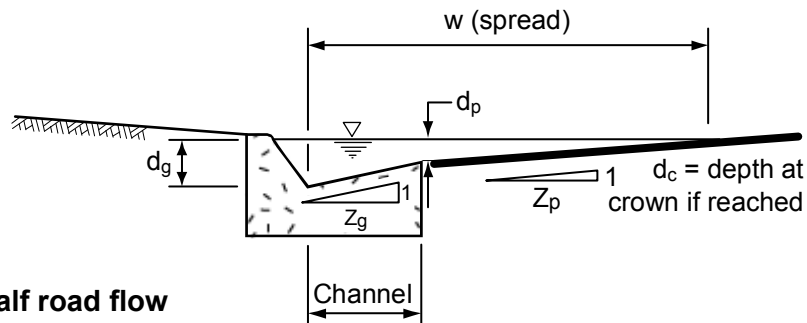
Figure 7.4.2 – Half road flow

where:

Q	=	Longitudinal flow down kerb (m3/s)
F	=	Flow Correction Factor
Z	=	Cross slope gradient
Z_g	=	Cross slope gradient of kerb
Z_p	=	Cross slope gradient of pavement
n	=	Manning's roughness
n_g	=	Manning's roughness of kerb
n_p	=	Manning's roughness of pavement
S	=	longitudinal slope of kerb
d	=	maximum depth of flow
d_g	=	depth of flow at kerb invert
d_p	=	depth of flow at edge of pavement
d_c	=	depth of flow at crown

Table 7.4.5 – Recommended values of Manning's roughness coefficient and flow correction factor for use in Izzard's equation [1]

Surface type	*n*
Concrete	0.013
Hot mix asphaltic concrete	0.015
Sprayed seal	0.018
Kerb and channel type	**F**
Semi-mountable type	0.9
Barrier type (300 mm channel)	0.9
Barrier type (450 mm channel)	0.9

Note (Table 7.4.5):

[1]   No recommendation is given in respect of the roughness on footpaths, it being normal practice to exclude the flow on the footpaths because of the likely presence of utility poles, landscaping etc.

(e)   Resurfacing allowance

It is recommended that consideration be given to the effect of future resurfacing of roadways. Where such provision is to be included, allowance for a standard 25 mm (asphaltic concrete) resurfacing is recommended unless directed otherwise by the local government.

Note: *That the construction of a 25 mm thick asphaltic concrete overlay can reduce the waterway area to 45 to 65 percent of that available prior to overlay for the same depth at invert. Some increase in flow depth for the same flow must inevitably occur following an overlay.*

7.5 Stormwater inlets

7.5.1 Types of stormwater inlets

The types of stormwater inlets discussed in this section include kerb inlets (section 7.5.3), field inlets (section 7.5.4) and open pipe inlets (section 7.5.5).

7.5.2 Provision for blockage

Local authorities may indicate the percentage of blockage that is to be applied to the theoretical inflow capacity of inlets.

Where such guidance is not provided, the recommendations in Table 7.5.1 should be adopted. Where the invert of the kerb is depressed at the inlet the capacity of the inlet should be adjusted accordingly.

Table 7.5.1 – Provision for blockage at kerb inlets

Inlet type	Blockage factor	
	Design value	**Severe conditions [1]**
Sag kerb inlets:		
Kerb inlet	20%	100 %
Grated	50 %	100%
Combination	[2]	100%
Continuous (on-grade) kerb inlets:		
Kerb inlet	20%	100%
Longitudinal bar grated	40%	100%
Transverse bar grate or longitudinal bar grate incorporating transverse bars	50%	100%
Combination	[3]	100%
Field (drop) inlets:		
Flush mounted	80%	100%
Elevated (pill box) horizontal grate	50%	100%
Dome screen	50%	100%
Open pipe inlets (blockage factors as per culverts)	Refer to Table 10.4.1	

Notes (Table 7.5.1):

[1] The likelihood of severe blockage should be considered during the assessment of the impacts of severe storms. Such blockage conditions should only be considered in circumstances where suitable blockage material (i.e. 'bridging' material) exists within the drainage catchment.

[2] At a sag, the capacity of a combination inlet (kerb inlet with grate) should be taken to be the theoretical capacity of the kerb opening with 100% blockage of the grate.

[3] On a continuous grade the capacity of a combination inlet should be taken to be 90% of the combined theoretical zero-blockage capacity of the grate plus kerb opening.

This Manual does not include inflow capacity charts for kerb inlets. These charts should be obtained from the relevant local authority. Such charts should reflect the theoretical or measured capacity of the inlet, to which the above percentages should be applied to allow for blockage.

7.5.3 Kerb inlets in roads

(a) Kerb inlet types

Four types of kerb inlets are in common use, they are:

- Grate only—for example, field inlets and anti-pooling gullies on kerb returns.

- Side inlet—these inlets rely on the ability of the opening under the backstone or lintel to capture flow. They are usually depressed at the invert of the channel to improve capture capacity.

- Combination grate and side inlet—these inlets utilise the backstone arrangement of the side inlet with the added capacity of a grate in the channel.

- Special site specific designs for high inflow.

Local authorities may determine appropriate kerb inlet types for a particular installation and should make available relevant standard drawings showing dimensions and set out details along with inlet capacity charts for those inlets.

(b) Location of kerb inlets

Kerb inlets should be provided at the following locations in kerb and channel:

(i)   In the low points of all sags in kerb and channel.

(ii)   On grades, to ensure compliance with the flow width limitations discussed in section 7.4.

(iii)   At the tangent point of kerb returns or small radius convex curves (kerb radius less than 15 m) such that the flow width around the kerb return (i.e. beyond the kerb inlet) during the minor design storm does not exceed 1.0 m measured from the invert of kerb and channel. This limitation will also be applicable at important vehicular turnouts or footpath crossovers, where high traffic volumes are anticipated, such as at entrances to shopping centres.

(iv)   Immediately upstream of potential pedestrian crossing and bus stops such that the flow width does not exceed 450 mm from invert of kerb and channel during the minor design storm.

(v)   Immediately upstream of any reverse crossfall pavement to prevent flow across the road during the minor design storm (i.e. at the start of crossfall transition from normal to reverse crossfall).

(vi)   Where superelevation or reverse crossfall results in flow against traffic islands and medians. Kerb inlets shall be provided along the length of the island or median as necessary to meet the flow width limitations as stated in section 7.4 and at the downstream end of the island or median to minimise the flow continuing along the road (see also (vii) below). Where sufficient width of island or median is available, grated kerb inlets should be recessed so that the grate does not project onto the road pavement. Alternatively side entry inlets with no grate should be installed.

(vii)   Where reverse crossfall on a road pavement causes flow onto the pavement. The extent to which such flow onto the pavement is permissible depends upon the catchment area involved and the risk of vehicle aquaplaning. For guidance on the management of aquaplaning, refer to the latest version of Queensland's Main Roads' 'Road Drainage Design Manual'.

(viii) Where it is anticipated that a parking lane may become an acceleration, deceleration or turn lane in accordance with Table 7.4.1.

(ix)   Consideration should be given to the positioning of kerb inlets relative to the side property boundaries. In residential and industrial locations, a kerb inlet located near the side property

boundary may cause difficulties with driveway access. In commercial areas and those where there is likely to be a high volume of pedestrian traffic, kerb inlets should be located to avoid set down points or locations where pedestrian movements are likely to be highest.

(c) Kerb inlets on grade

The procedure detailed in Figure 7.5.1 is recommended for determining the location of kerb inlets on-grade.

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                    ┌─────────────────────┐
                    │   SELECT TRIAL      │
                    │ GULLY INLET LOCATION│
                    └─────────────────────┘
                              │
                              ▼
                    ╭─────────────────────╮
     ┌──────────────│ DETERMINE ROAD FLOW │◄──────────────┐
     │              │   WIDTH LIMITATION  │               │
     │              │     APPLICABLE      │               │
     │              ╰─────────────────────╯               │
     │                        │                           │
     │                        ▼                           │
     │              ╭─────────────────────╮               │
     │              │DETERMINE LONGITUDINAL│              │
     │              │     ROAD GRADE      │               │
     │              │ AT SELECTED LOCATION│               │
     │              ╰─────────────────────╯               │
     │                        │                           │
┌──────────────┐    ╭─────────────────────╮    ┌──────────────┐
│MOVE SELECTED │    │CALCULATE THEORETICAL│    │MOVE SELECTED │
│LOCATION      │    │FLOW CAPACITY OF ROAD│    │LOCATION      │
│DOWNSTREAM TO │    │     (ONE SIDE)      │    │UPSTREAM TO   │
│INCREASE      │    ╰─────────────────────╯    │REDUCE        │
│CATCHMENT AREA│              │                 │CATCHMENT AREA│
│& FLOW        │              ▼                 │& FLOW        │
└──────────────┘    ┌─────────────────────┐    └──────────────┘
     ▲              │REDUCE FLOW CAPACITY TO│        ▲
     │              │  PRACTICAL CAPACITY  │         │
     │              │     IF DIRECTED     │         │
     │              │e.g. 80% OF THEORETICAL│       │
     │              └─────────────────────┘         │
     │                        │                     │
  ╭──────────╮      ╭─────────────────────╮    ╭──────────╮
  │FLOW TOO  │◄─────│   CHECK FLOW FROM   │───►│FLOW TOO  │
  │LITTLE    │      │ CATCHMENT UPSTREAM OF│    │GREAT     │
  ╰──────────╯      │    CRITICAL POINT   │    ╰──────────╯
                    ╰─────────────────────╯
                              │
                              ▼
                    ╭─────────────────────╮
                    │  FLOW AND PRACTICAL │
                    │ CAPACITY COMPARABLE │
                    ╰─────────────────────╯
                              │
                              ▼
                    ┌─────────────────────┐
                    │   ADOPT THIS LOCATION│
                    └─────────────────────┘
```

Figure 7.5.1 – Flow chart for determining kerb inlet positions on-grade

Notes (Figure 7.5.1):

[1]   Changes in catchment area may result in changes in time of concentration for a catchment.

[2]   The above procedure is iterative.

[3]   Selection of the initial trial kerb inlet location may be based on changes in road grade (e.g. steep to flat), physical restrictions in road (e.g. median or Residential Street Management devices), or by driveways, entrances or intersections etc.

Designers should be aware that kerb inlet capacity is controlled by the crossfall of the road pavement and the longitudinal grade. Also, the bypass flow from a kerb inlet must be accounted for in the design of the downstream kerb inlet that receives the bypass flow. There is no limitation to the amount of flow that may be bypassed from a kerb inlet provided that the flow width criteria discussed in section 7.4 are adhered to.

Note: That a number of road flow capacity calculations may be required, using actual crossfalls at the intersection, to check that all bypass flows are contained within the 1.0 m flow width limitation at kerb returns, under minor storm conditions.

Where bypass flow from a kerb inlet is required to follow a kerb return at an intersection, it may be necessary, where the longitudinal grade is steep, to check for the effect of flow superelevation upon flow spread. A procedure for the calculation of superelevation is given in equation 9.3.6(c).

(d)   Kerb inlets in sags

Kerb inlets in sags must have sufficient inflow capacity to accept the total flow (including bypass flows from upstream) reaching the inlet. Pooling of water at sag inlets should be limited to the widths discussed in section 7.4 particularly at intersections where turning traffic is likely to encounter ponding.

Where the longitudinal grades on either side of the sag are different, or where the flow from one direction is dominant, the location of the effective sag may move from the true sag and a hydraulic jump may form beyond the sag. Care should be taken, by the provision of extended or additional inlets, to ensure that capture capacity is maintained and that the water level does not cause flow over the footpath into the adjacent property. A procedure for checking whether this effect is occurring has been proposed by Black (1987a) and is detailed in figures 7.5.2 and 7.5.3.
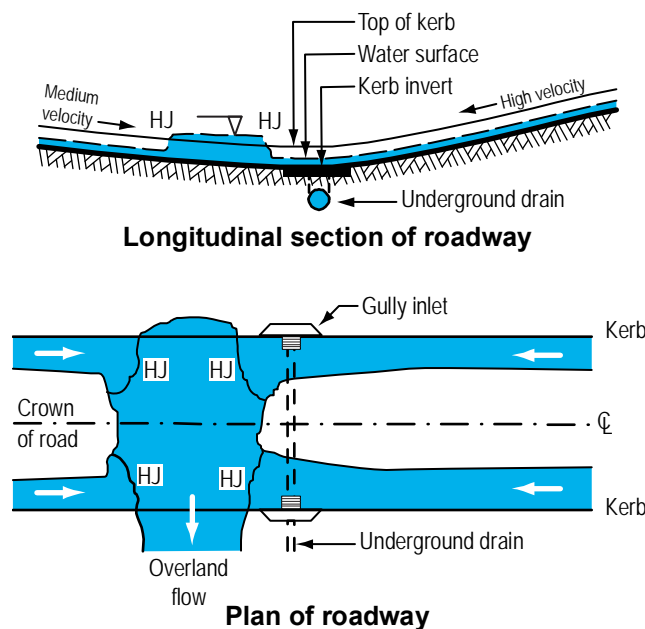
Figure 7.5.2 – A sag in a road with supercritical approach flows ('HJ' indicates a hydraulic jump)
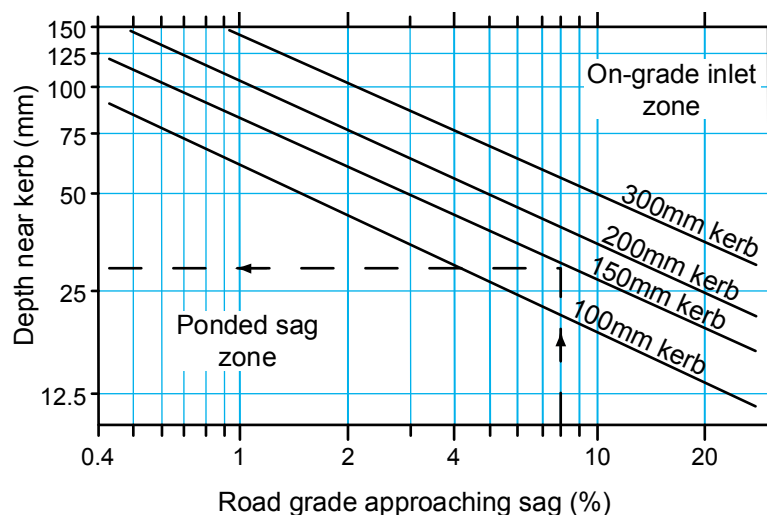
Figure 7.5.3 – Limiting condition for a sag inlet to act as an on-grade inlet (n = 0.013) (Source: Black, 1987a)

Note to Figure 7.5.3: For example, for a kerb height = 150 mm and approach slope = 8%, inlet on grade conditions will apply for flow depths near the kerb > 30 mm (i.e. pooling may exceed kerb height after hydraulic jump unless kerb inlets are extended towards the flatter side of the sag).

Technical note 7.5.1 A user guide to Figures 7.5.2 and 7.5.3

1. Determine the effective longitudinal slope of the kerb and channel on each side of the sag assuming that the established velocity on the approach slope will persist into the vertical curve forming the sag. Normally the effective longitudinal slope will be the slope on the tangent.

2. Determine the flow depth d_{p1} on each side of the sag.

3. Enter the chart (Figure 7.5.3) for the value of effective longitudinal slope for each side of the sag and the appropriate kerb height and determine the two values of d_{p2}.

4. Assess the total flow approaching the sag inlet and determine the head required to permit the full flow to enter the sag inlet. It is assumed that the inlet capacity has been selected such that the head required is no greater than the maximum permitted flow depth (see Table 7.4.2).

5. If all of the following conditions are met, the inlet is likely to operate without the pond overtopping the kerb. No further calculation is required.

   • $d_{p1} < d_{p2}$ in both directions.

   • The approach slope from each direction is similar, and the head required to permit the full flow to enter the sag inlet is less than the maximum permitted flow depth.

6. If any one of the conditions in point 5 (above) cannot be met, on-grade inlet conditions are likely to occur, inlet capacity is likely to be reduced and property flooding may occur. In this case the approach flows to the sag need to be reduced by the installation of gully inlets upstream of the sag.

Note: The above procedure has been based upon theoretical analysis only and has not been verified by testing. Designers should therefore exercise appropriate care in using the procedure.

Terminology:

d_{p1} = Flow depth at pavement edge (lip) upstream of sag (determined from road flow capacity charts).

d_{p2} = Limiting flow depth at pavement edge upstream of sag (determined from Figure 7.5.3).

(e) Intersections

Consideration needs to be given to the steepness of grade of the road and the possibility of momentum carrying water past the stormwater inlet/s, across the road and into properties opposite the intersection. Solutions to such problems may require extra inlets to be installed. Also refer to the discussion in section 7.4.2 (c).

Where two falling grades meet at an intersection, every endeavour should be made to locate the low point of the kerb and channel at one of the tangent points of the kerb return.

Where both grades are steep it may not be practicable to locate the low point at a tangent point. In this case, kerb inlets should be provided at both tangent points, with additional inlets provided upstream of the tangent points, if necessary, designed to limit the flow width beyond the kerb return. An anti-pooling kerb inlet (grate only) installed within the width of the channel—nominally 450 mm long by 300 mm wide with no kerb inlet should be provided at the low point.

The location of a kerb inlet, or a grated inlet that protrudes onto the pavement within a kerb return is considered unsatisfactory because of the risk of damage by and to vehicles.

(f) Safety issues

In locations where the kerb inlet is accessible by a small child, whether deliberate or as a result of a child being swept down the flooded kerb, then the maximum clear opening height for a kerb inlet shall not exceed 125 mm.

Local authorities may choose to reduce this maximum clear opening to 90 or 100 mm if the increased risk resulting from a 125 mm opening is considered unacceptable (refer to Technical note 7.5.2).

Technical note 7.5.2

Considerable debate exists regarding the recommended maximum clear opening for kerb inlets to provide safety for small children. Even though past history has shown the likelihood to be low, the consequences of a child being swept down a flooded kerb and into a stormwater inlet can be extreme.

After consideration of the various arguments presented to the QUDM Reference Group (2007), the recommendation for 125 mm maximum clear opening was accepted. However, the 125 mm opening still presents a risk of a small children partially entering (i.e. feet first) the inlet.

A maximum clear opening of 90 mm is recommended where it is necessary to exclude the entry of the torso of a 2-year-old child. Such consideration may apply in parks, schools and childcare centres.

7.5.4 Field inlets

Field inlets (also known as drop inlets) should be provided in parks, footpaths, medians, etc. to drain all low points. Field inlets can be provided within allotments for two reasons: to provide a disconnection between the roof and street drainage (i.e. WSUD) and to drain low points (in accordance with section 7.13).

Where there is considerable pedestrian traffic adjacent to a field inlet (e.g. in a footpath) a grate with close bar spacing should be used—recommended bar spacing are provided in section (d) below. Elsewhere a grate with wide bar spacing is preferable, because of the reduced risk of debris blockage.

Table 7.5.1 (section 7.5.2) provides suggested blockage factors for field inlets.

(a)   Inflow capacity

The inflow capacity of a field inlet depends upon the depth of water over the inlet. For shallow depths the flow will behave as for a sharp crested weir. At greater depths the inlet will become submerged and inflow will behave as for an orifice.

Orifice flow can either be represented by free flow conditions (atmospheric pressure within the chamber) or fully drowned conditions (non atmospheric). It is recommended that the capacity of the inlet be checked using both weir flow and orifice flow procedures and the lesser inlet capacity adopted.

(i)   Under weir flow conditions:

$$Q_g = BF \times 1.66 \, L.h^{3/2} \tag{7.4}$$

where:

Q_g = flow into field inlet (m^3/s)
BF = blockage factor = 0.5
1.66 = weir coefficient
L = weir length (m) (see note below)
h = depth of water upstream of inlet (relative to weir crest) where flow velocity is low (i.e. velocity head is insignificant) otherwise use the height of energy level above the weir crest (m)
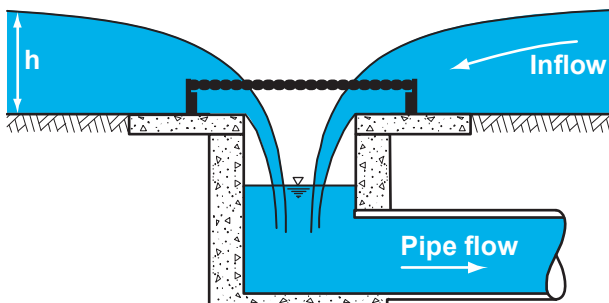
Note: The length referred to in this case is the effective weir length. Thus for a grated inlet adjacent to a kerb, the side along the kerb should be ignored. For a side inlet the length referred to is the length of the inlet.

Figure 7.5.4 – Field inlet operating under weir flow

(ii)   Under orifice flow conditions:

The orifice flow equation depends on the pressure gradient across the orifice. The standard orifice flow equation applies when atmospheric pressure conditions exist downstream of the grate, such as would exist if the design Water Surface Elevation (WSE) is 150 mm below the grate (as per Table 7.16.1 and Figure 7.5.5). The same equation can be used for a fully drowned conditions when non atmospheric conditions exists within the chamber, but the head 'h' is now represented by the total energy loss of the orifice.

Equation 7.5 is based upon a pressure change coefficient of K_g = 2.75.

$$Q_g = BF. \times 0.60 \, A_g .(2g.h)^{1/2} \tag{7.5}$$

where:

Q_g = flow into field inlet (m³/s)

BF = blockage factor = 0.5

A_g = clear opening area of grate (m²)

h = depth of approaching water relative to the orifice (m) for free (atmospheric)

= total energy loss through the orifice (m) for fully drowned (non atmospheric)

g = acceleration due to gravity (9.80 m/s²)

0.60 = constant = $(1/K_g)^{1/2}$ = $(1/2.75)^{1/2}$

K_g = pressure change coefficient for the grate

The pressure change coefficient (K_g) can vary significantly for unusual grate designs. The coefficient used in equation 7.5 is based on a typical open mesh grate. It is noted that the pressure change coefficient for the old cast iron 'City Grate' has been adopted as 2.23. Designers of unusual hydraulic structures should seek expert advice or review appropriate reference documents on orifice flow.

If the field inlet is fully drowned (i.e. no air gap exists below the grate and thus the hydraulic pressure below the grate is not atmospheric) then an estimate must be made of the head loss through the structures as per a normal Hydraulic Grade Line (HGL) analysis. Such calculations require considerable experience and hydraulic judgement. Guidance on head losses through screens is provided in sections 7.16.14(c) and 12.5.6.
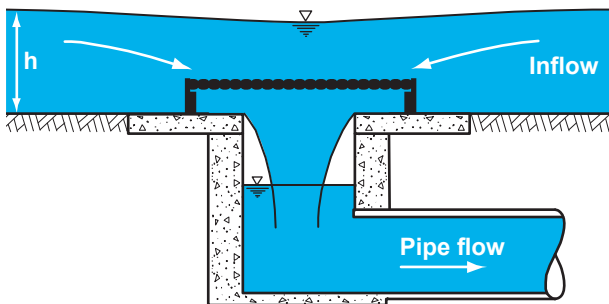
Figure 7.5.5 – Field inlet operating under free (atmospheric) orifice flow
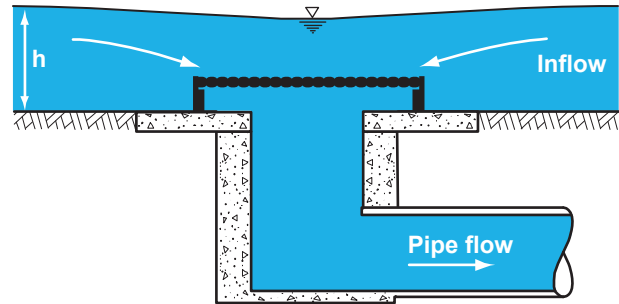
Figure 7.5.6 – Field inlet operating under fully drowned (non-atmospheric) conditions

(b)　Freeboard considerations

Freeboard provisions should be made at field inlets as follows:

- Where the inlet is contained within a pond formed by earth mounds or similar, freeboard should be 20% of the depth of the pond with a minimum of 50 mm under minor storm conditions. However where overflow must be avoided the design storm shall be the major storm event.

- Where flooding of buildings is possible freeboard provision should be in accordance with section 7.3 for the major storm event.

(c)　Minimum width of scour protection lip

Field inlets placed in grassed or open soil areas should be surrounded by a suitable scour protection lip. The concrete lip formed around a field inlet should have sufficient width to:

- minimise the risk of grass growing over the grate, or causing blockage of the grate

- prevent scour of an adjoining surface.

Unless otherwise supported by site specific hydraulic calculations, the minimum recommended lip width (Z) required to minimise the risk of scour within the adjoining 'grassed' surfaces may be determined from equation 7.6.

$$Z = 2.3\,A_g/L \qquad\qquad (7.6)$$

where:

Z = minimum lip width for scour protection (m)
A_g = effective clear opening area of drop inlet (m^2)
L = total internal circumference of drop inlet (m)

Thus, for square inlets ($A_g = y^2$ and $L = 4y$) the minimum lip width: $Z = 0.57y$

where:

y = internal side dimension of square drop inlet (m)
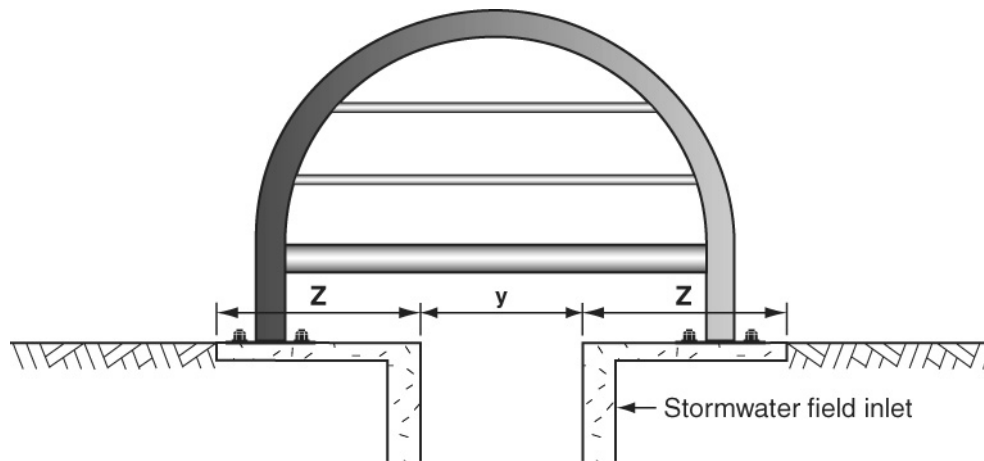
Figure 7.5.7 – Minimum lip width required for scour protection (dome inlet screen shown as example only)

(d)   Safety issues

Safety risks should be reviewed in circumstances where a field inlet is located within areas accessible to the public. The primary concern is for those circumstances where a child could be swept up against the screen **and** the water level could rise above the child's head. Safety considerations include the following:

- Safety risks associated with people tripping over the screen (i.e. if not set flush with the ground).

- Inlet screens located in vehicular or pedestrian areas shall comply with the requirements of AS 3996.

- If there is the risk of a child being swept by stormwater towards a horizontal inlet screen, then the maximum clear spacing of the bars shall be 90 mm.

- If there is the risk of a child being swept by stormwater towards a vertical or inclined inlet screen, then the maximum clear spacing of the bars shall be 125 mm.

- Maximum clear bar spacing of 89 mm if located within a park or playground (AS4685.1 Playgrounds and Playground Equipment), otherwise a maximum spacing of 125 mm.

- Flow velocities through the screen/grate sufficiently low to prevent a child from being held against the screen/grate by hydraulic pressure. It is recommended that the maximum flow velocity through the grate/screen should be 1 m/s.

Raised, horizontal screens are generally not acceptable adjacent footpaths, bikeways or public areas where significant numbers of people gather as these inlets may represent an unacceptable safety risk. In such circumstances, flush screens should be used, or possibly large dome screens if such screens are likely to be clearly visible and not represent a safety risk. Alternatively, marker posts or fencing may be used.

7.5.5   Open pipe inlets

Design conditions at large open pipe inlets often mimic the design rules for pipe culverts. Recommended debris blockage factors are the same as those provided for culverts (refer to Table 10.4.1).

Guidance on the design of inlet debris screens can be found in Table 5.8.1 (*Criteria for basin outlet* structures) and guidance on safety screens is provided in section 12.5.

7.6   Access chambers

7.6.1   General

Access chambers should be provided on drainlines:

* to provide access for maintenance
* at changes of direction, grade or level
* at junctions.

Consideration should be given to the placement of an access chamber at an obstruction or penetration by a conduit or service, to facilitate the removal of debris.

The maximum recommended spacing is given in Table 7.6.1.

Table 7.6.1 – Recommended maximum spacing of access chambers

Condition	Pipe size (mm)	Spacing (m)
Generally	Less than 1200	100
	1200 and above	150
Immediately upstream of outlet to tidal waterway	All	100
Roadways	All	200

The local authority may direct that a standard access chamber should be used and may make available standard drawings for these installations. However for multiple pipes, large diameter pipes, or odd configurations of pipes, it may be necessary to design a special chamber. Special chambers should be designed to accept the loadings detailed in section 7.9 of this Manual.

Benching of the floors of access chambers leads to a general reduction in losses and promotes improved hydraulic efficiency (Johnston et al. 1990).

Technical note 7.6.1 Benefits of benching junction chambers:

Benching does not necessarily help to align incoming flows with the outlet pipe. Instead, benching works by reducing the effects of flow expansion adjacent the base of the access chamber. The higher the benching and the more it removes effective deadwater zones around the base of the chamber, the more effective the reduction in losses (refer to discussion in section 7.16.8(b)).

Hydraulic improvements are difficult to quantify and the construction of benching can be costly. Benching is therefore recommended only when it is important to minimise losses. Further information is provided in section 7.16.8 (b).

Some local authorities exclude or limit the use of precast access chambers and designers should check that they are acceptable. In cases where precast chambers are used, the connecting stormwater pipes should not protrude into the chamber and should be sealed and finished in accordance with an approved construction detail.

The geometry of pipes at access chambers is critical in respect of hydraulic head loss. This matter is discussed further throughout section 7.16. The main principles to be followed to minimise head loss are:

- Minimise changes in flow velocity through the chamber.

- Minimise changes in flow direction.

- Avoid opposed lateral inflows, i.e. all incoming pipes should ideally be contained within a 90 degree arc, but certainly less than 180 degrees.

- Limit the deflection from inflow to outflow for pipes smaller than 600 mm diameter to 90 degrees, or 67.5 degrees for pipes 600 mm and greater in diameter.

- Avoid vertical misalignment, i.e. drop pits; unless deliberately intending to induce high head loss.

- Where practical, direct inlet pipes wholly into the barrel of the outlet pipe (Figure 7.6.2). It is noted that for various reasons, inflow pipes often need to be directed towards the centre of the pit (Figure 7.6.1) however, this will increase losses.
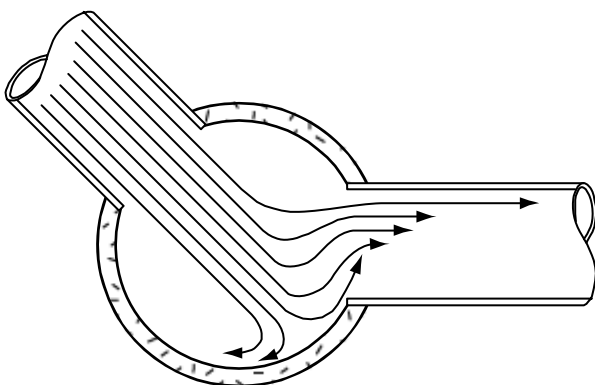
Figure 7.6.1 – Flow lines resulting from inflow pipe directed at pit centre
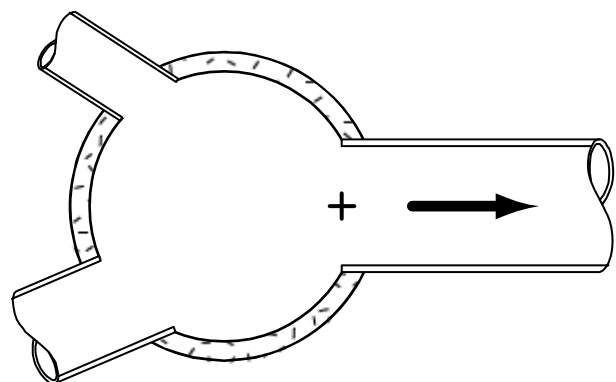
Figure 7.6.2 – Inflow pipe directed at centre of outflow pipe

- Rounding the entrance to the outlet pipe at a radius of one-twelfth of the outlet diameter will help to reduce losses (Figure 7.6.3).

- Where practical, the change of direction of flow should occur at or near the downstream face of the chamber.

- Head losses resulting from surface inflows (Figure 7.6.4) are reduced if the design water level in the chamber is well above the obvert of the outlet pipe.
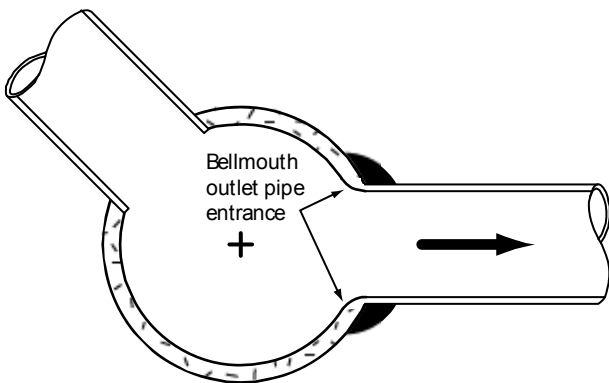
Figure 7.6.3 – Bellmouth entrance to outlet pipe
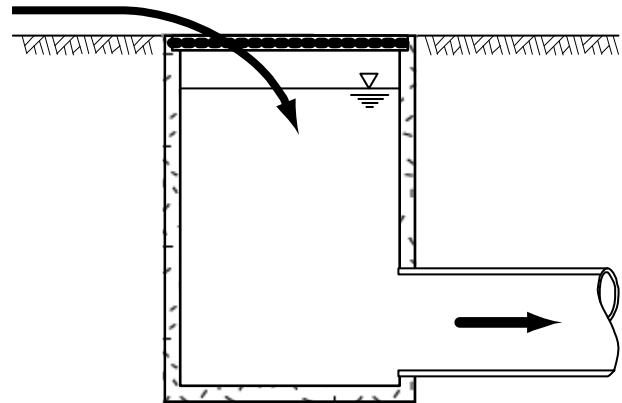
Figure 7.6.4 – Inlet chamber showing water level well above outlet obvert

7.6.2   Access chamber tops

Access chambers in a carriageway or paved surface should be finished with their tops flush with the finished surface. Where an access chamber is located within a carriageway, the chamber top, or access point, should be positioned to avoid wheel paths.

Elsewhere, access chambers should be finished 25 mm above natural surface with the topsoil or grassed surface around the chamber graded gently away. On playing fields they may be finished 200 mm below the finished level, but only when located in a straight line between two permanently accessible chambers.

7.6.3   Deflection of pipe joints, splayed joints etc

Changes of direction for drainage lines of 1200 mm diameter or greater may be achieved by deflection of pipe joints, the use of splayed joints or fabricated bends.

The recommended radius of curvature for pipes with deflected joints or splayed units should be as agreed with the relevant local authority in consultation with the pipe manufacturer.

Plans showing curved stormwater lines should show the radius of curvature, the total deflection angle, the maximum deflection per pipe length, the length of pipes and the joint type.

7.6.4   Reduction in pipe size

For single drainage lines, a downstream pipe of smaller diameter than the upstream pipe may be permitted as long as the system works hydraulically and as long as the change in diameter is no greater than the following:

Table 7.6.2 – Recommended maximum reduction in pipe size – SINGLE PIPES

Upstream pipe diameter (mm)	Allowable change in diameter
Less than 600	No change
675 to 1200	One pipe size
Greater than 1200	Two pipe sizes

The above recommendations are based upon the nominal sizes of pipes as manufactured in accordance with AS 4058.

At the location where the reduction in size occurs, pipes should be graded invert to invert to prevent the accumulation of sediment etc.

7.6.5 Surcharge chambers

Prior to incorporating a surcharge chamber into a drainage line, the following should be considered:

- The potential for a person (that has been swept into the upstream drainage system) being trapped inside the surcharge chamber unable to exit the chamber or the outlet pipe.

- Potential surcharge of the upstream system and flooding problems caused by debris blockage of the outlet screen.

- Structural integrity of the chamber, outlet screen, top slab and concrete coping, and its ability to withstand high outflow velocities and high-pressure forces caused by debris blockages. There is a need in many cases to ensure the surcharge screen is securely anchored to the top slab, and the slab to the chamber walls, to avoid displacement of the chamber lid/screen.

- Safe maintenance access to allow removal of debris trapped within the surcharge chamber.

The hydraulic analysis of surcharge chambers is presented in section 7.16.14.

7.7 Pipeline location

Minor pipes connecting one kerb inlet to another is acceptable at the top of the street drainage system. These pipes may be located under the kerb and channel.

For pipelines greater than 600 mm it is recommended that the location for drainage lines in the road pavement—other than a kerb inlet to kerb inlet connection—be 2.0 m measured towards the road centreline from the invert of the kerb and channel. The required location should be verified with the local government. Access chamber tops or access points should be located to avoid wheel paths.

Where sufficient verge width is available stormwater pipes may be located in the verge to suit the services allocations of the relevant local government.

In divided roads, drainage pipelines may be located within the median, normally offset 1.5 m from the centreline (as street lighting poles are normally on the centreline).

If reasonable alternative locations are available drainage pipelines should not be located within allotments. In many cases overland flow requirements will require the provision of a pathway, drainage reserve or park in which the pipelines may be located.

7.8 Pipe material and standards

7.8.1 Local authority requirements

The following provisions are included for general guidance. Specific advice should be obtained from the relevant local authority on what material types and other special requirements are applicable.

The following requirements are applicable to the trunk or local authority drainage system. Detailed requirements in respect of pipe work and related issues for the *Roof and Allotment Drainage System* are provided in section 7.13.

7.8.2 Standards

Materials used for the construction of stormwater systems should comply with the following Australian Standards and other Standards as applicable.

AS 1254	PVC Pipes and Fittings for Storm or Surface Water Applications
AS 1260	PVC-U Pipes and Fittings for Drain, Waste and Vent Applications
AS 1273	Unplasticized PVC (UPVC) Downpipe and Fittings for Rainwater
AS 1597	Precast Reinforced Concrete Box Culverts
AS 1646	Elastomeric Seals for Waterworks Purposes
AS 1761	Helical Lock-Seam Corrugated Steel Pipes
AS 1762	Helical Lock-Seam Corrugated Steel Pipes – Design and Installation
AS 2032	Code of Practice for Installation of UPVC Pipe Systems
AS 2041	Buried Corrugated Metal Structures
AS 2042	Corrugated Steel Pipes, Pipe-Arches and Arches – Design and Installation
AS 2566.1	Buried Flexible Pipelines – Structural Design
AS 2566.2	Buried Flexible Pipelines – Installation
AS 3500.3	National Plumbing and Drainage Code – Part 3: Stormwater Drainage
AS 3500.5	National Plumbing and Drainage Code – Part 5: Domestic Installations
AS 3571	Glass Filament Reinforced Thermosetting Plastics (GRP) Pipes – Polyester Based – Water Supply, Sewerage and Drainage Applications
AS 3600	Concrete Structures
AS 3725	Loads on Buried Concrete Pipes
AS 3735	Concrete Structures Retaining Liquids
AS 3996	Access Covers and Grates
AS 4058	Precast Concrete Pipes (Pressure and Non-Pressure)
AS 4139	Fibre Reinforced Concrete Pipes and Fittings
AS 4799	Installation of Underground Utility Services and Pipelines within Railway Boundaries
AS 5100	Bridge Design CD-ROM (AustRoads)
MRS 11.24	Manufacture of Precast Concrete Culverts (Main Roads Department, Queensland)
MRS 11.25	Manufacture of Precast Concrete Pipes (Main Roads Department, Queensland)

MRS 11.26   Manufacture of Fibre Reinforced Concrete Drainage Pipes (Main Roads Department, Queensland)

AS/NZS 5065   Polyethylene and Polypropylene Pipes and Fittings for Drainage and Sewerage Application

AustRoads   Guide to bridge technology (2005)

Waterway design – A guide to the hydraulic design of bridges (1994)

Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

Designers should also note the structural design and cover conditions outlined in section 7.9.

7.8.3   Pipes and pipe laying

It is recommended that jointing for pipes comply with Table 7.8.1.

Table 7.8.1 – Jointing requirements for pipes – normal conditions

Pipe size (mm)	Joint type
Up to 600	Spigot and socket, or rubber ring joint
675 and above	Flush jointed, external rubber band, or approved equivalent

Notwithstanding the requirements of Table 7.8.1 rubber ringed spigot and socket joints should generally be used for all sizes of pipe in unstable ground, when pipes are laid in sand, or where pipe movement is possible, such as on the side of fills or at transitions from cut to fill.

Rubber ringed spigot and socket joints should also be used where the normal groundwater level is above the pipe obvert or where the design HGL is significantly (1.5 m or greater) above obvert level.

(a)   Minimum pipe size

The minimum diameter of any pipe in a local government drainage system should be 375 mm. In the following circumstances a 300 mm diameter pipe may be acceptable subject to hydraulic analysis:

• a gully connection from a single gully

• the connection between twin spaced gullies

• the connection from a sag gully provided purely to prevent pooling after a storm may.

Recommendations in respect of pipe sizes for roof and allotment drainage are presented in section 7.13.

(b)   Lateral spacing of pipes

Where multiple pipes are used they should be spaced sufficiently to allow adequate compaction of the fill between the pipes. The clearance between the outer face of the walls of multiple pipes should generally be in accordance with Table 7.8.2. The local government may permit lesser spacing in special circumstances to reduce structure costs, where easement width is limited, or for relief drainage works.

Table 7.8.2 – Recommended minimum spacing of multiple pipes

Diameter of pipes (mm)	Recommended minimum clear spacing (mm)
Up to 600	300
675 to 1800	600

Notes (Table 7.8.2):

1. The above minimum spacing requirements may need to be modified to satisfy structural considerations especially when laid at depth, under traffic loads or for pipes greater than 1800 mm in diameter.

2. Where lean mix concrete vibrated in place or cement stabilised sand is used for backfill, the clear spacing may be reduced to 300 mm for all diameters, subject to structural considerations.

Pipe laying shall be carried out in accordance with the specification of the relevant local authority, or other specification acceptable to the local authority.

(c)   Pipe trench compaction

Construction supervisors and stormwater managers are warned about the potential damaging effects of compacting trenches with wheel roller attachments that can impart significant live loads on the pipe. The choice of pipe material and structural grade will depend on the chosen method of installation.

Recommendations on the compaction of earth around concrete pipes may be obtained from the Concrete Pipe Association's web site or Concrete Pipe Selection software.

7.8.4   Box sections

Box culverts may be used where available depth to invert is restricted or to provide maximum waterway area and minimum obstruction to flow.

The minimum waterway dimension of any box section should normally be 300 mm (or 375 mm for cross drainage road culverts). However in the case of a connection from a single gully pit, other than at a sag, the minimum vertical dimension may be 225 mm.

The minimum cover over a box section should normally be 400 mm. This may be reduced to 100 mm in conjunction with a concrete or asphaltic-concrete, full-depth surfacing (subject to structural considerations). The maximum depth of fill for box sections is normally limited to 10 m, again subject to structural considerations.

Where box culverts are constructed on a skew; special precautions may need to be taken to resist unbalanced earth pressures.

7.8.5   Access chambers and structures

All structural concrete work should be executed in accordance with the current edition of:

AS 3600   SAA Concrete Structures Code

AS 3610   Formwork for Concrete

AS 1302   Steel Reinforcing Bars for Concrete

Concrete finishes shall be in accordance with Table 3.3.1 of AS 3610, as follows:

(i)   Normally exposed to view e.g. faces of wingwalls   Class 3

(ii) Not normally exposed to view e.g. inside of access chamber   Class 4

(iii) Base slabs for box culverts, floors and benching of pits,   Dense, wood float finish
      aprons and channel inverts   of uniform texture

The minimum concrete class for stormwater drainage works should be as follows:

(i)   Major endwalls and other major structures   32 MPa

(ii) Access chambers, kerb inlets, minor endwalls and other minor structures   25 MPa

Requirements relating to the durability of concrete in aggressive groundwater and salt-water conditions are presented in section 7.9. Designers should also note the structural design and cover conditions outlined in section 7.9.

Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

7.9   Structural design of pipelines

Loads on buried pipelines include:

(a) Fill over the pipe, which is a function of:
   • height of fill
   • type of fill material
   • installation conditions (e.g. trench or embankment).
(b) Normal traffic loads
(c) Construction traffic loads
(d) Other or abnormal load conditions

The load bearing capacity of a pipeline is a function of:

• pipe strength class

• type of bedding and backfill material

• pipe diameter.

In the case of culverts, the invert level is generally fixed by the bed level of the adjacent watercourse. The design problem is thus to select a suitable class of pipe and type of bedding to suit the pipe diameter, height of fill over the pipe, type of fill material, installation condition and traffic load.

In urban drainage design the depth of the pipeline is usually not a constraint. In this case the design exercise is to select the most economic combination of pipe depth, strength class and bedding type.

The structural design of pipelines should be carried out in accordance with AS 3725 Loads on Buried Concrete Pipes, CPAA Pipe Class V1.1 Concrete Pipe Selection Software, the latest version of AustRoads Bridge Design Code, and AS 2566.2 Buried Flexible Pipelines – Installation

The absolute minimum cover over any pipe, irrespective of location, class and bedding, should be 300 mm, unless special protection is provided, such as a structural concrete slab. Table 7.10.1 details recommended minimum cover.

All pipes, box sections and access chambers in road reserves, whether under the road pavement or within the footpath area, and all pipes within Industrial and Commercial allotments, should be designed for a W7 wheel loading in accordance with AustRoads (2005) where applicable standard drawings are not available from the local authority. Note that the W7 loading should be modified for impact effects in accordance with the buried structures provisions and distributed in accordance with AustRoads (2005).

The minimum strength class for concrete drainage pipes should be Class 2. To achieve uniform pavement compaction, pipes under the road pavement should be laid prior to placing of the pavement material. Accordingly, such pipes should have adequate cover between the top of the pipe and the subgrade level, to support loads imposed by construction plant. In general, such loads may be taken as being equivalent to Standard W7 loading unless unusual conditions prevail.

Where pipelines, whether located under road pavements or otherwise, are laid prior to completion of bulk earthworks, the possibility of them being subjected to heavy construction traffic should be considered and extra cover provided, a stronger class of pipe used, or the pipes otherwise protected.

Where aggressive ground conditions exist, or where the system might be exposed to salt water, it may be necessary to provide additional concrete cover to reinforcement or protective coating to exposed surfaces. The supply and proper installation of high-quality impermeable concrete is the most effective means of corrosion prevention.

This can be achieved by designing a dense concrete mix with water:cement ratio less than 0.5 and cement content of at least 330 kg/m^3 and ensuring that placement is properly supervised. Designers should refer to Technical note TN57 (C&CA 1989) for more detailed recommendations. Cover requirements should comply with AS 1342 in respect of pipes, AS 1597 for box culverts and AS 3600 for access chambers.

7.10   Minimum cover over pipes

The minimum cover over pipes to be adopted for pipe grading purposes should be:

Table 7.10.1 – Recommended minimum cover over pipes

Location	Minimum cover (mm)	
	Rigid type pipes e.g. concrete, FRC	Flexible type pipes e.g. plastic, thin metal
Residential private property, and parks not subject to traffic	300	450
Private property and parks subject to occasional traffic	450	450
Footpaths	450	600
Road pavements and under kerb and channel	600	600

Notes (Table 7.10.1):

1.   For special cases, and with the agreement of the local authority, cover can be reduced by using a higher-class pipe, special bedding, concrete protection or a combination of these.

2.   Where pipes are to be laid under the footpath consideration should be given to the possibility of future road widening, both in respect of the reduced cover that might result from the widening and vehicle loading.

7.11   Flow velocity limits

The velocity of stormwater in pipes and box sections should be maintained within acceptable limits to ensure that:

• self cleaning of the pipe or box section is maintained

• scouring and erosion of the conduit (particularly the invert) does not occur.

The range of acceptable flow velocities are as detailed in Table 7.11.1.

Table 7.11.1 – Acceptable flow velocities for pipes and box sections

Flow condition	Absolute minimum [1] (m/s)	Desirable minimum [1] (m/s)	Desirable maximum [2] (m/s)	Absolute maximum [2] (m/s)
Partially full	0.7	1.2	4.7	7.0
Full	0.6	1.0	4.0	6.0

Notes (Table 7.11.1):

[1]   Minimum flow velocities apply to 63% AEP (1 year ARI) design storm, and apply to all pipe materials.

[2]   Maximum flow velocities apply to concrete pipes. For other pipe materials, refer to manufacturer's advice.

In steep terrain the velocity of flow should not be greater than the absolute maximum velocity of 6.0 m/s under 'pipe full' conditions. To achieve this requirement, it may be necessary to construct access chambers with drops to dissipate some of the kinetic energy of the flow, or to limit the pipe diameter.

Reference should be made to tables 9.5.1 and 9.5.3 for details of velocity limits for vegetated and grassed/unlined channels.

Notwithstanding the above suggested velocity limits, hydraulic considerations may require the velocity be controlled to well below the 'desirable maximum' and/or the pipe size increased to minimise structure losses and the slope of the hydraulic grade line.

7.12   Pipe grade limits

To conform with the requirements of section 7.11, and construction limitations the following maximum and minimum grades are recommended for design purposes:

Table 7.12.1 – Acceptable pipe grades for pipes flowing full

Pipe diameter (mm)	Maximum grade (%)	Minimum grade (%)
300	20.0	0.50
375	15.0	0.40
450	11.0	0.30
525	9.0	0.25
600	7.5	0.20
675	6.5	0.18
750	5.5	0.15
900	4.5	0.12
1050	3.5	0.10
1200	3.0	0.10
1350	2.5	0.10
1500	2.2	0.10
1650	2.0	0.10
1800	1.7	0.10
1950	1.5	0.10
2100	1.4	0.10
2250	1.3	0.10
2400	1.2	0.10

Notes (Table 7.12.1):

1.   Based on maximum velocity for pipe flowing full of 6.0 m/s.

2.   Based on minimum velocity for pipe flowing full of 1.0 m/s except where Note 4 is applicable.

3.   Manning's n = 0.013 for all cases (concrete pipes).

4.   The minimum grade of 0.10% (1:1000) is based on construction tolerance requirements.

5.   The **'maximum grade'** requirement applies to both the pipe grade and the hydraulic grade.

6.   The **'minimum grades'** apply to the pipe grade only.

7.   Where a pipe is flowing less than half full for the design flow being considered, it is permissible to exceed the above maximum grades provided that the velocity limits specified in Table 7.11.1 are not exceeded.

7.13   Roof and allotment drainage

7.13.1   General

Application of Water Sensitive Urban Design (WSUD) principles requires the disconnection of impervious areas from impervious drainage systems. This usually means a disconnection between the roofwater drainage systems and the trunk drainage network. This section of the Manual provides information on five levels of 'traditional' roof and allotment drainage for use in circumstances where the 'preferred' WSUD approach is impractical.

The reasons for selecting one of the following levels over another may be based on land use (e.g. commercial or residential), density of development, community standards, or the requirement for a given level of protection from flooding by storm runoff. In certain developments a combination of these systems may be required.

Design and construction of roof and allotment drainage systems and appurtenances should comply with AS 2180 and AS 3500.3.

7.13.2   Roof drainage

The design of gutters and downpipes for roof drainage should be undertaken in accordance with NSB 151, NSB 152 and NSB 153 (CSIRO) and AS 2180 to adequately convey the runoff from the design storm detailed in Table 7.13.1.

Table 7.13.1 – Design of roof gutters and downpipes

Design storm	39% AEP (1 in 2 years) Duration = 5 minutes [1]
Check storm [2]	1% AEP (1 in 100 years) Duration = 5 minutes [1]

Notes (Table 7.13.1):

[1]   The critical storm duration of 5 minutes should be adopted unless special circumstances justify a longer duration.

[2]   A design check should be undertaken to determine the effect of the check storm where the consequences of hydraulic failure are significant or where the system contains vulnerable components such as internal box gutters.

7.13.3   Roof and allotment drainage

Outside the requirements for WSUD, the drainage system provided within allotments for the disposal of roof and allotment drainage depends upon the topography, the importance of the development, and the consequences of failure. The local government may determine that the provision of a piped allotment drainage system to receive roof and allotment runoff is necessary in the following circumstances:

- where allotments fall away from the street

- where the proportion of impervious area within a development is such that the frequency and volume of surface runoff is likely to be intolerably high, e.g. industrial and multi-unit residential allotments

- where zoning may permit construction of buildings up to side or rear boundaries thus blocking or concentrating natural flow paths

- where there is significant catchment draining into the rear of the property.

7.13.4 Level of roof and allotment drainage system

The level of roof and allotment drainage system provided within a development is differentiated by the components making up the system and the sophistication necessary in the design of these components.

Depending upon the size or importance of a development or the consequences of failure of the roof and allotment drainage system, the local government may nominate the level of system to be provided. Figure 7.13.1 indicates the types of developments to which the various levels may be applicable. Table 7.13.2 details the various components and Table 7.13.3 indicates the level of system to which these are applicable.

Each of the examples provided in Figure 7.13.1 may be appropriately modified to incorporate the use of rainwater tanks and/or on-site detention systems to the discretion of the local government.

The following sections permit the design of underground allotment and rear of allotment drainage pipes in some cases to an AEP of lower standard (i.e. higher probability) than that detailed in Table 7.13.1. This implies that surcharge may occur from the underground system. The sections of underground pipe leading from the downpipes to the points where surcharge can occur should be sized to prevent a constriction of flow in the downpipe system. Beyond those points the provisions of Table 7.13.4 are applicable.

Table 7.13.2 – Roof and allotment drainage components (also see Table 7.13.3)

Identifier	Description of component
(a)	Guttering
(b)	Downpipes
(c)	Rainwater tanks
(d)	Minor pipes in allotment
(e)	Connection to kerb and channel
(f)	Seepage trenches or rubble pits (where permitted)
(g)	Connection to a kerb inlet or trunk drainage system in the street
(h)	Connection to rear of allotment drainage system
(i)	Rear of allotment drainage system designed to receive roof-water from one or more allotments and with a connection point to receive roof-water only at each allotment
(j)	Rear of allotment drainage system designed to receive both roof-water and allotment surface runoff from one or more allotments and with a connection point to receive roof-water and a grated kerb inlet to receive surface runoff at each allotment
(k)	Allotment drainage system designed to receive both roof-water and allotment surface runoff from one allotment or complex and comprising kerb inlets, junction pits or access chambers and underground pipe system etc. and discharging to a rear of allotment drainage system, kerb inlet or trunk drainage system
(l)	As for (j) but discharging normally only to a trunk drainage system or other nominated lawful point of discharge
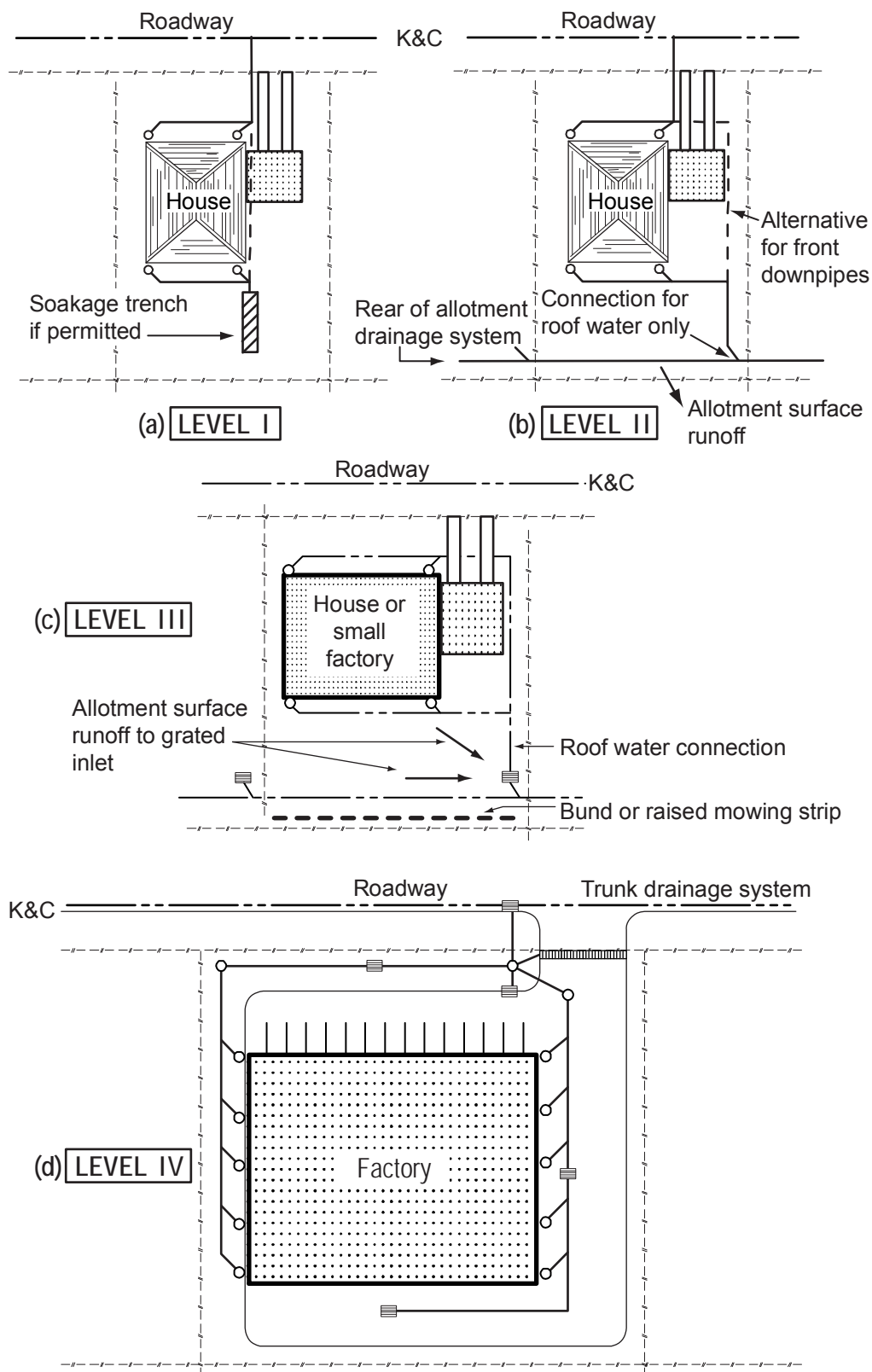
Figure 7.13.1 (a) to (d) – Levels of roof and allotment drainage system (see also Figure 7.13.1 (e))
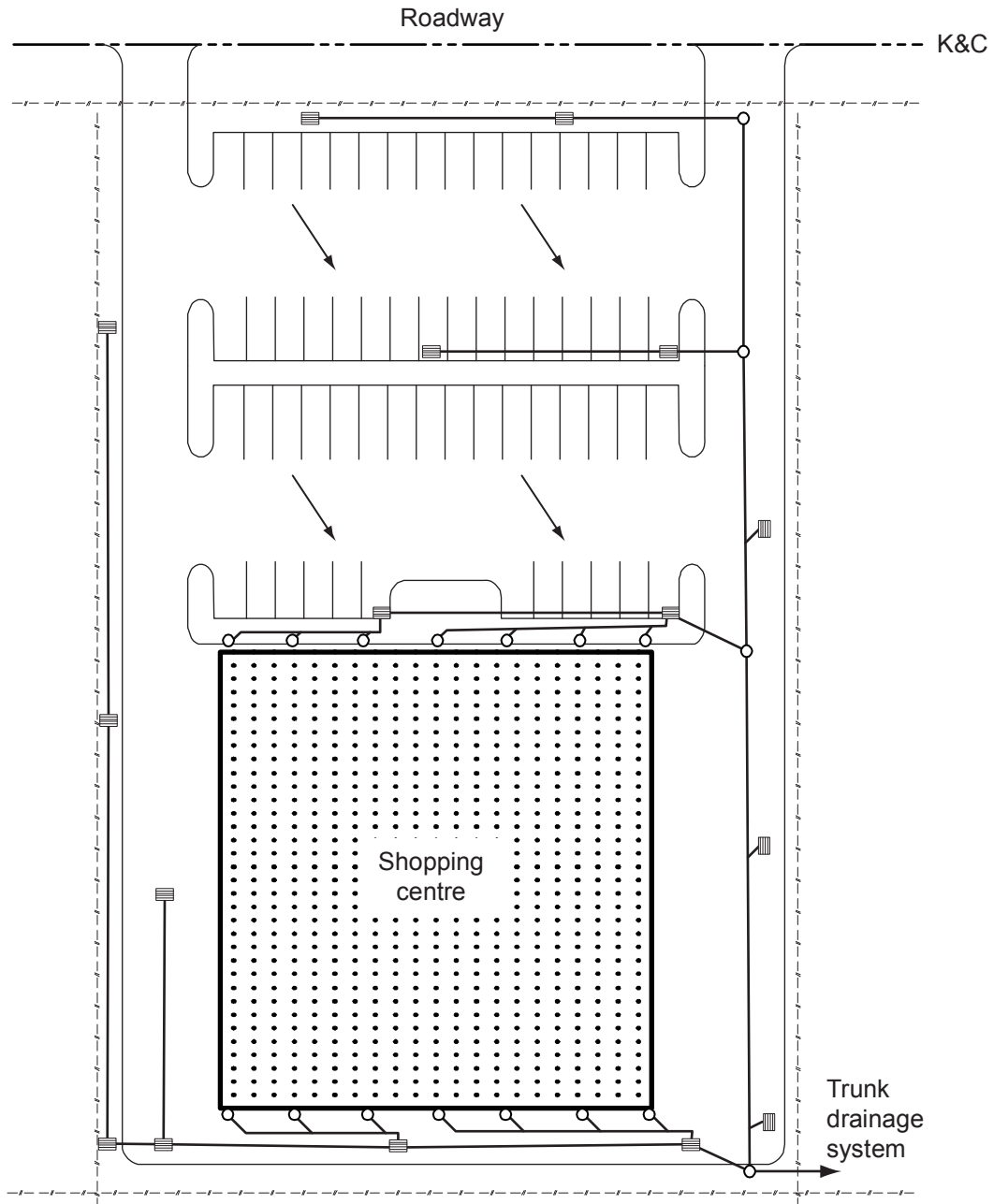
(e) LEVEL V

Figure 7.13.1 (e) – Levels of roof and allotment drainage system

Note: Each of the examples provided in figures 7.13.1(a) to (e) may be appropriately modified to incorporate the use of rainwater tanks and/or on-site detention systems to the discretion of the local government.

Table 7.13.3 – Levels of roof and allotment drainage

Level	Components (refer to Table 7.13.2)	Design complexity	Where normally applicable
I	(a), (b), (c), (d), (e) and (f) where permitted	NSB and nominal pipe sizes underground	Low density urban residential, corner stores and other minor developments
II (roofwater only)	(a), (b), (c), (d), (e), (h) and (i)	NSB Rational Method and pipe flow nomograph, or nominal pipe sizes—see Table 7.13.5	Low density urban residential and other minor developments as nominated by the local government
III (roof and allotment runoff)	(a), (b), (c), (d), (e), (h) and (j). (f) where nominated	NSB Rational Method and pipe flow nomograph, or nominal pipe sizes—see Table 7.13.6	Where nominated by local government
IV	(a), (b), (c) and (k). (d) where permitted	NSB Rational Method, full hydraulic analysis or pipe flow nomograph with allowance for structure losses	Commercial, industrial, high density urban residential and other developments as nominated by the local government
V	(a), (b), (c) and (l)	NSB Rational Method and full hydraulic calculations including structure losses and determination of HGL	Central business and large commercial, industrial, high density urban residential developments, or where nominated by the local government

Abbreviations (tables 7.13.3 and 7.13.4):

FRC = fibre reinforced cement (pipe)

NSB = Notes on the Science of Building (CSIRO)

RCP = reinforced concrete pipe

RRJ = rubber ring jointed

S & S = spigot and socket

UPVC = unplasticised polyvinyl chloride (pipe)

7.13.5   The rear of allotment drainage system

The rear of allotment drainage system is provided for the collection of storm runoff from allotments falling away from the street or from other allotments which are impeded from discharging runoff from the whole of the allotment to the trunk drainage system in the street. These systems are normally constructed by the developer and may or may not become part of the trunk drainage system owned and maintained by the local government.

The rear of allotment drainage system is sometimes referred to as 'inter allotment drainage'.

The system should be designed to receive the peak runoff as determined from the guidelines set out in Table 7.13.4. This table also contains certain recommendations in respect of construction requirements etc.

The location of the rear of allotment drainage system and boundary clearance should be as directed by the local government.
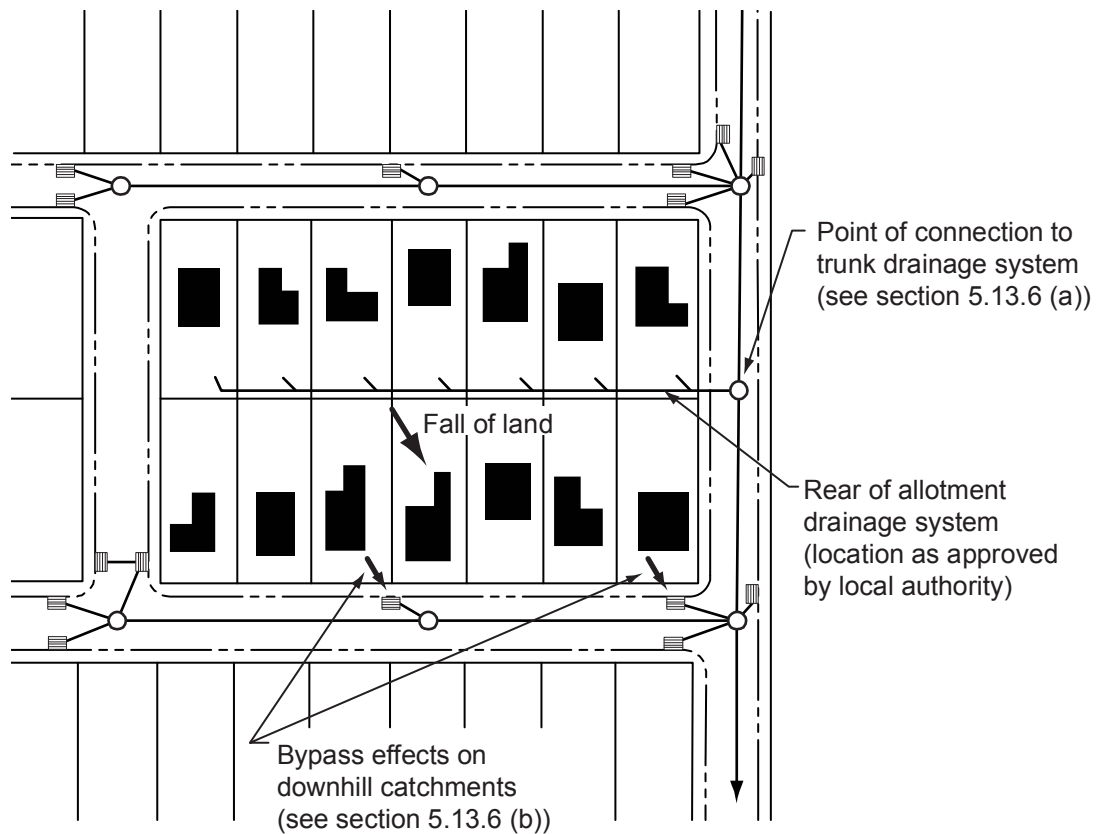
Figure 7.13.2 – Effects on trunk drainage network

Table 7.13.4 – Design recommendations for the rear of allotment drainage system (Refer to Table 7.13.3 for abbreviations)

Item	Level applicable				
	I	II	III	IV	V
Minimum pipe size	NA	150 mm	225 mm	375 mm [1]	
Minimum stub size	–	150 mm	150 mm	To be designed	
Pipe material	–	UPVC	UPVC, RCP, FRC	RCP, FRC	
Jointing system	–	RRJ, S&S	RRJ, S&S	RRJ, S&S	
Flow calculation	–	10 L/s per allotment	See Table 7.13.6	Rational Method or runoff model	
AEP for design	NA	See Table 7.13.5	See tables 7.13.5 and 7.13.1	39% (20 year ARI) [2]	
Pipe system design	NA	See Table 7.13.5	See Table 7.13.6	Full hydraulic analysis or pipe nomograph plus structure losses	Full hydraulic analysis with determination of HGL
Major Design Storm overland flow check	Ensure the land development and its drainage system does not unlawfully concentrate flows onto, or aggravate flooding within, neighbouring properties. The overland flow path is to be identified within the system design. Also refer to tables 7.13.7 and 7.13.8.				
				Refer to Note [2] below	

Note (Table 7.13.4):

[1]   Subject to hydraulic analysis the connection from a single kerb inlet may be 300 mm diameter.

[2]   For Level IV and V systems the underground drainage system should be designed to convey discharge for the Major System AEP storm from trapped sags and other locations where an acceptable overland flow path is unavailable.

Table 7.13.5 – Recommended design criteria for Level II rear of allotment drainage system

Item	Recommendation							
Maximum number of allotments served	20							
Flow applicable	10 L/s per allotment [1]							
Minimum pipe grade	0.35%							
Minimum pipe cover (mm)	500							
Pit dimensions for depth to invert (a) ≤ 750 (b) > 750	(a) 600 x 600 (b) 600 x 900							
Nominal pipe diameter (mm)	**Flow (L/s)** [2]							
	Pipe gradient (%) [3]							
	0.5	**1.0**	**1.5**	**2.0**	**2.5**	**3.0**	**4.0**	**5.0**
150	[4]	18	23	26	30	33	38	42
225	38	56	67	78	87	96	110	125
300	84	120	146	170	190	210	N.A.	N.A.

Notes (Table 7.13.5):

[1]   Based on roof areas of 180 m^2 and AEP = 5% for S.E. Queensland.

[2]   Based on Manning's n = 0.011 and the likely use of UPVC for smaller pipes.

[3]   Where the pipe gradient is in excess of 5% a more detailed hydraulic analysis should be undertaken including the assessment of structure losses, where appropriate.

[4]   Minimum grade 1% for 150 mm diameter pipe to comply with AS 3500.3.

Table 7.13.6 – Recommended design criteria for Level III rear of allotment drainage system

Item	Recommendation							
Maximum number of allotments served	20							
Flow applicable Allotment area ≤ 750m² Allotment area > 750m²	– Rational Method flow with pipe size from table below [1] – Rational Method flow < use a standard pipe nomograph							
AEP for design	Minor system AEP as per Table 7.3.1							
Minimum pipe grade	0.35%							
Minimum pipe cover	500 mm							
Number of allotments	**Recommended pipe diameter (mm) [1 & 2]**							
	Pipe gradient (%) [3]							
	0.5	1.0	1.5	2.0	2.5	3.0	4.0	5.0
1	225	225	225	225	225	225	225	225
2	300	300	225	225	225	225	225	225
4	375	300	300	300	300	300	300	225
6	450	375	375	300	300	300	300	300
8	450	450	375	375	375	375	300	300
10	525	450	450	375	375	375	375	375
12	525	450	450	375	375	375	375	375
14	525	450	450	450	375	375	375	375
16	525	525	450	450	450	375	375	375
18	600	525	450	450	450	450	450	375
20	600	525	525	450	450	450	450	450

Notes (Table 7.13.6):

[1]   The pipe sizes shown have been based on discharge from allotments of average size = 750 m², a 5 minute storm duration and AEP = 39% (ARI = 2 yr) for Brisbane, i.e. 150 mm/h. This equates to 20 L/s per allotment. **Note:** for other locations and/or allotment densities, pipe sizes should be adjusted accordingly.

[2]   Based on Manning's n = 0.013.

[3]   Where the pipe gradient is in excess of 5% a more detailed hydraulic analysis should be undertaken including assessment of structure losses, where appropriate.

[4]   Grated inlets should be designed with allowance for blockage as detailed in Table 7.5.1.

[5]   The gully inlet at each allotment should be located where possible at the lowest point and should be contained in a bund or catch drain to minimise bypass.

7.13.6 Effect of roof and allotment drainage system on trunk drainage network

There are two issues that need to be considered when assessing the effects of the roof and allotment drainage system on the design and performance of the trunk drainage network and the down-slope property drainage system (refer to Figure 7.13.2).These issues are:

- the hydraulic effects at the point of connection to the trunk drainage system
- potential impacts of bypass flows on down-slope properties.

(a)   Hydraulic effects at point of connection

This relates to hydraulic design of the trunk drainage system and the rear of allotment drainage system at the point of connection to the trunk drainage system. Table 7.13.7 details the manner in which this should be undertaken.

Table 7.13.7 – Design considerations for the connection of allotment drainage to the trunk drainage system

Level	Design considerations
I	Design for street flows and trunk network with appropriate catchment area—ignore local effect at connection to kerb and channel
II	(a) For minor storm AEP – Design for full discharge [1] from rear of allotment drainage system in trunk network downstream of connection—ignore structure losses at point of connection (b) For major storm AEP – Ignore rear of allotment drainage system [2]
III	(a) For minor storm AEP – Design for full discharge [1] from rear of allotment drainage system and associated structure losses at point of connection [3] (b) For major storm AEP – Ignore rear of allotment drainage system [2]
IV and V	(a) For minor storm AEP – Design for full discharge from roof and allotment system and associated structure losses at point of connection [3] (b) For major storm AEP – Check ability of trunk network to accept flow at point of connection and design for inflow accordingly including associated structures losses [3]

Notes (Table 7.13.7):

[1]   The full discharge referred to corresponds to 100% of the calculated discharge determined in accordance with Table 7.13.5 or Table 7.13.6.

[2]   For Level II and III systems it is assumed that the rear of allotment system will be ineffective during the major design storm and that roof and allotment runoff will bypass to the downstream catchments.

[3]   Although the roof drainage and pipes connected immediately thereto will be designed for the AEP detailed in Table 7.13.1, the design storm applicable to the roof and allotment drainage system to satisfy the design check required by Table 7.13.7 should be that of the trunk drainage system to which it is connected.

(b)   Bypass effect on down-slope catchments

Concurrent with the design discharge to the trunk drainage system referred to in (a) above, allowance should be made for bypass flows resulting from possible inefficiency of collection associated with the roof and allotment drainage system. The down-slope catchment should be designed to receive the bypass as detailed in Table 7.13.8.

Table 7.13.8 – Bypass from roof and allotment drainage system to down-slope catchments

Level	Bypass allowance
I	100% of calculated runoff
II	(a) For minor design storm AEP – 100 % of allotment runoff (i.e. roof runoff not bypassed) (b) For major design storm AEP – 100% of roof and allotment runoff
III and IV	(a) For minor design storm AEP – Nil (b) For major design storm AEP – 100% of roof and allotment runoff for major design storm less minor design storm capacity of roof and allotment drainage system
V	(a) For minor design storm AEP – Nil (b) For major design storm AEP – 100% of roof and allotment runoff for major design storm less calculated capacity of roof and allotment drainage system during the major design storm

7.14   Public utilities and other services

7.14.1   General

In urban areas, drainage is only one of many public utility services that must be provided. Appropriate consideration should be given to all services, with priority being given to those services which are grade dependent, e.g. sewer and stormwater. Designers should check for potential conflicts and allow for these in the design.

The following is a list of services commonly encountered:

- Water supply – reticulation and trunk
- Sewerage – reticulation and trunk
- Telecommunication – distribution, coaxial and fibre optic
- Gas – distribution and trunk
- Oil and natural gas pipelines
- Electricity – distribution and mains
- Water service crossings
- Sewer house connections
- Roof-water drainage
- Other stormwater

7.14.2   Clearances to services

Where conflicts exist in the alignment and level of services it will be necessary to ensure that adequate clearance is provided between the outer faces of each service. The nominated clearance should allow for collars and fittings on pipes and special protection if required (e.g. a concrete surround).

In general the minimum clearance between the outer faces of services should be 200 mm, or as permitted by the services authority.

Penetrations by services through stormwater pipes should be avoided. Where it is necessary for a service to penetrate a stormwater pipe or access chamber allowance should be made for the hydraulic losses in the system resulting from the penetration. In addition the service should be contained in a pipe or conduit of sufficient strength to resist the forces imposed on it by flow, including debris, in the stormwater system. Unless otherwise agreed by the local authority and/or utility owner, penetrations should be constructed using ductile iron pipe. To assist in the removal of debris collected on service pipes or conduits passing through a drainage system it is recommended that an access chamber be located at the pipe or conduit penetration.

Reference should be made to the utility allocations applicable in the local government area, when designing the stormwater system.

7.15   Discharge calculations

7.15.1   General

The objectives and design philosophy outlined in Chapter 1 seeks to limit flooding of property and to ensure a reasonable level of pedestrian and vehicular traffic safety and accessibility. These objectives are met by ensuring that major and minor storm flows are managed within specified limits, and by designing both major and minor system components in conjunction.

If the major and minor components of the surface system do not have the capacity to carry the difference between the respective design peak flow and the pipe flow, then additional inlets and hence larger pipes are required to ensure that the surface system operates within the specified limits.

Where the drainage system contains few or no underground pipe components, it will be necessary for the surface system to perform within the limits detailed in section 7.3.6 and Chapter 9 – *Open channel hydraulics* as applicable.

7.15.2   General principles

The following principles apply to the design of urban drainage systems:

- The drainage system as a whole is provided to mitigate against property flooding and to ensure the safety and convenience of pedestrians and vehicles.

- The minor drainage system (comprising underground pipes and/or surface flow paths) is designed to provide for the safety and convenience of pedestrians and vehicles.

- Where flood immunity cannot be provided for property and buildings under major storm conditions via overland flow paths, the capacity of the underground pipe system and the inlets leading to it need to be increased in order to reduce surface flows to acceptable levels.

- Under normal conditions the capacity of the underground pipe system should not be less than its minor storm flow conditions while the system is operating under major storm conditions. The exceptions would be when tailwater levels downstream have a significant effect on the

system's hydraulic gradeline, or the surface gradient is considerably flatter than the pipe gradient, thus causing the HGL to rise above the ground surface.

- The underground system should be designed with a suitable allowance for blockage at kerb inlets as described in section 7.5.2. In this way the full design capacity of the underground system is likely to be available under both major and minor storm conditions.

7.15.3   Design procedure

The design procedure is detailed below and in Figure 7.15.2.

Note that the procedures described herein do not attempt to ascribe an AEP to the flow conveyed in the pipe system, or even set the type of Minor Drainage System (e.g. pipe or swale). Rather the total system is designed to convey the calculated peak flows during major and minor storm events of selected AEP whilst adhering to public safety and convenience criteria separately applicable under relevant conditions.

Phase A: Layout and topographical assessment

(i)   Identify the preferred location of major overland flow paths as discussed in section 7.1.

(ii) Decide preliminary road layout and road widths (if not existing). Depending on the results of Phases D and E, this preliminary layout may need to be altered to optimise the stormwater drainage system.

(iii) Assess where trapped sags or other topographical constraints will result in a need for an overland flow path other than along a road. Use this as a basis for locating parks, drainage reserves, etc.

Note: Relief drainage or upgrading works may involve flow through existing private allotments.

Phase B: Water Sensitive Urban Design

(i)   Identify opportunities for application of the principles of Water Sensitive Urban Design (section 11.3.2).

(ii) Identify opportunities for the retention and/or rehabilitation of natural waterways (section 9.2(b)) and other natural water features that will be compatible with the urban landscape.

Phase C: Conceptual design of stormwater quality requirements

(i)   Identify stormwater quality requirements from an existing Stormwater Quality Management Plan or identified Water Quality Objectives (sections 2.6 and 11.6).

(ii) Identify those areas of land with topographic features best suited to specific stormwater treatment systems (e.g. natural detention areas for wetland placement, and highly porous soils for infiltration systems).

(iii) Prepare a preliminary design of the stormwater treatment system using appropriate modelling techniques.

Phase D: Minor storm initial assessment

(i)   Assess critical locations in the street network where roadway flow width is likely to be the limiting criterion under minor storm conditions. Refer to the limitations detailed in figures 7.3.1, 7.3.2 and 7.4.1 and tables 7.3.1, 7.4.1 and 7.4.3, e.g. intersections, sags, bus stops, kerb returns and intermediate locations. This provides an indication of sub-catchment boundaries.

(ii) Determine the area of the critical sub-catchments at the locations determined in (i) and calculate peak discharges for the minor storm event at these locations using standard inlet

times (Table 4.6.2), the design average recurrence interval for the minor storm (Table 7.3.1) and weighted coefficients of runoff (section 4.5).

Notes:

- Significant bypass will not normally occur at kerb inlets under minor storm conditions. Accordingly the use of standard inlet times will be appropriate when planning the initial layout of the system. If significant bypass does take place the time of concentration at downstream inlets will need to be appropriately adjusted.

- Depending upon the local rainfall intensity regime, kerb inlet capacity and assessed sub-catchment coefficient of runoff, the designer can readily determine the approximate maximum size of sub-catchment area that is likely to be acceptable.

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e.g.

For Q	$=$	0.352 m³/s [1]
t^c	$=$	10 minutes
$^{10min}I_2$	$=$	120 mm/h
C_{10}	$=$	0.76
C_2	$=$	0.65

</div>

$$A = \frac{Q}{(2.78\times10^{-3}).C_y.(^tI_y)} = \frac{0.352}{(2.78\times10^{-3})\times0.65\times120} = 1.62\,ha$$

Note: 0.352 m³/s corresponds to the half road flow capacity of a road measuring 8 m invert to invert and with 3% longitudinal slope and 2.5% crossfall.

(iii)   For the longitudinal road slope determine the road capacity at critical locations based upon flow width and depth limitations.

(iv)   From (ii) and (iii) determine the required inlet capacity and underground pipe capacity (if used) at the critical locations together with surface flows and bypasses for minor storm conditions.

Phase E: Major storm initial assessment

(i)   Assess those critical locations in the street and overland flow network where flow capacity is likely to be the limiting criterion under major storm conditions. Refer to figures 7.3.1 and 7.3.2, and tables 7.3.1 and 7.4.3.

(ii)   Determine total catchment peak discharge $Q_{T()}$ at the critical locations under major storm conditions.

Notes:

- The critical locations under these conditions are likely to require a number of minor storm sub-catchments and significant bypass between sub-catchments will be permitted. Based upon a detailed assessment of overland flow time and channel flow time the peak discharge from the critical catchments can be determined.

- Where there is a significant difference between overland flow travel time and pipe flow time to the location in question, designers should consider the travel time of least duration, otherwise the designer should evaluate the catchment hydrology using an appropriate runoff-routing model.

(iii)   Determine the permissible street flow capacity based on major storm criteria at the critical locations, $Q_{LIM()}$.

(iv)   Starting at the top of the catchment determine the pipe flow at the upstream end of the sub-catchment under consideration $Q_{PU()}$ (see Figure 7.15.1).

(v)   Subtract $Q_{PU()}$ from $Q_{T()}$ to establish the net surface flow at the critical location under consideration.

(vi) Where the net surface flow at the critical location is less than the permissible street or overland flow move to the next downstream critical location.

(vii) Where the net surface flow at the critical location is more than the street or overland flow capacity then:

- allow for the provision of increased inlet and underground pipe capacity upstream of that point to accept the excess;
- modify the street cross-section;
- or otherwise increase the surface flow capacity.

(viii) Check that the calculated pipe capacity at the critical location is not less than that required upstream of that point. A reduction in pipe capacity would not occur unless provision is made for surcharge outflow.

(ix) Adopt trial pipe sizes for the hydraulic analyses to suit the greater of the flows derived during the major and minor storm hydrologic checks in accordance with the Flow Chart in figures 7.15.2 (a), (b) & (c).

(x)   The calculated major storm kerb inlet inflows and pipe flows are used for subsequent hydraulic analysis of the performance of the system under major storm conditions.

Note: This procedure allows the identification of points where underground capacity needs to be increased to cater for the flow requirements of both the major and minor design storms. It ensures the selection of pipe sizes that are capable of conveying both major and minor storm discharges, and largely obviates iterative hydrologic and hydraulic analysis of the pipe system.

Thus, if $Q_{T()}$ =   peak discharge from the total catchment at the critical location under consideration, based on Rational Method theory, i.e. it is not the sum of upstream sub-catchment discharges.

$Q_{LIM()}$ =   permissible major storm street or overland flow at the critical location under consideration.

$Q_{P()}$ =   required pipe discharge capacity at the critical location under consideration, i.e. at the downstream end of the catchment being considered. Thus $Q_{P(A)}$ = required pipe discharge at A.

$Q_{PU()}$ =   sum of the pipe discharges at the critical locations immediately upstream of the location now under consideration, i.e. sum of $Q_{P()}$ values upstream.

$Q_{SURF()}$ =   net surface flow at the critical location assuming that no kerb inlets have been provided in the section immediately upstream of the critical location now under consideration.

$Q_{gs()}$ =   required kerb inlet capacity of the inlets located in the section upstream of the critical location, i.e. between the location under consideration and the next upstream critical locations.

Then $Q_{SURF()}$ = $Q_{T()}$ - $Q_{PU()}$ (7.7)

$Q_{P()}$ = $Q_{T()}$ - $Q_{LIM()}$ (7.8)

$Q_{gs()}$ = $Q_{P()}$ - $Q_{PU()}$ (7.9)

$Q_{P()}$ not less than Q_{PU}, or provide surcharge outflow structure if appropriate (see Note 4)

Figure 7.15.1 explains the above procedure, (example only).
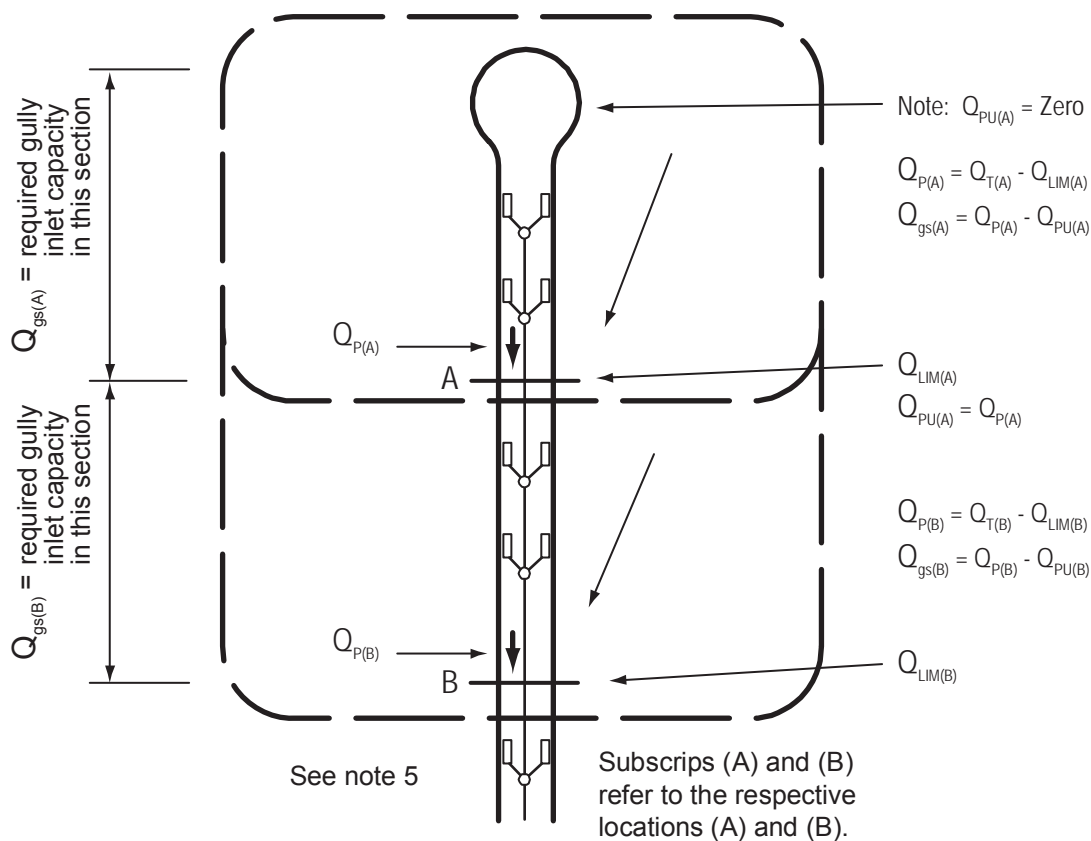
Figure 7.15.1 – Kerb inlet capacity for major storm

Notes (Figure 7.15.1):

1. The inflow capacity at kerb inlets under major storm conditions is expected to be equal to the inlet capacity under minor storm conditions unless elevated tailwater conditions under major storm conditions result in significantly reduced capacity, or the surface gradient is significantly flatter than the pipe gradient.

2. Where a number of minor storm sub-catchments exist upstream of the location being considered the capacity of the kerb inlet at that location may need to significantly exceed the minor storm inflow, in order to satisfy major storm criteria.

3. It should be assumed that kerb inlets will be designed with provision for blockage as detailed in Table 7.5.1.

    Accordingly there will be no need to further reduce the capacity of the underground drainage system under major storm conditions. This approach differs from that proposed by some authorities e.g. Argue (1986) etc. where reduction to 50% or zero pipe capacity is suggested.

4. Where equation 7.9 results in a negative value of $Q_{gs()}$ the kerb inlet capacity required in that section to satisfy road flow capacity requirements is nil. In this case the method may also indicate a reduced pipe capacity requirement in the lower reach. However the pipe capacity will normally not be reduced unless provision is made for surcharge outflow.

5. Note that at point B, the peak discharge $Q_{T(B)}$ comprises flow from both catchments A and B etc.

Notes (figures 7.15.2 (a), (b) and (c)):

1. Designers should endeavour to place kerb inlets at locations on grade where the width of spread of roadway flow is at the allowable limit as detailed in section 7.4 and Figure 7.5.1.

2. The capacity of kerb inlets shall be determined from the Kerb Inlet Capacity Charts made available by the relevant local authority and modified to allow for blockage in accordance with Table 7.5.1.

3. Constraints on the levels and gradient for pipe reaches may be caused by:

<div style="margin-left: 2em;">

    (i)   existing or future services e.g. sewer, water, gas, electricity

    (ii)   minimum cover under roadways

    (iii)   minimum or maximum depth for kerb inlets.

</div>

4.   The bypass referred to is from other upstream catchments not from the uppermost of the two under consideration.

5.   The peak discharge needs to be assessed for the full or partial area as for the minor storm design.

6. Q_u and Q_g are the inflows to the structure in accordance with Rational Method theory and do not equal the sum of the upstream pipe and kerb inlet flows. Q_u may include lateral inflows.

7.   The velocity limits indicated are those that should give optimum hydraulic conditions.
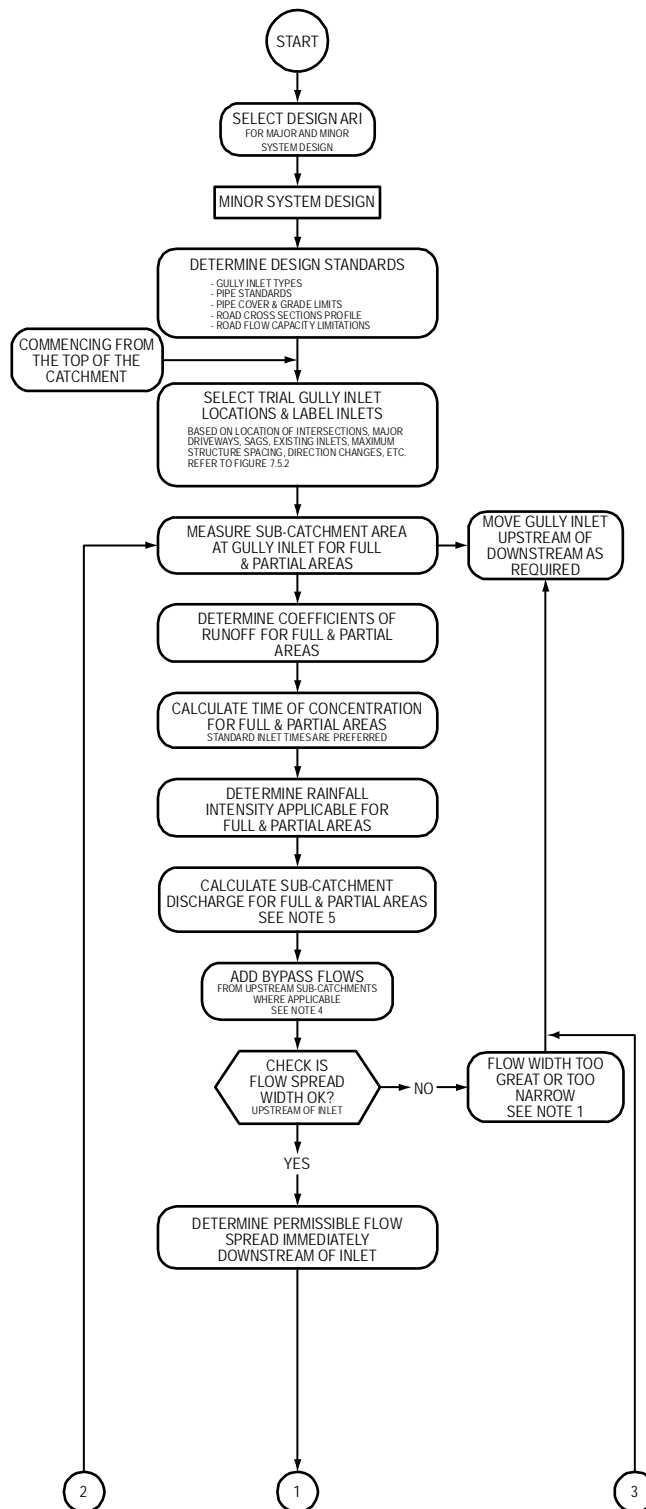
Figure 7.15.2 (a) – Flow chart for initial design assessment

Continued on Figure 7.15.2 (b)
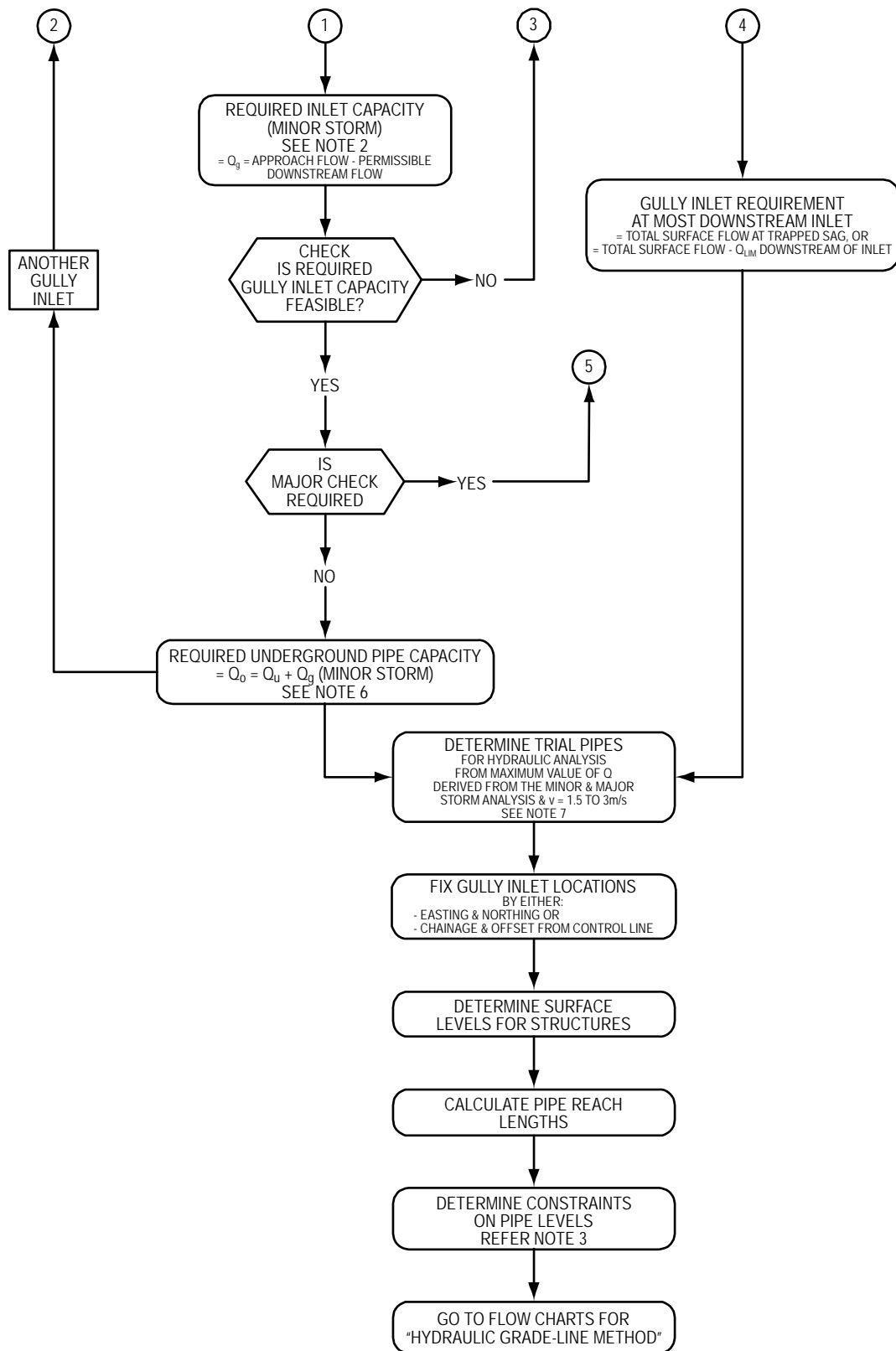
Figure 7.15.2 (b) – Flow chart for initial design assessment

Continued on Figure 7.15.2 (c)

MAJOR SYSTEM DESIGN

ASSESS CRITICAL LOCATIONS
- UPSTREAM OF INTERSECTION
- ON FLAT GRADES
- APPROACHING SAGS
- UPSTREAM OF DRIVEWAYS

5

DETERMINE FREEBOARD &
MAXIMUM WATER DEPTH
LIMITATIONS

COMMENCING FROM
THE TOP OF THE
CATCHMENT

CALCULATE PEAK DISCHARGE (Q_T)
FOR TOTAL CATCHMENT
AT THE CRITICAL LOCATION

CALCULATE PERMISSIBLE ROAD OR
OVERLAND FLOW CAPACITY
AT CRITICAL LOCATIONS - Q_{LIM}

CALCULATE Q_{PU} BASED ON SUM
OF Q_P VALUES UPSTREAM

GO TO NEXT
CRITICAL
LOCATION

CALCULATE POSSIBLE SURFACE
FLOW IN THE SECTION
UPSTREAM OF THE CRITICAL
LOCATION = Q_{SURF}

CHECK
$Q_{SURF} > Q_{LIM}$ — NO

YES

CALCULATE PIPE CAPACITY
REQUIRED IMMEDIATELY UPSTREAM
OF CRITICAL LOCATION, $Q_P = Q_T - Q_{LIM}$

CHECK
$Q_P > Q_{PU}$ — NO

YES

REQUIRED PIPE
DISCHARGE CAPACITY IS
BASED ON Q_{PU}, OR PROVIDE
SURCHARGE STRUCTURE
WITHIN THE SECTION

REQUIRED PIPE
DISCHARGE CAPACITY IS
BASED ON Q_P

DETERMINE Q_{gs} REQUIRED
IN THE SECTION UPSTREAM
OF CRITICAL LOCATION
$Q_{gs} = Q_P - Q_{PU}$

CHECK IS
REQUIRED GULLY
INLET CAPACITY
FEASIBLE — NO

ADJUST No. OF
GULLY INLETS &/OR
CATCHMENT TO THE
CRITICAL LOCATION

YES

CHECK IS
THERE ANOTHER
CRITICAL LOCATION — YES

NO

4

Figure 7.15.2 (c) – Flow chart for initial design assessment

7.16   Hydraulic calculations

7.16.1   General

The detailed hydraulic grade line (HGL) method is recommended for the analysis of underground stormwater pipe systems. It is further recommended that this be based on an analysis proceeding from downstream to upstream through the system.

The above method is logically consistent with the concept of backwater analysis and enables the prediction of hydraulic grade line and water surface level throughout the system. It permits control points or points of potential surcharge to be visualised and for system layout and pipe sizes to be optimised.

Guidance on the selection of a starting hydraulic grade level (tailwater level) at the outlet, or downstream end of the system is given in section 7.16.5 and Chapter 8. The determination of friction losses in pipes should be based on the use of Manning's Equation.

There are some circumstances where hydraulic design on an upstream to downstream basis may be necessary. Where a branch line on flat terrain enters a trunk drainage system, a critical hydraulic grade level situation may occur because of the possibility of surcharging in the branch line system. Accordingly, the branch line may be designed on an upstream to downstream basis and the hydraulic grade line predicted at the trunk line is then used as a control for subsequent downstream to upstream calculations in the trunk line system.

In circumstance where a new drainage network crosses more than one land use category resulting in a change in design standard (i.e. some parts of the Minor Drainage System are designed to a 39% AEP (1 in 2 year) standard while other parts are designed to a 10% AEP standard) then the network shall be analysed for each AEP.

7.16.2   Pipe and structure losses

Losses due to friction in pipes may be expressed as:

$$h_f \;=\; S_f \,.L \tag{7.10}$$

where:

h_f = head loss in pipe due to friction (m)
S_f = friction slope (m/m)
L = length of pipe reach (m)

Losses due to obstructions, bends or junctions in pipelines may be expressed as a function of the velocity of flow in the pipe immediately downstream of the obstruction, bend or junction as follows:

$$h_s \;=\; K.V_o^2/2g \tag{7.11}$$

where:

h_s = head loss at obstruction, bend or junction (m)
K = pressure change coefficient (dimensionless)
V_o = velocity of flow in the downstream pipe (m/s)
g = acceleration due to gravity (9.80 m/s^2)
$V_o^2/2g$ = velocity head (m)

Pressure change coefficients K (sometimes referred to as structure loss coefficients) are dependent on many factors, for example:

- junction structure geometry
- pipe diameters
- bend radius
- angle of change of direction
- relative diameter of obstructions.

Section 7.16.8 of this Manual discusses pressure change coefficients in detail.

7.16.3   Hydraulic grade line and total energy line

The HGL is a plot of the pressure head at any point in a pipeline.

The HGL may be thought of as the 'Effective Water Level' in the system—the level to which water would rise in an open-topped vertical pipe inserted into the drainage line in a manner that did not cause energy/pressure loss. Note however that at access chambers and kerb inlets the water surface elevation (WSE) is normally higher than the theoretical HGL because the latter reflects the HGL immediately upstream of the structure. Determination of the HGL does not distinguish between pressure gains or losses at the inlet to or outlet from the structure, but relates to the structure as a whole. Figure 7.16.1 explains this effect.

Pressure head is normally lost in both pipes and access chambers due to friction and turbulence, and the form of the HGL is therefore a series of downward sloping lines over pipe lengths, with steeper or vertical drops at access chambers.

In some circumstances there may be a pressure gain and therefore a rise in the HGL at a structure. In these cases the gain should be taken into account in the hydraulic calculations.

The assumption of straight hydraulic grade lines is usually made. This is not strictly correct but is sufficiently accurate in most cases.

The level and grade of the HGL varies with flow. For design purposes the HGL calculated and plotted on the longitudinal section is that applicable to the flow resulting from the Design Storm. For a pipe to run full, the obvert must be at or below the HGL If a pipe runs part-full, the HGL is at the water surface in the pipe.

The velocity of flow and accordingly the discharge capacity of a pipe is a function of the Hydraulic Grade (slope of the HGL) not the actual pipe grade.

A pipe may be located at any grade and at any depth below the HGL without altering the velocity and flow in the pipe subject to the grade limitations outlined in section 7.12. Hence, pipe grade may be flattened to provide cover under roads, or clearance under other services, without sacrificing flow capacity, provided sufficient head is available.

The HGL and the Water Surface Elevation (WSE) must be below the surface level at pits and kerb inlets, or the system will surcharge.

The level of the Hydraulic Grade Line (HGL) for the design storm should be calculated at the following locations:

- upstream and downstream side of every kerb inlet or access chamber
- at points along a pipe reach where obstructions, penetrations or bends occur

- where a branch pipeline is connected to the pipe system without an access chamber.

Note: That branch pipelines without access chambers should only be constructed if so approved by the relevant local authority.

It is recommended that designers check that the elevation of the total energy line falls progressively as flow passes down through the drainage system. This is an important check that should be undertaken where the drainage system is complex and where the configuration of pipes/structures etc. does not conform with the structure loss charts available.
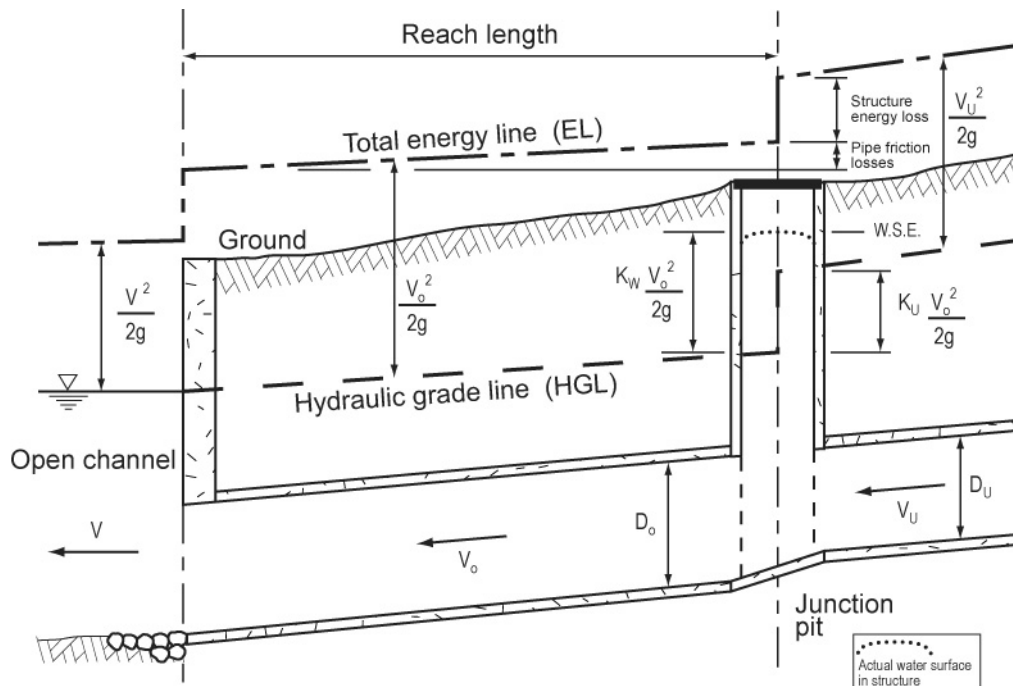
Figure 7.16.1 – Hydraulics for a single pipe reach

The total energy line under steady flow conditions is located above the HGL by an amount equal to the velocity head. This is shown diagrammatically in Figure 7.16.1. Note that under quiescent conditions in a pond or storage with no flow the HGL and energy line coincide.

7.16.4   Methods of design

Pipeline design by the HGL method is most conveniently carried out by working upstream from the outlet because:

- The outlet is often the only point for which the HGL may be readily determined.

- Head losses in pits and gullies are expressed as a function of the velocity in the downstream pipe—hence the pipe downstream of each structure must be designed before the head loss in that structure can be determined.

Designing drainage systems from downstream to upstream is considered the preferred methodology.

Experienced designers however, may adopt a procedure of designing from upstream to downstream and this may be essential in parts of the drainage system located in flat or undulating terrain. The procedure is as follows:

(a) Assess the critical start point or points in the system (e.g. sag gully inlet).

(b) Allow minimum freeboard to determine the permissible water surface level in that pit, (normally 150 mm).

(c) Select pipe diameters and depths to suit hydraulic and economic considerations.

(d) Calculate hydraulic grade line proceeding downstream from the starting water surface level determined in (b) above.

(e) The procedure is iterative, however experience should reduce the number of iterations.

Both procedures for detailed calculation are outlined in the flow charts contained in figures 7.16.2 and 7.16.3.

Notes (Figure 7.16.2):

1. The downstream HGL should be derived from the tailwater level in the receiving waters (see section 7.16.5) or from the HGL calculated in the structure downstream.

2. The pipe size selected becomes D_o for the next structure upstream.

3. In this case 150 mm freeboard has been allowed above the WSE. This limit may need to be modified to suit other constraints including the hydraulics of upstream or lateral pipes.

4. The performance of a reach is dependent on the characteristics of the other reaches. Accordingly the most economic design is not that which optimises each reach but that which performs best overall.

5. During the **design** of piped drainage systems it is usually advisable to assume the HGL does not fall below the obvert level of the pipe (i.e. the pipe is flowing full). The reasons for this are summarised below:

    *   If the pipe is not flowing full during the design discharge, then this is likely to mean that the pipe is over-designed.

    *   The most common situation where the tailwater level is below the pipe obvert is at the pipe outlet. In most cases this results from a low-gradient pipe (which otherwise would have been flowing full) spilling out into an open channel. In such circumstances the flow conditions at the pipe outlet often consist of rapidly varied flow, where the streamlines are converging and have a steep gradient. Within such flow conditions the laws of hydrostatics (on which most 1-dimensional numerical models are based) begin to break-down, thus numerical errors can be expected. Fortunately it has been found that if it is assumed that the HGL at the pipe outlet is at the elevation of the pipe obvert, then a standard hydraulic analysis will achieve the correct HGL at a distance of 10 to 20 pipe diameters upstream of the outlet.

    *   If the calculated HGL is below the pipe obvert within a junction chamber, then at the peak design discharge it often takes only a minor debris blockage, or a structural flaw in the chamber's construction, to cause the pit's energy loss to increase to the point where the water level in the chamber (WSE) reaches the obvert level of the discharge pipe.

    However, when **analysing** the hydraulic grade line of the final design layout or an existing pipe network, there are occasions where this rule is inappropriate, as outlined below:

    *   If the pipe is flowing partially full for the full length of the upstream reach, from which it then discharges into an open channel without the water surface dipping rapidly (i.e. the water does not undergo rapidly varied flow conditions) then the pipe outlet can be analysed using open channel hydraulics, and the pipe network should be analysed assuming the HGL at the outlet is equal to the actual water elevation.

    *   If the pipe is flowing partially full for the full length of the pipe's reaches both upstream and downstream of a chamber, then the pipe network can be analysed as an open channel.

    *   If the hydraulic analysis is being performed for a discharge significantly less that the design discharge, and parts of the pipe network are flowing partially full, then the hydraulic analysis should assume the starting HGL at the end of each pipe reach does not fall below the 'normal' or

'uniform' flow depth (i.e. the flow depth assuming uniform flow conditions within a long pipe of constant gradient).

START → SELECT DOWNSTREAM (d/s) H.G.L. OR TAILWATER LEVEL, SEE NOTE 1 ← d/s H.G.L. = u/s H.G.L.

INPUT REACH FLOW Q_O → SELECT TRIAL REACH PIPE DIAMETER (D_O) SEE NOTE 2

CALCULATE PIPE FRICTION HEAD LOSS (H_F) AND H.G.L.

SELECT OBVERT LEVEL FOR THIS PIPE REACH

TEST: H.G.L. > OBVERT LEVEL — NO → H.G.L. = OBVERT

YES

TEST: H.G.L. > S.L. - 300mm — YES →

NO

INPUT Q_{LR}, Q_{LL} Q_g etc. IF ANY → SELECT TRIAL PIPE DIAMETER AT UPSTREAM STRUCTURE D_U, D_{LR}, D_{LL}

CALCULATE K_U & K_W FROM CHARTS CALCULATE u/s H.G.L. & u/s W.S.E.

TEST: u/s W.S.E. > S.L. - 150mm SEE NOTE 3 — YES TRY AGAIN →

NO

NEXT REACH? SEE NOTE 4 — YES →

NO

END

LEGEND
d/s = DOWNSTREAM
u/s = UPSTREAM

Figure 7.16.2 – Hydraulic grade line design method flow chart – designing from downstream to upstream (Preferred method)
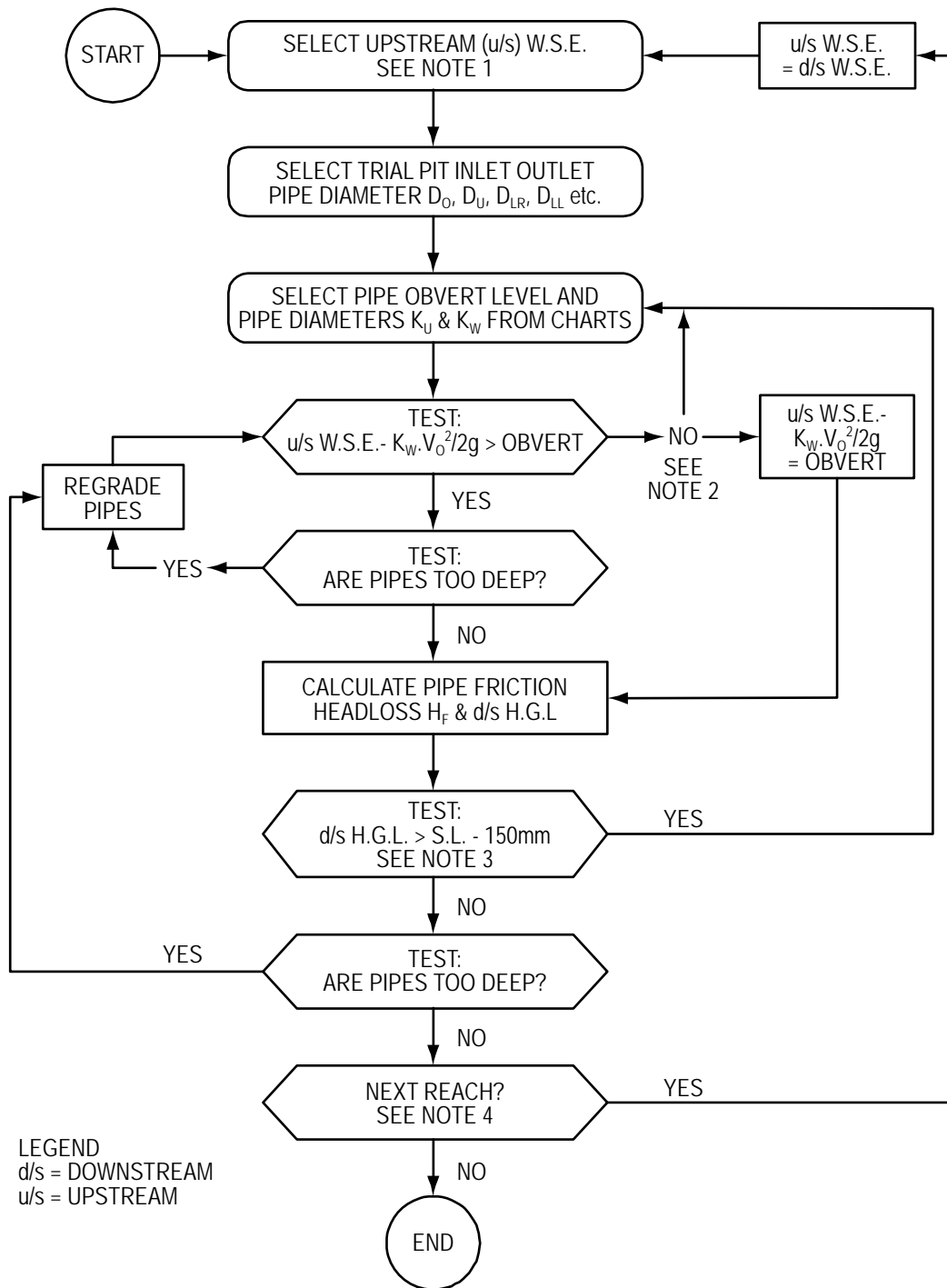
Figure 7.16.3 – Hydraulic grade line design method flow chart – designing from upstream to downstream

Notes (Figure 7.16.3):

1. The upstream WSE should not be higher than the surface level less 150 mm.

2. Conditions may be such that regardless of the outlet diameter this condition cannot be satisfied. To avoid excessive looping check this first.

3. The final hydraulic grade line level at the downstream pit may be set at levels other than that specified provided that outfall conditions are known.

4. The performance of a reach is dependent on the characteristics of the other reaches. Accordingly the most economic design is not that which optimises each reach but that which performs best overall.

7.16.5   Starting hydraulic grade level

In order to carry out a Hydraulic Grade Line backwater analysis for an urban piped drainage system it is necessary to determine a starting HGL or downstream HGL for the calculations.

This section of the Manual deals with determining a starting HGL for the discharge conditions most commonly encountered and should be read in conjunction with the information supplied in section 8.3 of this Manual.

The designer should in all cases give careful consideration to the adopted starting HGL and if necessary, liaise with the relevant regulating authority to establish an agreement.

(a)   Outfalls generally

During subcritical outflow conditions the position of the starting HGL will depend upon the relationship between the calculated tailwater (TWL) in the receiving waters, the critical depth (d_c) of the particular flow under consideration in the outfall pipe and the obvert level (OL) of the pipe. The following general rules should apply (Figure 7.16.4):

(a) If TWL > OL, then start HGL = TWL

(b) If d_c ≤ TWL ≤ OL, then start HGL = OL

(c) If TWL < d_c (i.e. free outfall), then start HGL = the normal flow depth (d_n) in the outfall pipe for the given flow rate
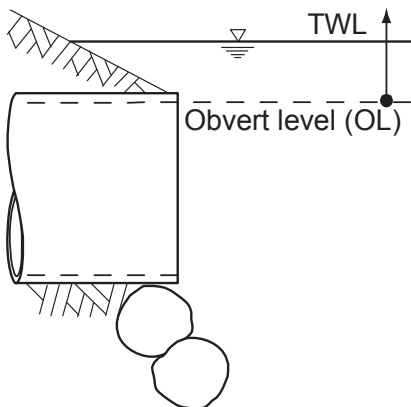
Figure 7.16.4 (a) – Tailwater above obvert

Starting HGL = TWL
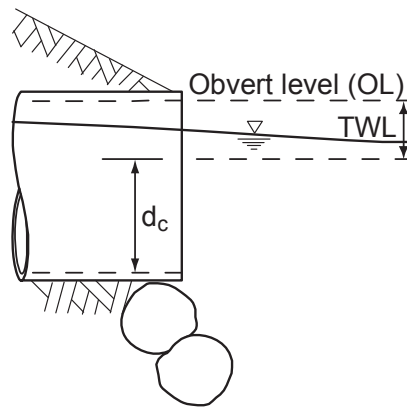
Figure 7.16.4 (b) – Tailwater below obvert

Starting HGL = OL
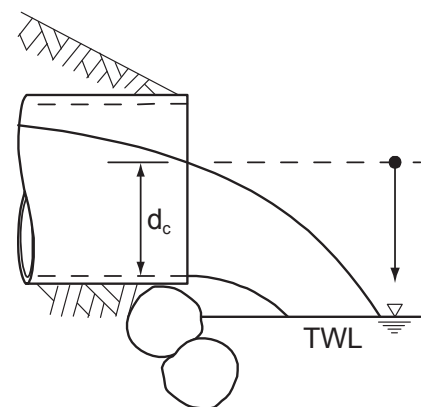
Figure 7.16.4 (c) – Tailwater below pipe invert

Starting HGL = normal depth in pipe (not 'd$_c$')

The startling HGL conditions presented in figures 7.16.4 (b) and (c) do not necessarily apply to the following cases:

• The analysis of discharges from pipes where the pipe is flowing partially full for the full length of the upstream reach, and the water level is significantly below the obvert of the pipe. In such case the starting HGL should be the greater of the actual water level, or the 'normal' or 'uniform' flow depth within the pipe (see Note 5 for Figure 7.16.2).

• The analysis of discharges from short conduits such as most culverts.

(b)   Existing pipe network

When connecting into an existing pipe network, designers should determine the HGL of the downstream system for the design ARI. Full account of structure losses should be made in the existing system.

If a full HGL analysis is considered impractical due to the length or complexity of the existing pipe network, then an appropriate estimation of the HGL in the existing network must be made. When determining an estimation of the starting HGL, consideration should be given to:

- the existence of a downstream surcharge chamber (if any)
- the existence of a downstream pipe possibly operating under partial flow (such a condition may be unlikely during a design storm event)
- otherwise, with approval from the local authority, adoption of a starting water level 150 mm below the grate/inlet elevation (minor design storm conditions only).

In any case, modifications to an existing drainage system, including changes to inflows, must not compromise the system's performance relative to the desired performance standard without approval from the relevant local authority.

(c)   Future pipe network

If design of a piped system is being undertaken in the upstream section of a catchment prior to the design of the downstream system, then the designer should undertake sufficient preliminary planning of the downstream system to permit design of the upstream system.

This planning should incorporate preliminary road layouts and levels along with preliminary drainage line locations and levels. To allow for possible inaccuracies associated with such a preliminary design, a factor of safety may need to be allowed. For example:

- allow a nominal height above the assessed HGL at the proposed connection to the downstream system
- adopt the HGL equal to the natural surface at the location of the next downstream structure in the proposed future pipe network, or
- adopt a starting HGL as approved by the local authority.

7.16.6   Freeboard at inlet and junctions

For the design of underground systems a freeboard should be provided above the calculated WSE to prevent surcharging and to ensure that unimpeded inflow can occur at kerb inlets. Table 7.16.1 provides recommendations for freeboard for kerb inlets and access chambers.

Table 7.16.1 – Minimum freeboard recommendations for kerb inlets and pits

Situation	Recommendation
Kerb inlet on grade	Freeboard = 150 mm below invert or kerb and channel [1] & [2]
Kerb inlet in sag	Freeboard = 150 mm below invert or kerb and channel [1]
Field inlet	Freeboard = 150 mm below top of grate or lip of inlet
Access chambers or junction structure [3]	Freeboard = 150 mm below top of lid

Notes (Table 7.16.1):

[1]   Where the channel is depressed at a kerb inlet the freeboard should be measured from the theoretical or projected invert of the channel.

[2]   Where an inlet is located on grade the freeboard should be measured at the centreline of the chamber.

[3]   Where it is necessary for the HGL to be above the top of an access chamber or junction structure, a bolt-down lid should be provided.

The maximum permitted WSE should allow for the head loss resulting from surface inflow through grates etc. into the structure being considered. The charts contained in Appendix 1 permit the determination of water surface elevation coefficient K_w for many types of structures.

Where an appropriate chart is not available it is recommended that the WSE be arbitrarily adopted at the height above the calculated HGL in accordance with equation 7.12.

$$WSE - HGL = 0.3\ V_u{}^2/2g \qquad\qquad (7.12)$$

where:

$V_u{}^2/2g$ =   upstream velocity head

The freeboard recommendations should be applied as detailed in Table 7.16.2.

Table 7.16.2 – Application of freeboard recommendations

Design conditions	Minor storm analysis		Major storm analysis	
	HGL and WSE calculations required	Freeboard to WSE	HGL and WSE calculations required	Freeboard to WSE
(a) Underground system designed for minor storm. Overland flow check for major storm requires no increase in size of pipe system.	Yes	As per Table 7.16.1	No	NA (See Note 2)
(b) Underground system designed for minor storm. Overland flow check for major storm requires increase in size of pipe system.	Yes	As per Table 7.16.1	Yes	As per Table 7.16.1 (See Notes 1, 2 and 3)
(c) Underground system designed for major storm.	No	NA	Yes	As per Table 7.16.1 (See Notes 2, 3 and 4)

Notes (Table 7.16.2):

1.   The major storm HGL may only need to be calculated from the point where the increase in pipe size is required downstream to the outfall e.g. downstream from a trapped sag.

2.   The freeboard requirements to the floor level of adjacent buildings etc. as detailed in Table 7.3.5 are applicable to the overland or street flow.

3.   Notwithstanding the presence of overland or street flow on the surface it is recommended that for design purposes the calculated WSE in the underground pipe system not exceed the requirements of Table 7.16.1.

4.   This situation will apply where the opportunity for overland flow is nil or extremely limited.

7.16.7   Pipe capacity

The capacities of stormwater pipes flowing full, but not under pressure, should be calculated using Manning's equation.

Manning's equation:

$$V = (1/n).R^{2/3}.S_f^{1/2} \qquad\qquad (7.13)$$

where:

V =   Velocity (m/s)

A =   Area of flow (m^2)

P =   wetted perimeter (m)

R =   Hydraulic radius = A/P (m)

S_f =   Friction slope (m/m)

n =   Manning's roughness coefficient

Table 7.16.3 gives recommended surface roughness coefficients for the types of pipes encountered in urban stormwater design.

Table 7.16.3 – Recommended values for surface roughness (average pipe condition)

Type of pipe	Manning's n
Reinforced concrete (RCP and RCBC)	0.013
Fibre reinforced cement (FRC)	0.013
UPVC	0.011
GRP	0.011

More information on surface roughness can be found in AS 2200; ARR-1998, Technical note 8; p.325, Argue (1986) Table 6.1, and the Concrete Pipe Association web site.

Design chart A1-1 is provided in Appendix 1 for the solution of Manning's equation. The nomograph is based on nominal internal diameters and a Manning's roughness, n = 0.013. Designers should check actual internal diameters for the type and class of pipe being designed and make the necessary correction where this is significant.

Stormwater systems are not normally designed to flow under pressure, but whenever the HGL rises above ground level and the junction pits are fitted with bolt down lids, the system will become pressurised. The analysis of pressurised systems should be checked using software that takes account of pressurised flow.

7.16.8   Pressure changes at junction stations

(a)   General

Pressure loss (or head loss) at junctions may be expressed as a function of the velocity head of the flow in the conduit downstream of the junction, $V_o^2/2g$:

thus: $h_s = K \cdot V_o^2/2g$ (7.14)

where:

h_s =   pressure change at a structure
K =   pressure change coefficient

The charts contained in Appendix 1 of this Manual provide pressure change coefficients for junction types commonly encountered in urban drainage design.

Note that where a structure has lateral as well as through flow the pressure change coefficient which applies to the through (main) line may be different to that for the lateral line i.e. K_U may not equal K_L.

The appropriate charts should be used to determine correct values of K_U and K_L. The pressure change coefficients K_U and K_L should be applied to the velocity head $V_o^2/2g$ in the **outlet** pipe from the structure.
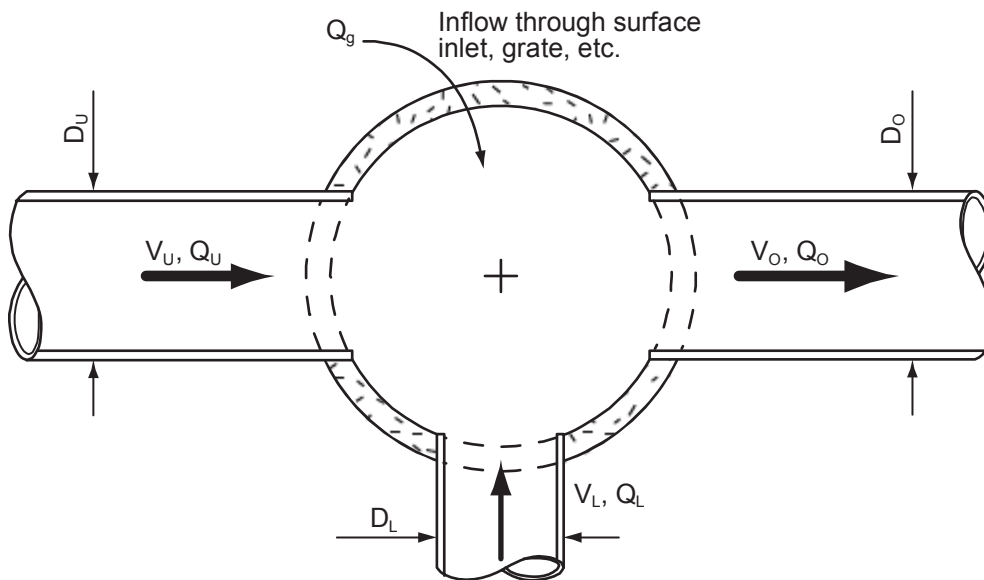
Figure 7.16.5 – Nomenclature at structures

START → SELECT PIT CONFIGURATION TO LOCATE CHART(S) TO BE UTILISED

INPUT Q_U, Q_O, Q_g AND Q_L AS REQUIRED → SELECT TRIAL D_U, D_O, AND D_L

CALCULATE CHART(S) ENTRY RATIOS
$$\frac{Q_U}{Q_O} \quad \frac{Q_g}{Q_O} \quad \frac{Q_L}{Q_O} \quad \text{AND} \quad \frac{D_U}{D_O} \quad \frac{D_L}{D_O}$$
AS REQUIRED FOR THE CHART(S) USED

SET INITIAL RATIOS, S/D_O, $[B/D_O]$ AS REQUIRED FOR THE CHART(S) USED

FROM CHART(S) DETERMINE K_W

REVISE RATIOS S/D_O AS REQUIRED FOR THE CHART(S) USED

TEST:
DO THE CURRENT VALUES OF K_U AND K_W VARY FROM THE PREVIOUS VALUES BY LESS THAN 10%?

NO

YES

CALCULATE K_U

END

Figure 7.16.6 – Coefficients K_U and K_W calculation procedure flow chart

The flow chart in Figure 7.16.6 can be used to determine both the HGL and the WSE at the junction. The values of K_U and K_W should be applied to the velocity head in the outlet pipe i.e. $V_o^2/2g$.

(b)   Benching

Benching of the floors of junction pits leads to a general reduction in losses and promotes improved hydraulic efficiency. Table 7.16.4 provides an indication of the potential decrease in pressure change coefficient that can be achieved in square pits as a result of benching (Johnston et al 1990, Dick and Marsalek 1985, and Lindvall 1984). It should be emphasised that these improvements have been measured for square pits. Testing of circular pits (but without benching) would indicate that these improvements may be less for circular pits.

It is noted that benching reduces pit losses not by directing flows towards the outlet, but by reducing the effective 'deadwater' volume in the pit and reducing flow contraction at the entrance to the outlet pipe. Thus benching reduces flow expansion within the chamber and reduces turbulence around the entrance of the outlet pipe.

Table 7.16.4 – Potential decrease in pressure change coefficient as a result of benching

Access chamber type [3]	Potential decrease in pressure change coefficient (%)	
	Half-height benching [1]	Full-height benching [2]
Straight through	30	40
90^o bend	20	40
Tee chamber with lateral inflow less than 50%	Nil	Nil
Tee access chamber with lateral inflow approximately 50%	Nil	10
Tee access chamber with lateral inflow approximately 100%	20	40

Notes (Table 7.16.4):
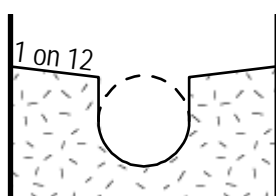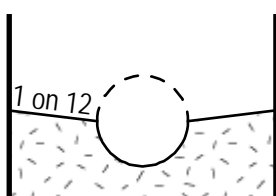
Note [3]:
Results based upon testing of square pits.

Note [1]: Figure 7.16.7 (a) – Half-height benching

Note [2]: Figure 7.16.7 (b) – Full-height benching

(c)   Use of pressure change charts in Appendix 1

Various pressure change coefficient design charts are provided in Appendix 2 of this Manual along with detailed discussion on the application of these charts.

It is important to note that the Hare Charts (Hare, 1980), Missouri Charts (Sangster et al, 1958) and the Cade and Thompson Charts (Cade & Thompson, 1982) have been prepared predominantly for values of B/D_o approximately equal to 2 (refer to Figure 7.16.5 for definition of B and D_o). In cases where $B/D_o > 2$, it can be expected that values of K_U and K_W will likely be greater than those given by these charts.

7.16.9   Inlets and outlets

(a)   Entrance losses

Where the inlet structure is an endwall (with or without wingwalls) to a pipe or culvert, an allowance for head loss should be made. Table 7.16.5 provides entry loss coefficients K_e to be applied to the velocity head for the downstream pipe or culvert, where the approach velocity is effectively zero. Where there is an appreciable approach velocity the entrance loss coefficient should be applied to the absolute value of the difference in the two velocity heads as presented in equation 7.15.

$$\Delta H = K_e . ABS[(V_o^2/2g) - (V_u^2/2g)] \tag{7.15}$$

where:

ΔH	=	energy (head) loss at entry
K_e	=	entry loss coefficient
V_o	=	average flow velocity within pipe or culvert (m/s)
V_u	=	upstream velocity (m/s)

The pressure change coefficient (K) for use in a HGL analysis may be determined from equation 7.16.

$$K = K_e + 1 \tag{7.16}$$
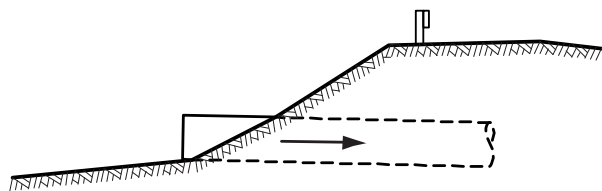
Figure 7.16.8 (a) – Projecting from fill
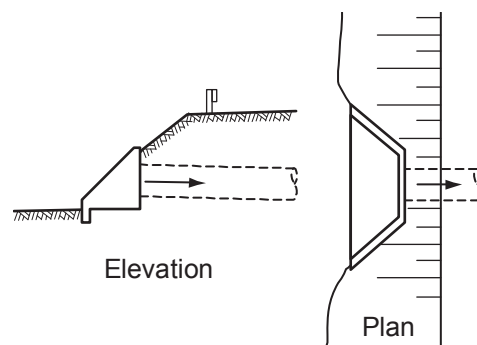
Figure 7.16.8 (b) – Headwall with wing walls
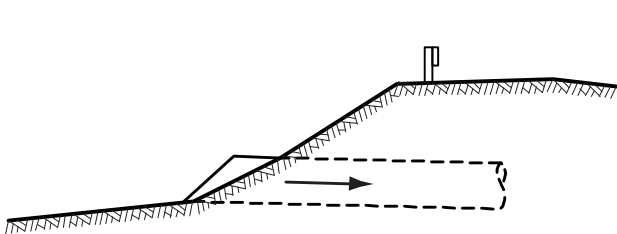
Figure 7.16.8 (c) – Mitred to conform to fill slope
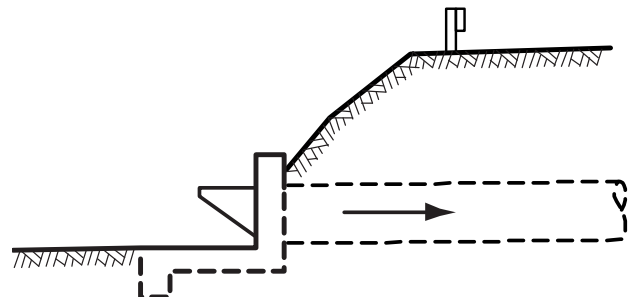
Figure 7.16.8 (d) – Hooded entrance

Table 7.16.5 – Entrance (energy) loss coefficients [1]

Type of structure and design of entrance	Coefficient K_e
Concrete pipe:	
Projecting from fill, socket end (groove end)	0.2
Projecting from fill, square cut end	0.5
Headwall or headwall and wing walls:	
• socket end of pipe (groove end)	0.2
• square edge	0.5
• rounded (radius = D/12)	0.2
• mitred to conform to fill slope	0.7
• end section conforming to fill slope.	0.5
Hooded inlet projecting from headwall	Note [2]
Corrugated metal pipe:	
Projecting from fill (no headwall)	0.9
Headwall or headwall and wing walls square edge	0.5
Mitred to conform to fill slope	0.7
End section conforming to fill slope	0.5
Reinforced concrete box:	
Headwall parallel to embankment (no wing walls):	
• square edged on 3 edges	0.5
• rounded on 3 edges to radius of 1/12 barrel dimension.	0.2
Wing walls at 30° to 70° to barrel:	
• square edged at crown	0.4
• crown edge rounded to radius 1/12 barrel dimension.	0.2
Wing walls at 10° to 25° to barrel:	
• square edged at crown.	0.5
Wing walls parallel (extension of sides):	
• square edged at crown.	0.7

Note (Table 7.16.5):

[1]   Adapted from Hee (1969).

[2]   Refer Argue (1960) and O'Loughlin (1960).

(b)   Exit losses

It is a common misconception that the full velocity head is always lost at a pipe or culvert exit. Exit losses are primarily a result of the energy required to produce 'induced' flow currents within the outlet channel or water body. Exit loss is a function of the change in velocity and the degree of 'confinement' of the outlet jet (i.e. existence of channel bed and walls that restrict expansion of the outlet jet).

Exit loss (energy loss) may be determined from equation 7.17.

$$\Delta H = K_{exit} \left[(V_u^2/2g) - (V_o^2/2g) \right] \qquad (7.17)$$

where:

ΔH = energy (head) loss at exit
K_{exit} = exit loss coefficient (see below)
V_u = average flow velocity within pipe or culvert (m/s)
V_o = average flow velocity downstream of the outlet (m/s)

(i)   Unconfined outlet jet (Figure 7.16.9)

$$K_{exit} = 1.0$$
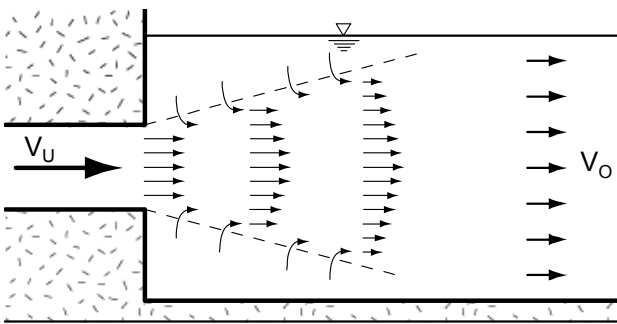
Figure 7.16.9 (a) – Side view of flow expansion
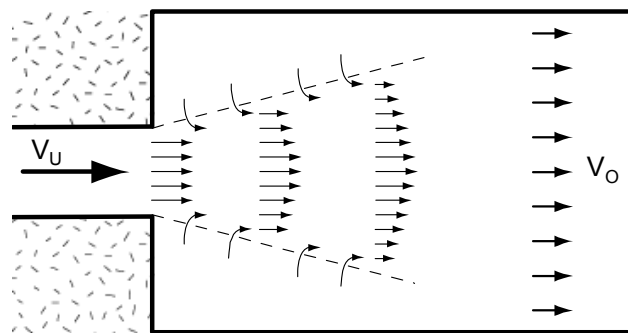
Figure 7.16.9 (b) – Plan view of lateral flow expansion

(ii)   Outlet jet confined on one side (Figure 7.16.10)

Typically this occurs when the pipe/culvert discharges onto a solid (scour resistant) channel bed with the same invert as the outlet pipe.

$$K_{exit} = 0.7\text{–}0.8$$

In culvert analysis it is typical to adopt an exit loss coefficient of 0.7 based on the assumption that a scour-resistant outlet pad exists that prevents the formation of an outlet scour hole. If a scour hole is allowed to form, then an exit loss coefficient of 1.0 would be more appropriate.
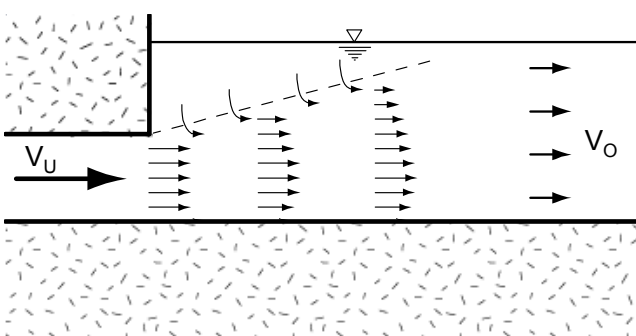
Figure 7.16.10 (a) – Side view of flow expansion
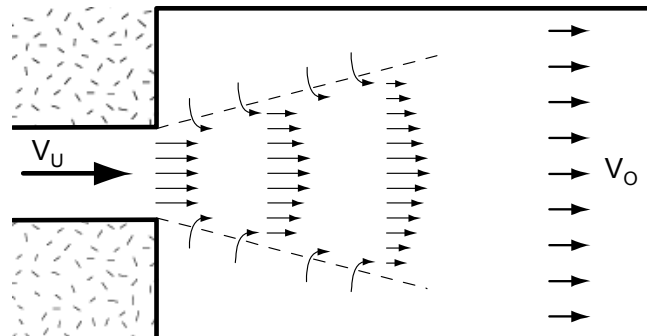
Figure 7.16.10 (b) – Plan view of lateral flow expansion

(iii)   Outlet jet confined on two sides (Figure 7.16.11)

The example shown in Figure 7.16.11 expansion of the outlet jet is confined on both the bed and one outlet channel wall.

$$K_{exit} = 0.5\text{--}0.7$$
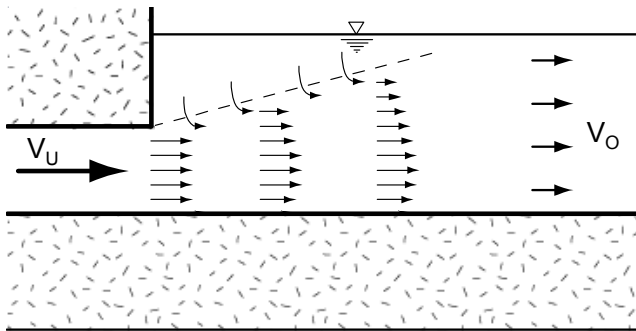
Figure 7.16.11 (a) – Side view of flow expansion
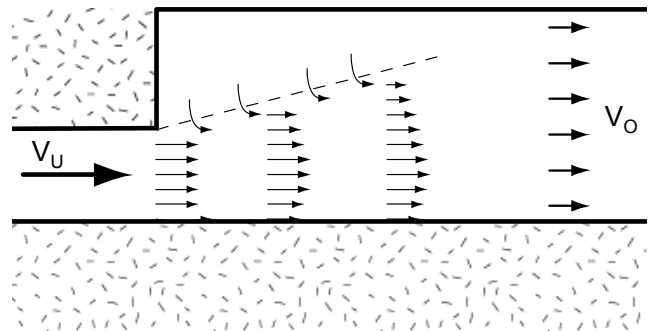
Figure 7.16.11 (b) – Plan view showing lateral flow expansion limited to one side

(iv)   Outlet jet confined on three sides (Figure 7.16.12)

The example shown in Figure 7.16.12 expansion of the outlet jet is confined on both the bed and both outlet channel walls.

$$K_{exit} = 0.3\text{--}0.5$$
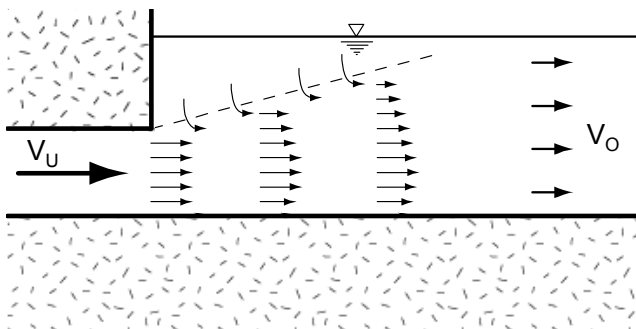
Figure 7.16.12 (a) – Side view of flow expansion
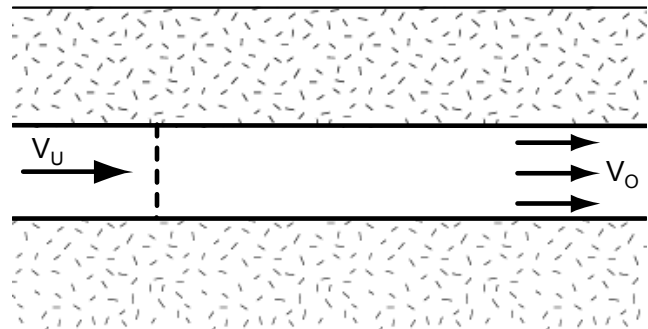
Figure 7.16.12 (b) – Plan view showing no lateral flow expansion

The pressure change coefficient (K) for use in a HGL analysis may be determined from equation 7.18 (note for the above cases K will be negative).

$$K = K_{exit} - 1 \qquad (7.18)$$

It is noted that the above analysis assumes the outlet (V_u) is not less than the downstream velocity (V_o).

7.16.10 Bends

Under certain circumstances it may be permissible to deflect the pipeline (either at the joints or using precast mitred sections) to obviate the cost of junction structures and to satisfy functional requirements, e.g. negate need for access chambers on playing fields.

Where pipelines are deflected an allowance for energy loss should be made. The energy loss is a function of the velocity head and may be expressed as:

$$h_b = K_b \, (V^2/2g) \tag{7.19}$$

where:

h_b = head loss through bend

K_b = bend loss coefficient

Note: *That the head loss due to the bend is additional to the friction loss determined for the reach of pipe being considered.*

Figure 7.16.13 should be used to determine the bend loss coefficient at a gradual bend.
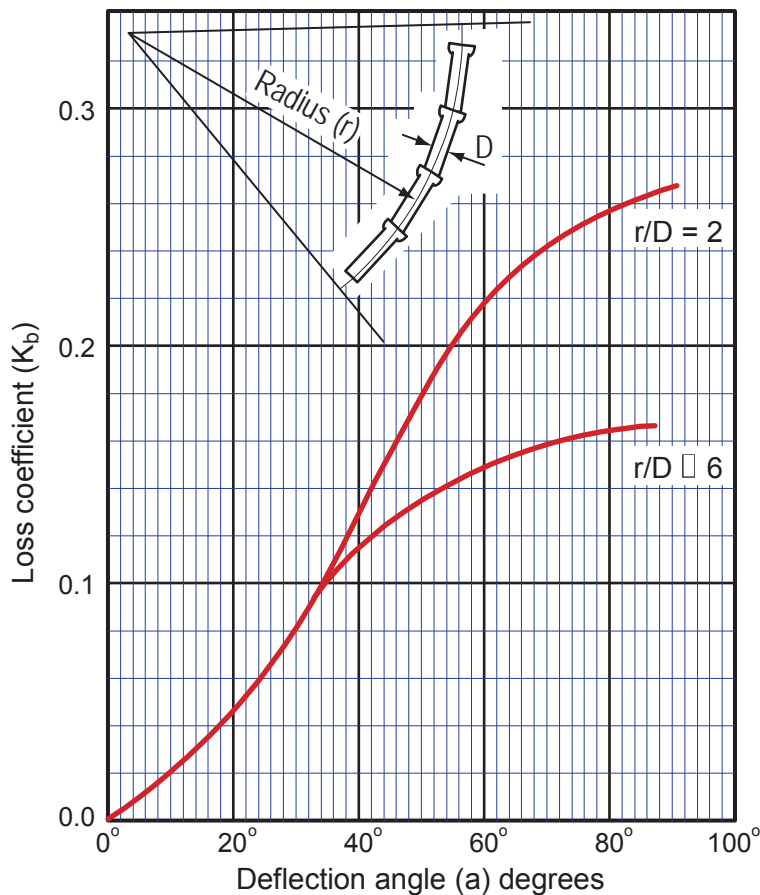
Figure 7.16.13 – Bend loss coefficients (Source: DOT, 1992)

At mitred fittings the pressure loss coefficients in Table 7.16.6 are recommended.

Table 7.16.6 – Pressure loss coefficients at mitred fittings

Type	K_b
90° double mitred bend	0.47
60° double mitred bend	0.25
45° single mitred bend	0.34
22½° single mitred bend	0.12

Source: ARR, 1987 (p.327)

The coefficients presented in Table 7.16.6 are used to determine the HGL at a mitred bend in a pipe. The mitred bend will be formed either as a single or double mitre by the manufacturer.

For double mitre bends, the length of the intermediate length of pipe should ideally be 1.5 times the pipe diameter (measured on the outside of the bend) and no more than 6 times the pipe diameter

7.16.11 Obstructions or penetrations

An obstruction or penetration in a pipeline may be caused by a transverse (or near transverse) crossing of the pipe by a service or conduit, e.g. sewer or water. Where possible, such obstructions should be avoided as they are likely sources of blockage by debris and damage to the service. To facilitate the removal of debris, it is suggested that an access chamber be provided at the obstruction or penetration.

The pressure change coefficient K_p at the penetration is a function of the blockage ratio. Figure 7.16.14 may be used to derive pressure change coefficients which are then applied to the velocity head.
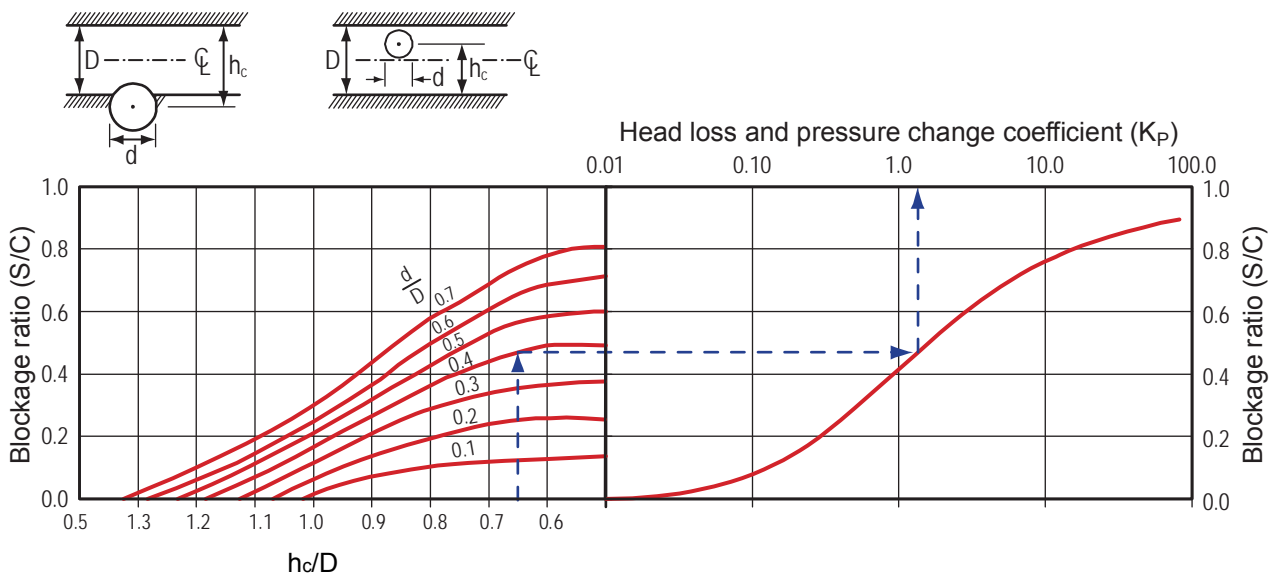
Figure 7.16.14 – Penetration loss coefficients (Source: Black, 1987b)

$$h_p = K_p (V^2/2g) \qquad (7.20)$$

where:

      h_p = head loss at penetration

      K_p = pressure change coefficient of penetration

Where an access chamber is provided at an obstruction or penetration it is necessary to add the structure loss and the loss due to the obstruction or penetration, based upon the velocity in the downstream pipe.

While no method is currently available for the determination of pressure change at an obstruction or penetration at a manhole, it is suggested that the use of the above design data based on the assumption that the downstream pipe is continuous across the manhole (enabling h_c, D and V to be determined) will give reasonable if conservative results.

7.16.12   Branch lines without a structure

It is sometimes necessary to construct a branch line or lateral pipe connection to another pipeline without providing a junction structure. Where possible such connections should be avoided.

Where branch connections are unavoidable, appropriate allowance for head loss at the junction should be made.

Designers should be aware that the pressure change coefficient and therefore the head loss at the junction may be different for the main line and the branch line. Pressure change coefficients for junctions with branch line connections should be determined from Design Charts in Appendix 1, an example of which is provided in figures 7.16.16 and 7.16.17.
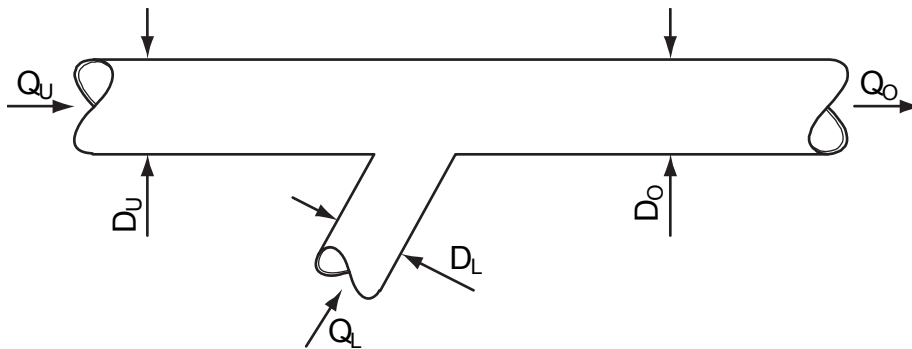
Figure 7.16.15 – Branch line nomenclature

Both the pressure change coefficients K_L (branch line) and K_U (main line) should be applied to the velocity head of the downstream combined flow $V_o^2/2g$ to determine the head loss applicable to each line.

A junction node label or structure number should be given at the connection between the main line and the branch line.

It is recommended that the diameter of field constructed branch lines not exceed 50% of the diameter of the main line. Where larger diameter branches are required it is recommended that an access chamber be installed.

Note: *The coefficients presented in Miller (1990) and figures 7.16.16 and 7.16.17 are 'energy loss' coefficients and therefore, a conversion must be made to obtain the appropriate pressure*

change coefficients. Alternatively, an energy loss analysis may be performed on the structure, with upstream HGL determined from the upstream EL minus velocity head.
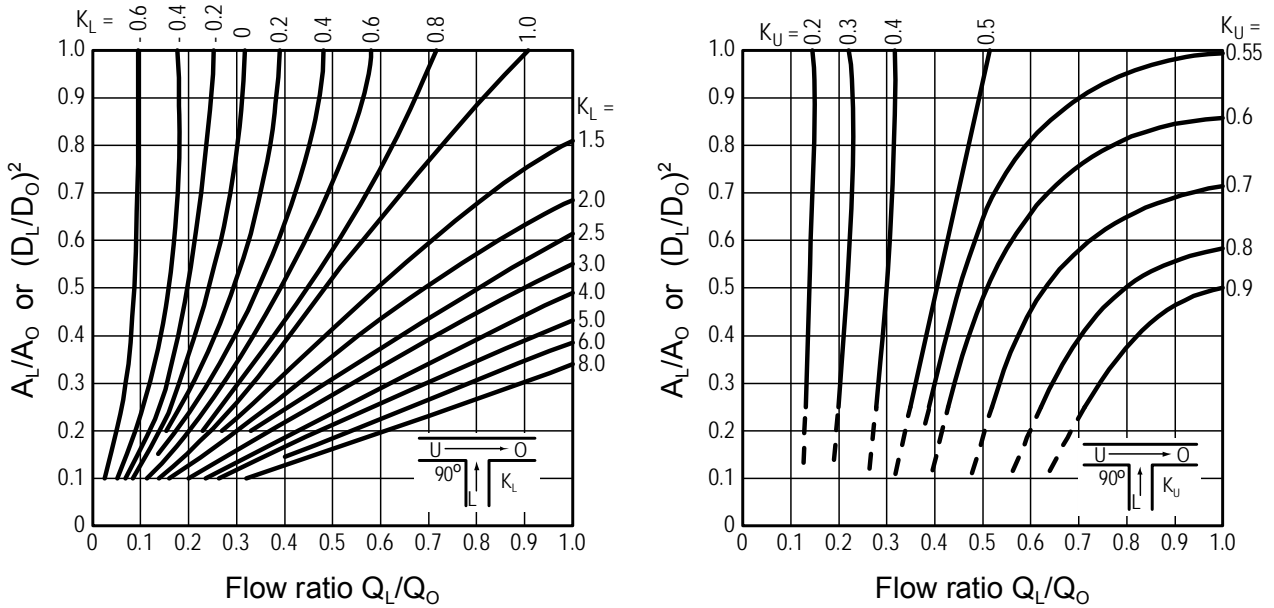
Figures 7.16.16(a) & (b) – Energy loss coefficients at branch lines (Source: Miller, 1990)
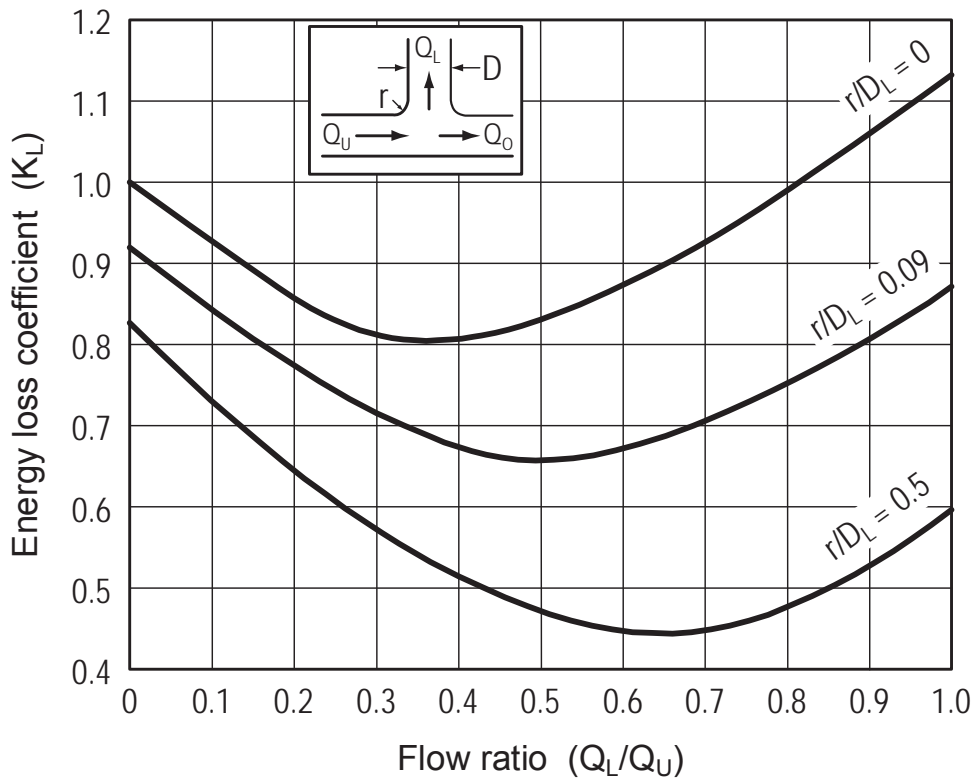
Figure 7.16.17 – Energy loss coefficients at branch lines (Source: Miller, 1990)

The coefficients provided in Figure 7.16.17 are relative to the upstream velocity head ($V_u^2/2g$) **not** the outlet velocity head.

7.16.13 Expansions and contractions (pipes flowing full)

Sudden expansions or contractions in stormwater pipelines should normally be avoided. They may however need to be installed as part of a temporary arrangement in a system being modified or upgraded, or in a relief drainage scheme.

Where the above arrangement is unavoidable, an appropriate allowance for head loss should be made. The pressure change can be derived using the energy loss coefficients determined from Table 7.16.7. The equivalent pressure change coefficients (K_U and K_O) are provided in Table 7.16.8.
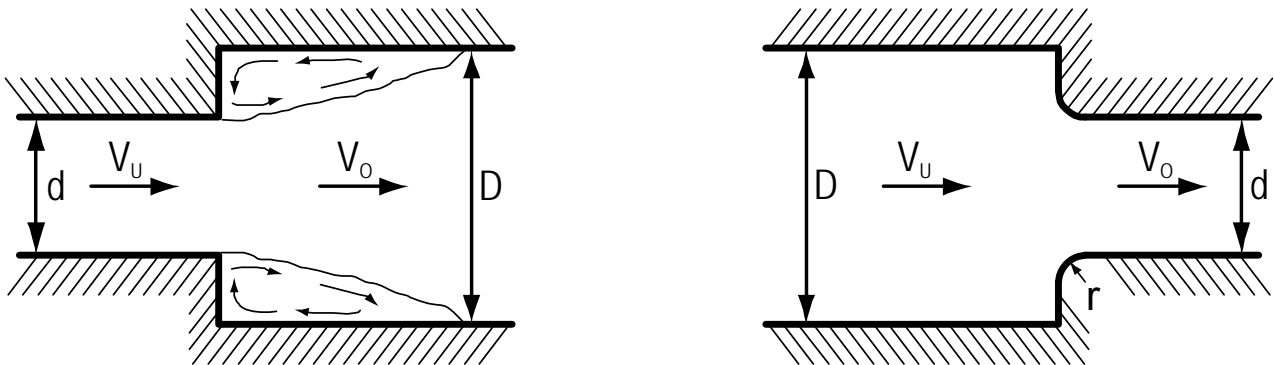
Figures 7.16.18(a) & (b) – Flow conditions for sudden expansion and contraction

Table 7.16.7 – Energy loss coefficients for flow expansions and contractions within pipes [1]

A_U/A_O or A_O/A_U	d/D	Sharp expansion [2]	Contraction [3]				
			Sharp edge	r/d = 0.02	r/d = 0.04	r/d = 0.06	r/d = 0.1
1	1.000	0.000	0.000	0.000	0.000	0.000	0.000
0.8	0.894	0.081	0.079	0.058	0.043	0.036	0.027
0.6	0.775	0.200	0.248	0.165	0.121	0.091	0.060
0.4	0.632	0.377	0.371	0.255	0.187	0.137	0.077
0.2	0.447	0.659	0.442	0.324	0.234	0.169	0.086
0.1	0.316	0.833	0.471	0.353	0.245	0.180	0.087
0	0.000	1.000	0.500	0.376	0.250	0.185	0.087

Notes (Table 7.16.7):

[1]   Sourced from Miller (1990).

[2]   Energy loss coefficient (K_{exit}) relative to upstream velocity head ($V_u^2/2g$).

[3]   Energy loss coefficient (K_{entry}) relative to downstream velocity head ($V_o^2/2g$).

The pressure change coefficient for an expansion or contraction may be determined from the energy loss coefficient using equations 7.21 and 7.22 respectively.

(Expansion) $$K_U = K_{exit} + (A_U/A_O)^2 - 1 \qquad (7.21)$$

(Contraction) $$K_O = K_{entry} - (A_O/A_U)^2 + 1 \qquad (7.22)$$

Table 7.16.8 – Pressure change coefficients for expansions and contractions [1]

A_U/A_O or A_O/A_U	d/D	Sharp expansion [2]	Contraction [3]				
			Sharp edge	r/d = 0.02	r/d = 0.04	r/d = 0.06	r/d = 0.1
1	1.000	0.000	0.000	0.000	0.000	0.000	0.000
0.8	0.894	−0.279	0.439	0.418	0.403	0.396	0.387
0.6	0.775	−0.440	0.888	0.805	0.761	0.731	0.700
0.4	0.632	−0.463	1.211	1.095	1.027	0.977	0.917
0.2	0.447	−0.301	1.402	1.284	1.194	1.129	1.046
0.1	0.316	−0.157	1.461	1.343	1.235	1.170	1.077
0	0.000	0.000	1.500	1.376	1.250	1.185	1.087

Notes (Table 7.16.8):

[1]   Sourced from Miller (1990).

[2]   Pressure change coefficient (K_U) relative to upstream velocity head ($V_U{}^2/2g$).

[3]   Pressure change coefficient (K_O) relative to downstream velocity head ($V_o{}^2/2g$).

7.16.14   Surcharge chambers

(a)   General

The following discussion relates to an energy loss analysis, not a HGL analysis.

Surcharge chambers operate as three-dimensional hydraulic structures. The complicated hydraulic interaction between the various structural components makes it inappropriate to simply add the head loss for each component. The following is presented as a **guide** to the determination of energy loss (head loss) through surcharge chambers.

The results obtained from the following analytical procedures may not be appropriate in all circumstances. Designers should use professional judgement with regard to the appropriate application of these procedures. Specifically, designers should review the final results and assess its reasonability.

Surcharge chamber with or without an outlet pipe (Figure 7.16.19):

Energy loss components:

The sum of:

(i)   modified 90° mitre bend loss

(see (b) below), plus

(ii)   expansion loss component

= $[(V_u{}^2/2g) - (V_L{}^2/2g)]$

(iii) plus screen loss (see (c) below)

(iv)   plus exit loss component = $(V_L{}^2/2g)$

(v)   plus friction loss in chamber (typically only significant for $L > 10D_L$)
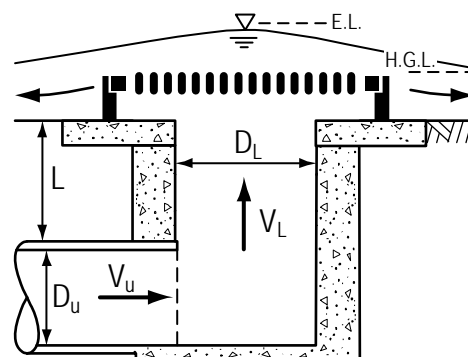
Figure 7.16.19

The HGL at any location should be taken as the energy level at that location minus the local velocity head ($V^2/2g$).

Surcharge chamber with multiple inflow pipes (with or without low-flow outlet pipe – Figure 7.16.20):

HGL analysis:

Step 1: Determine the water surface elevation and flow velocity (V_s) just downstream of the surcharge chamber.

Step 2: Calculate the energy level above the screen:
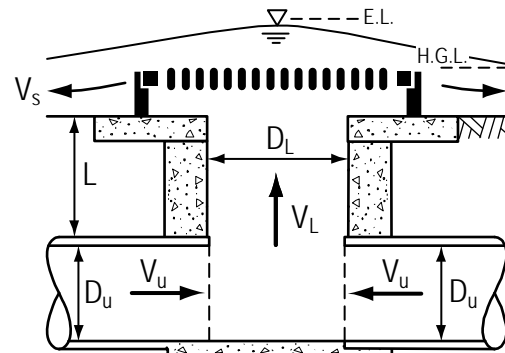
Figure 7.16.20

EL_{outlet} = downstream water elevation + ($V_s^2/2g$) + ($V_L^2/2g$)

Note: The downstream water elevation (HGL) must be determined at the same location as the water velocity (V_s).

Step 3: Calculate head loss (ΔH_{screen}) through screen (see (c) below)

Step 4: Calculate V_u (the actual inflow velocity for the inflow pipe being analysed)

Step 5: Calculate friction loss ($\Delta H_{friction}$) within the surcharge chamber (usually only significant if $L > 10D_L$)

Step 6: Calculate head loss (ΔH_{inflow}) for flow entering the surcharge chamber as the sum of:

(i)   modified 90° mitre bend loss (see (b) below), plus

(ii)   expansion loss component = [$(V_u^2/2g) - (V_L^2/2g)$]

Step 7: Calculate energy level (EL) inside the relevant inflow pipe:

$EL_{pipe} = EL_{outlet} + \Delta H_{screen} + \Delta H_{friction} + \Delta H_{inflow}$

Step 8: Calculate HGL in relevant inflow pipe = $EL_{pipe} - (V_u^2/2g)$

Surcharge chamber with outlet pipe of equivalent size (Figure 7.16.21):

Energy loss components:

(i)   T-junction loss K_L

(see Figure 7.16.17)

(ii)   plus screen loss (see (c) below)

(iii) plus exit loss component

       = ($V_L^2/2g$)
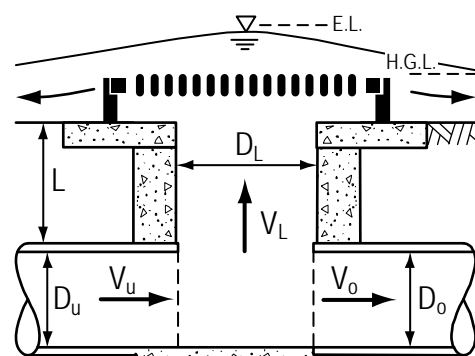
Figure 7.16.21

(iv) plus friction loss in chamber (typically only significant for $L > 10D_L$)

Otherwise, if the chamber design is outside the range of Figure 7.16.17 then determine the losses as per the recommended analysis for Figure 7.16.19.

Surcharge chamber with smaller low-flow outlet pipe (Figure 7.16.22):

Determination of the pressure change coefficient (K_U) for low-flow outlet pipe (D_o):

Step 1: Determine equivalent inflow pipe diameter (D_u^*) that carries only the low-flow pipe discharge (Q_o):

$$D_u^* = D_u(Q_o/Q_u)^{0.5}$$
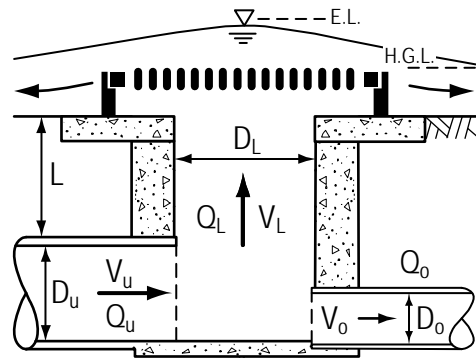
Figure 7.16.22

Step 2: Calculate (K_U) based on normal pit charts for (D_u^*/D_o) in Appendix 1 (i.e. by ignoring that portion of flow (Q_L) discharging from the chamber, thus $Q_g = 0$)

(b)   90 Degree mitre bend losses

The energy loss coefficient (K_b) presented in equation 7.24 for a 90-degree mitre bend was originally developed for a conduit of constant diameter (i.e. $D_u = D_L$). For the purpose of analysing energy losses within surcharge chambers a 'modified' energy loss equation has been presented (equation 7.23) which adopts the same coefficient (K_b) but allows for cases where the chamber velocity (V_L) may be less than the upstream velocity (V_u).

Equation 7.23 can **only** be used in association with an energy loss correction for flow expansion (as presented in the above design procedures), and only when the chamber velocity is equal to, or less than, the upstream velocity.

A coefficient multiplier (C_o) is applied to the energy loss coefficient to account for a short chamber length (L). This correction makes allowance for additional energy losses caused by poor flow distribution within the surcharge chamber immediately after a sharp bend.

$$\Delta H = K_b (V_L^2/2g) \qquad (7.23)$$

$$K_b = 1.2 (C_o) \qquad (7.24)$$

where:
   K_b = head loss coefficient
   C_o = correction for short outlet pipe length (refer to Table 7.16.9)

Table 7.16.9 – Mitre bend outlet length correction factor [1]

L/D_U [2]	C_O
0	2.8
0.5	2.0
1.0	1.5
> 1.7	1.0

Notes (Table 7.16.9):

[1]   Sourced from Miller (1990).

[2]   If more than one pipe enters the chamber, then let Du equal the average pipe diameter.

(c)   Screen losses

Head loss through a clean or partially blocked screen may be assessed based on equation 7.25.

$$\Delta H = K_t^* \, (V_n^2/2g) \qquad\qquad (7.25)$$

where:

$$K_t^* = 2.45 \, A_r - A_r^2 \qquad\qquad (7.26)$$

and:

ΔH =   Head (energy) loss [m]
K_t^* =   head loss coefficient based on velocity through screen
A_r =   Area ratio = $A_b/A = 1 - A_n/A$
A_b =   Blockage surface area of the screen bars (including debris blockage where applicable and that part of the screen not directly impacted by the outlet jet) (m^2)
A_n =   Net flow area through screen that is in direct alignment with the outflow jet (i.e. excluding bars, debris and any non-effective flow area of the screen)
A =   Gross flow area at the screen, $A = A_b + A_n$ (m^2)
V_n =   flow velocity through the partially blocked screen (m/s)
V_s =   surface flow velocity well downstream of the screen (m/s)
g =   acceleration due to gravity (9.80 m/s^2)

Technical note 7.16.1

Equation 7.25 has been developed from the original recommendations of US Bureau of Reclamation (1987). The coefficients are generally higher than those recommended by researchers such as Miller (1990) but are considered to provide more realistic values for heavily blocked screens. The coefficients provided by equation 7.26 for a 'clean' screen (say A_r < 0.2) are however comparable with those recommended by Miller. A detailed discussion on screen losses is provided in section 12.5.6 of this Manual.

7.16.15   Hydraulic grade line (pipes flowing partially full)

For established flow in a pipe running partially full the HGL will correspond with the water surface.

At the upstream end of a pipe reach at a structure the position of the HGL and water surface will depend upon the depth of flow in the downstream pipe and the head loss occurring at the structure.

The following procedure is commonly used to determine the HGL and water surface at the structure.
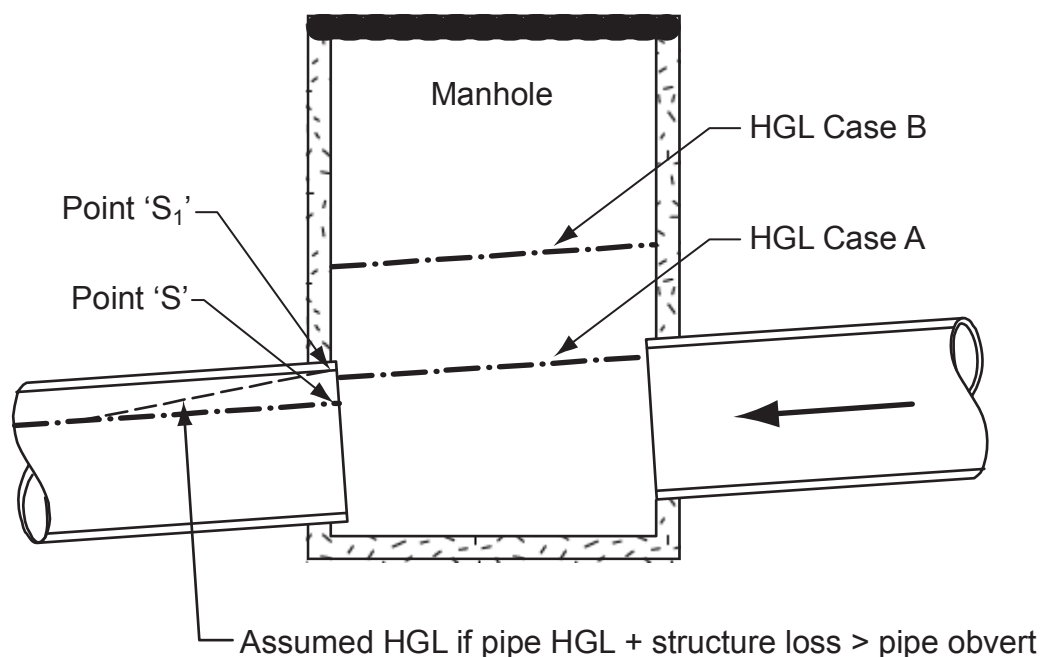
Figure 7.16.23 – HGL determination for pipes flowing partially full

Configuration 1: Straight through line

(a)   Determine HGL at the entry to the outlet pipe (location 'S') for pipe running partially full.

(b)   Add structure loss ($K_u \cdot V_o^2/2g$) where V_o is the velocity in the downstream pipe running partially full and $K_U = 0.5$.

(c)   **Case A:**   If the calculated HGL at the structure is less than the obvert level of the outlet pipe (location S_1) adopt the calculated HGL as the HGL.

       Case B: If the calculated HGL at the structure is greater than the obvert level of the outlet pipe (location S_1) then assume that the downstream pipe is running full at the outlet from the structure. A revised HGL at the structure should then be determined using the appropriate head loss chart based upon the velocity in the downstream pipe running full, and with the structure loss added to the level of the obvert of the outlet pipe (location S_1)

Other configurations:

A similar procedure should be used for the determination of the HGL except that in assessing the trial HGL in Step (b) the following values (Table 7.16.10) of K_U are recommended.

Table 7.16.10 – Trial values of K_U for use in determining HGL under partially full flow conditions

Configuration	K_u
Straight through line	0.5
Change of direction: 0^o to 45^o	0.75
Change of direction: 46^o to 90^o	1.0
Multiple pipe	1.0

Determination of water surface in the structure:

It is recommended that the water surface in the structure be determined using the above procedure for establishing HGL in the downstream pipe, then proceeding as follows:

- Where the calculated HGL at the structure is below obvert of outlet pipe, adopt WSE = HGL.

- Where the calculated HGL at the structure is above the obvert, adopt the obvert (point S_1) as the starting point and add the value of $K_W.V_o^2/2g$ determined from the appropriate design chart, based upon the velocity in the downstream pipe running full.

7.16.16 Plotting of HGL on longitudinal section

It is recommended that the HGL be plotted on the longitudinal section. The plotted HGL should normally be for the AEP for which the pipe system is designed.

Where different design AEPs have been adopted for separate parts of the system, the HGL appropriate to that part of the system should be plotted. This may occur in a system where, for example, the upper reaches are designed for 39% AEP (1 in 2 years) and the lower reaches are designed for 1% AEP because of the occurrence of a trapped sag or the like.

7.16.17 Equivalent pipe determination

Where multiple pipes or combinations of pipes and box culverts occur at a drainage structure the following procedure may be used for the determination of head losses:

$$D_e = \left[\frac{4}{\pi}\right]^{0.5} . \frac{\Sigma Q_n}{\left[\Sigma (Q_n . V_n)\right]^{0.5}} \tag{7.27}$$

$$V_e = \frac{\Sigma (Q_n . V_n)}{\Sigma Q_n} \tag{7.28}$$

Where pipes only are involved, equations 7.27 and 7.28 may be expressed as follows:

$$D_e = \frac{\Sigma Q_n}{\left[\Sigma (Q_n^2 / D_n^2)\right]^{0.5}} \tag{7.29}$$

$$V_e = \frac{4}{\pi} . \frac{\Sigma Q_n}{D_e^2} \quad or \quad \frac{4}{\pi} . \Sigma \left(\frac{Q_n^2}{D_n^2}\right) . \frac{1}{\Sigma Q_n} \tag{7.30}$$

where:
  V_e = Equivalent flow velocity
  D_e = Equivalent pipe diameter
  D_n = Diameter of pipe *'n'*
  Q_n = Flow for pipe *'n'*
  V_n = Flow velocity for pipe *'n'* (based on pipe flowing full)

8. Stormwater outlets

8.1 Introduction

The design of stormwater outlets can attract significant public and council attention; consequently their design usually needs to satisfy a range of diverse and sometimes conflicting requirements. The relative importance of each design requirement will vary from site to site. Designers should consult with the local government on the preferred location, design and layout of stormwater outlets prior to commencement of detailed design.

This chapter commences with a discussion on the selection of tailwater levels for the hydraulic analysis of stormwater pipes commencing at their outlets. Following this is a discussion on the design of tidal and non-tidal outlets, the use of backflow control devices, and the design of outlet energy dissipaters.

8.2 Factors affecting tailwater level

8.2.1 Contributing factors

The starting water level used in the hydraulic analysis of stormwater drainage systems may be influenced by the following factors:

- tidal variations
- storm surge
- wave setup
- climate change
- coincident flooding (refer to section 8.3.4)
- superelevation of channel water surface (refer to section 9.3.6 (c)).
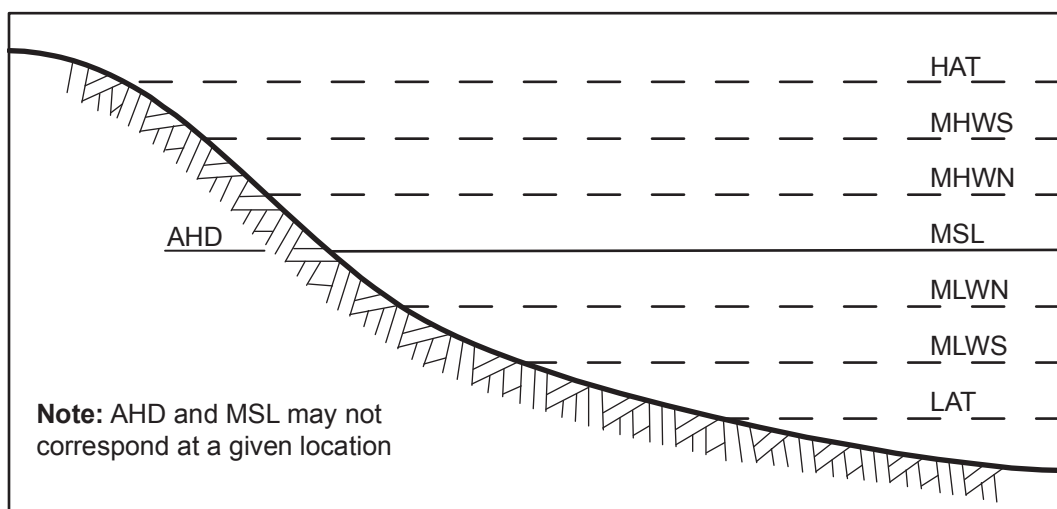

Figure 8.1 – Tidal variations

8.2.2 Tidal variation

Annual tide tables published by the Queensland Government predict tide levels throughout the year and define the average levels of the tidal planes at standard ports and secondary places along the Queensland coast.

Care must be taken when referencing these tide tables to correctly translate the quoted levels from their local Low Water Datum to the survey datum used for the drainage design (normally AHD).

It should be noted that tide tables do **not** predict *actual* sea levels. Actual sea levels are the result of a combination of the above factors. Therefore, HAT (Highest Astronomical Tide) does **not** represent the likely highest possible sea level (refer to the Glossary for the definition of terms in Figure 8.1).

8.2.3   Storm surge

A storm surge (or 'meteorological tide') is an atmospherically driven ocean response caused by extreme surface winds and low surface pressure associated with severe weather conditions, usually cyclones. Strong offshore winds can generate significant ocean currents. When these currents approach a barrier such as a shoreline, sea levels increase ('wind setup') as the water is forced up against the land. The low atmospheric pressures associated with cyclones can also raise sea levels well above predicted tide levels.

Storm induced wave action can produce both a 'wave setup' (a rise in mean sea level as waves approach a shoreline) and 'wave run-up'. Wave run-up is generally not considered in the selection of tailwater level; however, both the actions of wave run-up and wave splash (carried by onshore winds) can significantly influence local flooding.

When a storm surge and wave setup are combined with the normal astronomical tide the resulting mean water level (MWL) reached is called the 'storm tide level'.

Designers should note the following issues:

- Predicted storm surge elevations along the Queensland coastline can vary significantly.
- A storm surge is more likely to be associated with a long duration storm event such as a cyclone.
- The existence of a storm surge is highly probable during peak flooding of large creeks and small rivers. However, it is likely the effects of storm surge would have passed before the flood peak is reached in a large river system (e.g. a river with a time of concentration of days, not hours).
- A storm surge will likely be coincident with the peak outflow from occasional minor and major storm events on minor drainage systems and small creeks.

It is recommended that designers confer with the local government in order to determine an appropriate tailwater level for piped and open channel outfalls to tidal waterways. The Queensland Government provides information on predicting surge levels along the Queensland coast.

8.2.4   Wave setup

Wave setup is defined as the superelevation of water levels due to the onshore movement of water by wave action alone. Wave setup is the change in mean water level due to wave action. It is **not** the actual wave height. It may occur during, or in the absence of, a storm event.

Wave setup is likely to occur during severe storms and should be incorporated into the storm surge prediction for coastal waters.

Wave setup can also occur on large water bodies such as lakes. Consideration should be given to the likely water level increase caused by wave setup when nominating the starting water level in large lakes; however, this is only likely to be in the order of a few centimetres.

Guidelines for the determination of wave setup may be obtained from U.S. Army Corps of Engineers (1984).

8.2.5 Climate change

Designers should consider the impact of climate change. Predictions of the possible effect on sea level and other effects are given in the International Panel on Climate Change 4th Assessment Report, IPCC 2007; CSIRO & Australian Greenhouse Office 2006; CSIRO & Bureau of Meteorology 2007; and Engineers Australia 2011.

Sea level rise considerations need to take account of both a rise in mean sea level, as well as a potential rise on storm surge. The NSW Department of Environment Climate Change and Water 2010 reports potential sea level rise, relative to 1990, of 0.4 m by 2050 and 0.9 m by 2100. CSIRO & Bureau of Meteorology 2007 reports a potential increase in storm surge of 0.3 m for the 1 in 100 year event for the Cairns region in addition to any mean sea level rise.

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

8.3 Selection of tailwater level

8.3.1 Tailwater levels for tidal outfalls (oceans and bays)

Designers should confer with the relevant local government to establish an appropriate tailwater level for the design of stormwater outfalls discharging to oceans or bays. Consideration should be given to the joint probability of occurrence of the design storm, tide level and storm surge together with an allowance for climate change.

Whilst it is not possible here to provide specific recommendations, some suggested levels are provided in Table 8.3.1. These suggestions should in no way replace the need to confer with the local government and for the application of sound engineering judgement.

Table 8.3.1 – Suggested tailwater levels for discharge to tidal waterways

Design condition	Design tailwater level [1]
Minor storm [2]	In the range: MHWN to MHWS
Major storm [2]	In the range: MHWS to HAT
Climate change	Additional 0.3 m (minimum) [3]

Notes (Table 8.3.1):

[1] The start HGL adopted for design should be determined in accordance with the rules detailed under *Outfall generally* in section 7.16.5(a) of this Manual.

[2] Designers should also examine the effect of increased tailwater level resulting from climate change.

[3] For new developments, the local government may determine appropriate minimum floor and/or site filling levels taking into account the predicted impact of climate change.

It is noted that the design capacity of the underground drainage system will be reduced when the water level exceeds the design tailwater for the minor storm. This reduction in the capacity of the underground system needs to be taken into account when determining the flow capacity for the drainage system.

8.3.2 Tailwater levels for tidal outfalls (rivers and creeks)

Designers should confer with the relevant local government to establish an appropriate tailwater level for the design of stormwater outfalls to tidal waterways. Consideration should be given to the joint probability of occurrence of the design storm, tide level (at the outfall), storm surge and coincident flooding together with allowance for the potential effects of climate change.

The potential impact of coincident flooding (flood surcharge) on design tailwater levels is discussed in section 8.3.4.

8.3.3 Tailwater levels for non-tidal outfalls

The design of a drainage systems which discharges to a non-tidal outfall, e.g. a lake, open channel, creek or river, needs to take into account the expected tailwater level in the receiving waters.

In cases where the tailwater level is not affected by stormwater runoff from an external catchment, e.g. in a detention basin or an open channel receiving water from only the subject drainage system, the tailwater level should be determined in accordance with (a) and (b) below.

In cases where the tailwater level is affected by stormwater runoff from an external catchment, the critical design situation for surcharging of the drainage system may occur when the flow rate in the drainage system is less than the design flow rate. In such cases, the critical tailwater level and the drainage discharge should be determined by an investigation of the joint probabilities of flooding in both the subject drainage system and its receiving waters. Suggested procedures for assessing coincident flooding are provided in section 8.3.4.

In situations where the catchment area of the receiving waters is relatively large in comparison with the catchment area of the drainage system, it may be appropriate to treat the two waterways as independent drainage systems.

(a) Outlet to lakes and dams

Design tailwater levels for outfalls discharging into large lakes may need to consider the effects of 'wave setup' as discussed in section 8.2.4 as well as potential seasonal variation in water level.

As a design storm event is likely to occur following a period of consistent rainfall, it is reasonable to assume that the lake or dam will be at or approaching full capacity at the time the design storm occurs. The starting HGL for the design storm should therefore be set at the overflow level of the lake or dam (e.g. crest of the emergency spillway) or at a level above the overflow level consistent with the calculated total inflow to the storage.

Note that under certain circumstances, the starting HGL may be lower than that discussed above. For example, where the AEP of design storm for the side catchment is low (e.g. 1 in 2 years) and the lake is large; thus the lake may or may not be full. In such cases the starting HGL should be determined in consultation with the relevant local government.

(b) Outlets to detention/retention basins

It is usual for a detention basin to be designed and checked for a number of storm events. The starting HGL level for the design AEP of the pipe system should be determined by analysing the detention basin for the same AEP as the pipeline being designed. If other pipe systems contribute and have catchment characteristics vastly different to those for the system being designed, then the designer must consider the behaviour of the system as a whole.

8.3.4 Coincident flooding

Water levels within receiving waters may be affected by flood flows passing down the receiving waterway. The severity of this coincident flooding will depend principally on the ratio of the time of concentration of the side channel/drain relative to that of the receiving waterway.

Various procedures which permit the assessment of the most critical combination of flow and tailwater are described below. The appropriate maximum tailwater derived after consideration of each procedure should be adopted.

Consideration should also be given to the rules for determining starting HGL as detailed in section 7.16.5 of this Manual.

The following procedures are based on Carroll (1990).

(a)   Simplified rational method for discharge to smaller creeks

To determine the critical combination of tailwater level and stormwater discharge (Q_s), check both cases (i) and (ii) below and where appropriate, any additional intermediate cases. The tailwater level should be based on the combined channel flow rate ($Q_{combined}$).

Subscripts 's' and 'm' refer to the 'side drain' or stream and 'main stream' respectively as shown in Figure 8.2.
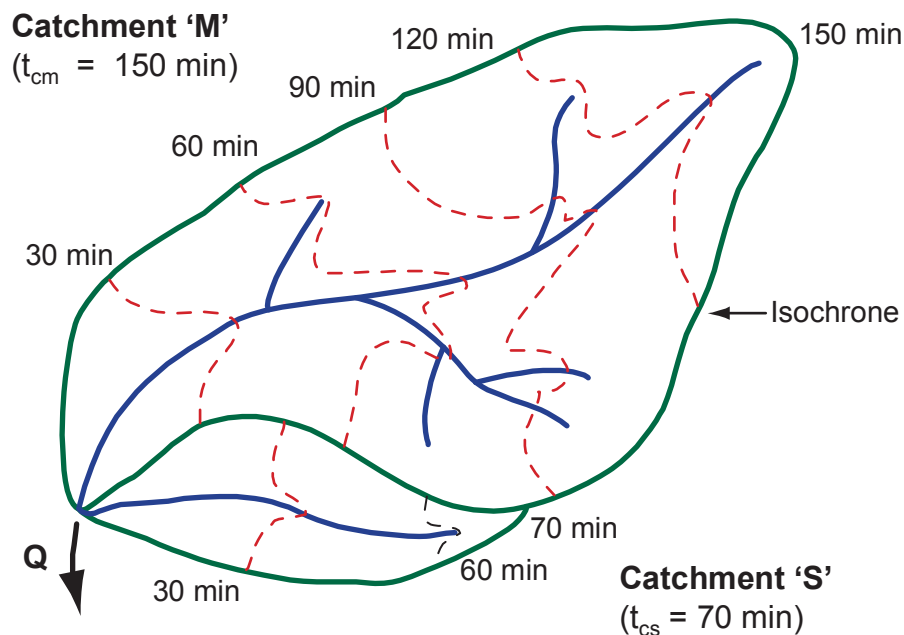
Figure 8.2 – Example catchment showing side drain and main catchment

(i)   Case with rainfall intensity corresponding to time of concentration for side drain.

$$Q_{combined} = Q_s + Q_m \tag{8.1}$$

where:
$$Q_s = C_s \cdot I_s \cdot A_s \tag{8.2}$$

$$Q_m = C_m \cdot I_s \cdot A_m \, (t_{cs}/t_{cm}) \tag{8.3}$$

In this case Q_m is the flow in the main stream occurring when the peak in the side drain Q_s takes place.

(ii)   Case with rainfall intensity corresponding to time of concentration of main stream.

$$Q_{combined} = Q_s + Q_m \qquad (8.4)$$

where:

$$Q_s = C_s \cdot I_m \cdot A_s \qquad (8.5)$$

$$Q_m = C_m \cdot I_m \cdot A_m \qquad (8.6)$$

In this case Q_s is the flow in the side drain occurring when the peak in the main drain Q_m takes place.

Example:

Determine the critical combination of discharge and tailwater for a design AEP of 2% (1 in 50 year) for discharge from the side drain under the following circumstances.

Catchment parameters	A (ha)	C_{50}	t_c (min)	I_{50} (mm/hr)
Side drain	5	0.87	12	200
Main stream	500	0.82	120	90

	Case (i)	Case (ii)	Intermediate
Q_s	2.42	1.09	1.84
Q_m	22.78	102.50	31.74
$Q_{combined}$	25.20	103.59	33.58
Tailwater level	12.0	14.2	12.3

Stream bed level at outfall   R.L. 10.00
Side drain invert level   R.L. 10.50
Side drain obvert level (say)   R.L. 11.70

The intermediate case considered above has been assessed for t_c = 22 min.

(b)   Hydrograph procedure for non-tidal creeks and rivers

Step 1   Using an appropriate runoff/routing model determine the runoff hydrographs for the main and side catchments using the critical design storm duration for the side catchment.

Step 2   From the hydrograph for the main catchment read the discharge in the main stream at the time corresponding to the peak in the side drain. Determine the tailwater level in the main stream for this discharge and undertake backwater analysis in the side drain for this tailwater level and the side drain peak discharge.

Step 3   From the hydrograph for the side catchment read the discharge in the side drain at the time corresponding to the peak in the main stream. Determine the tailwater level in the

main stream at the peak discharge and undertake backwater analysis in the side drain for this tailwater level and corresponding discharge in the side drain.

Step 4   Repeat the above analysis for the critical design storm for the receiving waterway, and for intermediate storm periods if appropriate. If the receiving waterway has a time of concentration significantly larger than the side catchment, then it may be reasonable to consider only the critical design storm duration of the side catchment.

Step 5   Adopt the envelope of the highest calculated backwater profiles in the side drain. The designer should consider an appropriate range of coincident flood levels.

(c)   Quick IFD method

This method is useful in providing a quick result and an indication of the ARI of the corresponding events. It is similar to procedure (a) above and it is not intended to replace more rigorous procedures such as (b) above.

PART 1:

Step 1   Plot the point corresponding to the design ARI and t_{cs} for the side drain on the IFD curves (Figure 8.3) for the location in question.

Step 2   Calculate the peak discharge for the side drain for the design ARI.

Step 3   Calculate the total rainfall depth in mm that has fallen on the side catchment in period t_{cs}.

Step 4   Determine the rainfall intensity applicable to the main catchment by application of the same depth over the period t_{cm}.

Step 5   Calculate the peak discharge for the full area of the main catchment for the intensity determined in Step 4 and determine the applicable tailwater level for the combined peak discharges.

Step 6   Plot the point on the IFD curves for the intensity determined in Step 4.

Step 7   The ARI determined in Step 6 is an indication of the ARI of the discharge in the main stream corresponding to the design ARI for the side drain.

PART 2:

Step 8   Plot the point on the IFD curve (Figure 8.3) corresponding to t_{cm} and I_m for the main stream, for an appropriate main stream ARI.

Step 9   Draw a horizontal line, i.e. constant intensity to intersect the IFD curve at t_{cs} for the side drain. Note the ARI.

Step 10   Calculate peak discharge for both catchments for intensity I_m and determine tailwater level for combined discharges.

Step 11   Carry out backwater analysis for the side drain discharge and tailwater level determined in Step 10.

Step 12 The ARI determined in Step 9 is an indication of the ARI of the discharge in the side drain corresponding to a selected rarer event in the main stream.
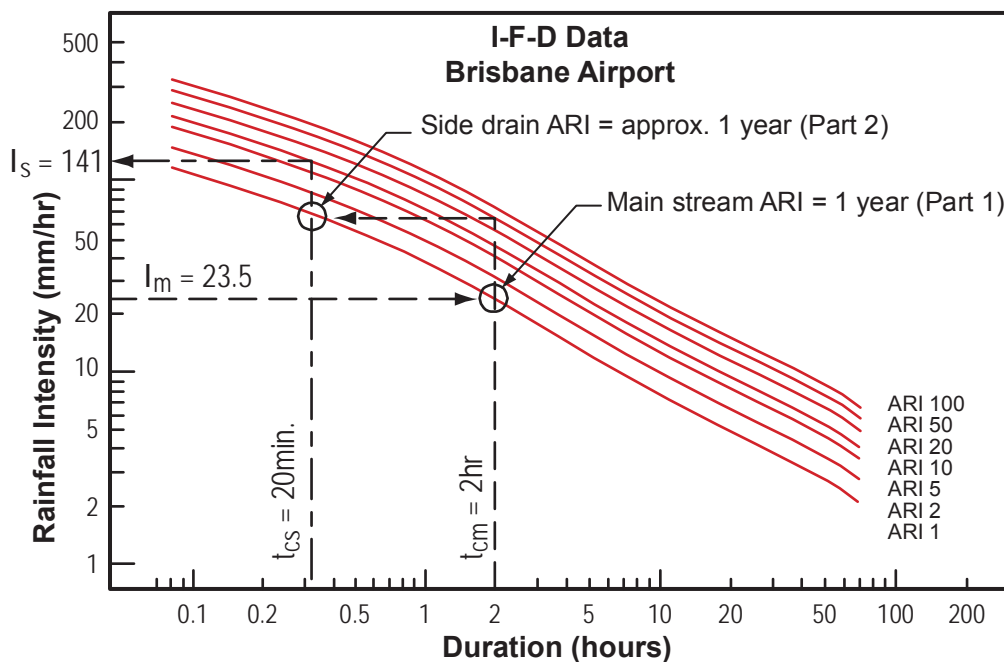
Figure 8.3 – Intensity-frequency-duration plot (Brisbane Airport)

Example:

PART 1

Side drain ARI = 10 years
t_{cs} = 20 minutes
I_s = 141 mm/h
Rainfall depth = 47 mm
t_{cm} = 2 hr
I_m = 23.5 mm/h
Main stream ARI = 1 year

PART 2

Main stream ARI = 50 years
I_m = 62 mm/h
Side drain ARI = approximately 1 year

8.4 Design of tidal outlets

Works constructed in tidal areas may need to comply with the requirements of a number of government agencies. The areas of particular concern include:

- areas below MHWS
- Fish Habitat Reserves
- Tidal Wetland Reserves
- National Parks
- State Marine Parks
- Great Barrier Reef
- Coastal Management Areas
- areas controlled by a Port Authority
- areas controlled by a Waterways Authority

8.4.1 All tidal outlets

Design considerations for all tidal outlets include:

• All relevant design issues presented in section 8.5.

• The recommendations and preferences of the local authorities, including maintenance capabilities of the intended asset manager.

• Use of an appropriate energy dissipater to control undesirable scour. Outlets located above MLWS should incorporate sufficient scour protection to allow discharge during low tide conditions.

• The ecological impact of gross pollutants, in particular plastic bags. Typically these impacts are greatest when discharged directly into tidal waters, therefore appropriate treatment of the stormwater should be considered.

• Use of appropriate concrete specifications as per AS 3600.

8.4.2 Open channel outlets (tidal)

Design considerations for tidal open channel outlets include:

• The risk of channel erosion caused by tidal flow velocities (tidal waters passing in and out of the channel).

• The use of natural marine vegetation to stabilise channel banks.

• Sustainable management of vegetation growth within the channel.

• If heavy reed/mangrove growth is expected within the outlet channel and such vegetation could adversely affect the passage of water through the channel, or upstream discharge of stormwater into the channel, then consideration may be given to the inclusion of an elevated bypass channel as shown in Figure 8.4. The bypass channel should be located above the elevation at which mangroves grow—thus minimising the need for regular maintenance clearing of mangroves from the channel.
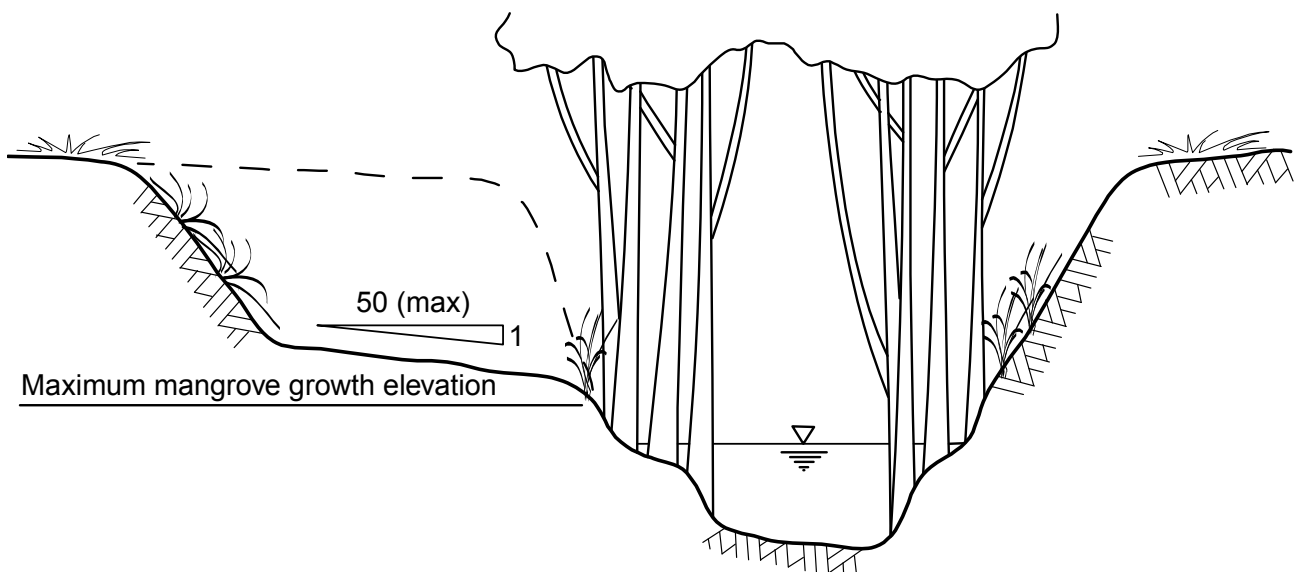

Figure 8.4 – Tidal channel with high level bypass channel

8.4.3   Piped outlets (tidal)

Design considerations for tidal pipe outlets include:

•   Invert level should be above LAT, preferably somewhere between MLWS and MSL

•   The obvert level is normally below HAT

•   Likely impact of sand/sediment blockages of outlet. Elevated outlets may reduce the risk of sand blockage and allow maintenance inspection of the pipe; however, aesthetic considerations may require outlets be located below low tide level.

•   Use of flapgates or similar, where appropriate, to prevent the intrusion of salt water and/or sediment into the pipe. The flapgate may need to be located within the first access chamber set back from the beach to protect its operation from vandalism, wave attack, debris and sand blockage. However, in some cases it may be preferable for the flapgate to be located at the end of the pipe for ease of maintenance inspection (refer to Figure 8.5).

8.4.4   Outlets to tidal estuaries and waterways

Design considerations for piped outlets discharging to tidal estuaries and waterways include:

•   Aesthetics of the outlet as observed by waterway users including boaters and canoeists. Where appropriate, the outlet may need to be recessed into the river/creek bank and/or coloured to minimise its visual impact. It is noted that large tidal outlets can be subjected to unsightly graffiti.

•   The possibility of natural or accelerated bank or bed erosion/accretion—not necessarily resulting from the outlet—and the potential impact of the outlet headwall. This may best be determined by reference to historic maps and photographs (including aerial photographs) of the site.

•   Where practical, stormwater outlets should be located away from highly mobile or erodible stream banks, including the inside or outside of sharp river bends.

•   If an outlet must be located on the inside or outside of a channel bend, then it should be designed to tolerate expected bank erosion. In such cases it may be desirable to recess the outlet into the bank and construct a connecting drainage channel, or to locate the outlet in a position less likely to experience bank scour/deposition.

8.4.5   Outlets to beaches

In addition to the issues raised in sections 8.4.1 and 8.4.3, design consideration for piped outlets that discharge near beach zones include:

•   Observed advantages and disadvantages, including invert levels, of existing outlets located in similar coastal environments.

•   Possible undermining of the structure by wave action and longshore currents.

•   Lateral loads that might be applied by differential sand levels each side of the pipe caused by longshore littoral drift.

•   Potential adverse effects on the adjacent coastline caused by changes in the natural longshore littoral drift caused by sand deposition against the pipeline.

•   The potential for sand deposition, debris and fouling that may impede the function of flapgates (also refer to the final dot point in section 8.4.3).

•   The need for, and the provision of, maintenance access to remove sand and sediment deposition/debris from within the pipe.

Technical note 8.4.1 Beach outlets

It should be noted that the natural erosion and accretion of sand on a beach is a function of the wave action and the porosity of the sand. If a stormwater outlet discharges regular dry weather flows causing long-term saturation of the sand adjacent the outlet, then an un-natural loss of sand (beach erosion) is likely to occur around the stormwater outlet. In such cases, it may be desirable to investigate measures that would reduce these dry weather flows.

Ideally, the outlet should be positioned to minimise sand blockage of the outlet. Advice should be obtained from the local authority and where appropriate, from the Department of Environment and Heritage Protection in regard to the local beach behaviour and littoral processes in each instance.

Figure 8.5 provides an example of a sediment control system which could be located within the last access chamber prior to discharge into coastal waters. The features of this system are:

- sand is prevented from passing up the pipeline beyond the first chamber
- capacity exists for 100% bypass of pipe flows past the gate even if the gate is blocked shut by sand deposition
- the energy released by overtopping (bypass) flows will hopefully mobilise the deposited sand allowing it to be washed out of the pipe
- once the sand is mobilised, the gate should open allowing improved flow capacity and increasing the self-cleaning capabilities of the outlet pipe.
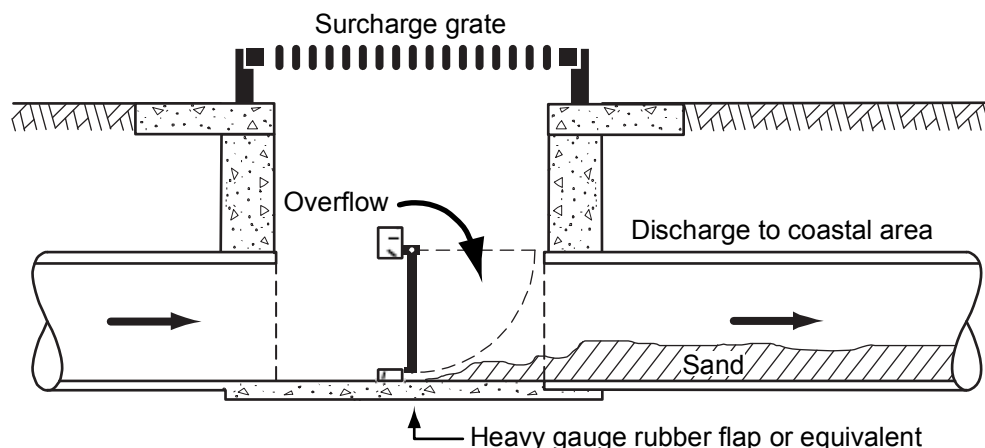
Figure 8.5 – Possible arrangement of sediment backflow control device in coastal zones

8.4.6   Outlets subject to severe wave action

Design considerations for piped outlets subjected to severe wave action include:

- Possible benefits of extending the outlet pipe at a low level through the beach zone to discharge beyond the breaker line and below the low-tide level (can be problematic).
- Any environmental and coastal stability issues.
- Structural design of the outlet to withstand wave impact loadings. Procedures for assessing wave impact loadings are described in U.S. Army Corps of Engineers (1984).

8.4.7   Outlets discharging through acid sulfate soils

Guidelines for the design of drainage systems located within potential acid sulfate soils are presented in section 9.7.9 – *Design and construction through acid sulfate soils*.

8.5 Design of non-tidal outlets

8.5.1 General

The design of all stormwater outlets should consider the following issues. Consideration of these issues will in general be subject to the requirements of the local authority.

(a) Integration into the local character

- Appropriate integration of the outlet into the aesthetics and functions of the immediate area.

- Stormwater outlets within or adjacent to public areas should not interfere with the intended functions and management of the local area.

- Stormwater outlets may or may not incorporate a headwall, depending on local conditions.

- Outlet headwalls may be formed from materials such as precast concrete, decorated in-situ concrete, stacked rock, grouted rock, gabions, or integrated into non-related structural features such as observation decks or retaining walls.

(b) Safety aspects

- Barricades should be installed where applicable. If the drop height exceeds 1 to 1.5 m (refer to local authority) fencing is recommended and should be designed to sustain the imposed actions specified in AS1170.1. In any case, safety aspects shall comply with the requirements of the local authority.

- To the maximum degree allowable within the relevant codes, the choice of materials used in the construction of safety barriers (e.g. tubular metal, treated timber logs, vegetative barriers) should integrate well with the character of the area.

- Wherever practical, the use of outlet screens should be avoided.

- Outlet screens should not be used in circumstances where a person could either enter, or be swept into, the upstream pipe network. In this context, the term 'outlet' refers to stormwater discharge points, not to outflow systems in water storage structures such as detention/retention basins.

- Maximum 150 mm clear bar spacing for outlet screens. Bar screens should also be set a maximum 150 mm above the pipe/channel invert.

- Appropriate access must be provided to the screen for dry weather maintenance including the removal of debris.

- Outlet screens should have a removable feature for maintenance access.

- Outlet screens on pipe units up to 1800 mm in width should be designed such that the full width of the outfall pipe/box can be accessed for periodic maintenance.

- All screens should be secured with tamper-proof bolts or locking device.

- Outlet screens should be structurally designed to break away under the conditions of 50% blockage during the pipe's design storm event.

- Consideration should be given to the hydraulic consequences, including upstream flooding, resulting from debris blockage of outlet screens.

- Also refer to the recommendations of Chapter 12 – *Safety aspects*.

(c)   Location of outlets

- Where practical, stormwater outlets should be recessed into the banks of any watercourse that is likely to experience bank erosion, channel expansion, or channel migration. Typically the minimum desirable setback is the greater of:
    o   3 times the bank height from the toe of the bank
    o   10 times the equivalent pipe diameter (single cell) or 13 times the equivalent diameter of the largest cell (multiple outlets) measured from where the outlet jet would strike an erodible bank (Figure 8.6, also refer to (d) below).
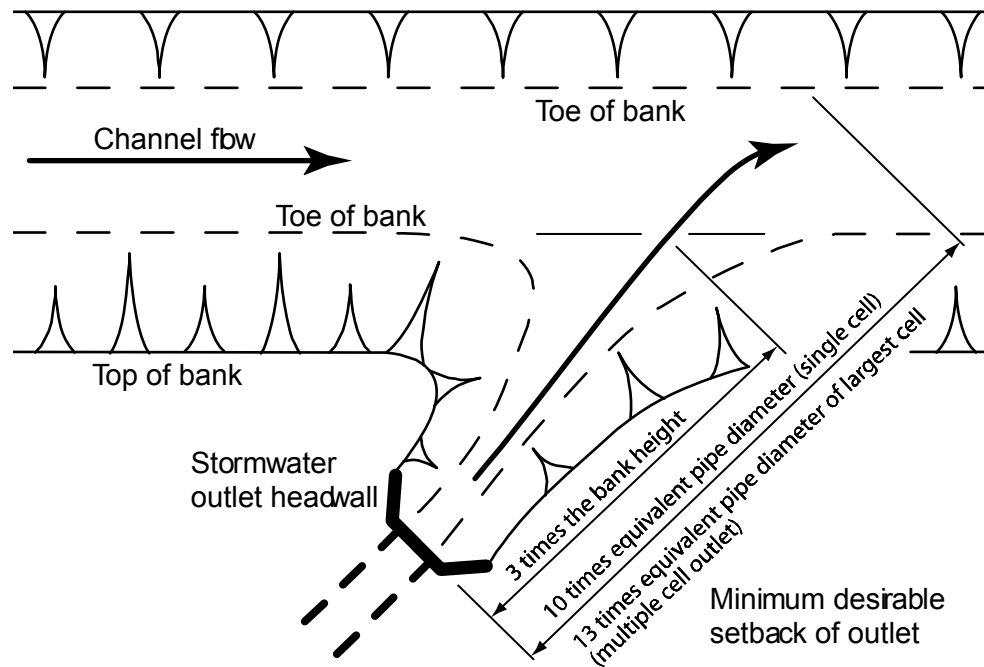

Figure 8.6 – Minimum desirable outlet setback

- Prior to recessing an outlet into a waterway bank, consideration should be given to the long-term impact on the riparian zone.

- Where it is not practical to recess the outlet into the bank, and outlet jetting from the pipe is likely to cause erosion on the opposite bank, then consideration should be given to measures that would reduce the outlet velocity.

- Where practical, stormwater outlets should be located away from highly mobile or erodible stream banks, or the outside of channel bends where turbulence generated by the outlet structure could initiate or aggravate bank erosion.

(d)   Direction of outlets

- Outlets that discharge into a 'narrow' receiving channel should be angled 45 to 60 degrees to the main channel flow. A receiving channel is considered 'narrow' if:
    o   the channel width at the bed is less than 5 times the equivalent pipe diameter, or
    o   the distance from the outlet to the opposite bank (along the direction of the outlet jet) is less than 10 times the equivalent pipe diameter, and
    o   the inflow is more than 10% of the receiving channel flow.

- Stormwater outlets that discharge in an upstream direction need to be avoided wherever practical.

(e)   Elevation of outlets

- Guidelines on desirable invert elevation for outlets discharging to grass swales, channels and lakes are provided in sections 8.5.2 to 8.5.6.

- If the outlet discharges into a permanent sedimentation basin or other stormwater treatment system, then the outlet should discharge above the designated sediment clean-out level.

- Submerged outlets should be avoided for reasons of maintenance, including inspections and de-silting operations.

(f)   Sedimentation and pollution control

- To the maximum degree practical, the outlet should **not** provide suitable habitat for the breeding of biting or nuisance insects. This may be achieved through appropriate design of the outlet, and/or by controlling sedimentation within and immediately adjacent to the outlet.

- To minimise sedimentation within the pipe, a minimum 63% AEP (1 in 1 year) flow velocity of 1.2 m/s is desirable.

- If significant sedimentation problems are expected at, or within the outlet, then the local government shall be consulted in regards to their preference for an open channel or piped outlet.

(g)   Maintenance requirements

- Consideration to be given to the requirements for safe inspections and maintenance access.

(h)   Erosion control

- To the maximum degree practical, stormwater discharge from the outlet shall not cause bed or bank erosion within the receiving waterway/channel.

- If outlet flow velocities are to be reduced by lowering the gradient of the final length of pipe immediately upstream of the outlet, then this length of pipe should be at least 15 times the hydraulic depth (partial full flow).

- Nominal scour protection should be included for a minimum distance of three pipe diameters from the face of the outlet if exit velocities do not exceed 2 m/s.

- If exit velocities exceed 2 m/s, then a site-specific outlet scour control/energy dissipater will be required (refer to section 8.7).

8.5.2   Discharge to grass swales

Reference is made here to the design of outlets that discharge to drainage swales, grass channels, or spoon drains as shown in Figure 8.7. It is noted that the swale, grass channel, or spoon drain may not necessarily discharge directly into a watercourse as depicted in Figure 8.7.
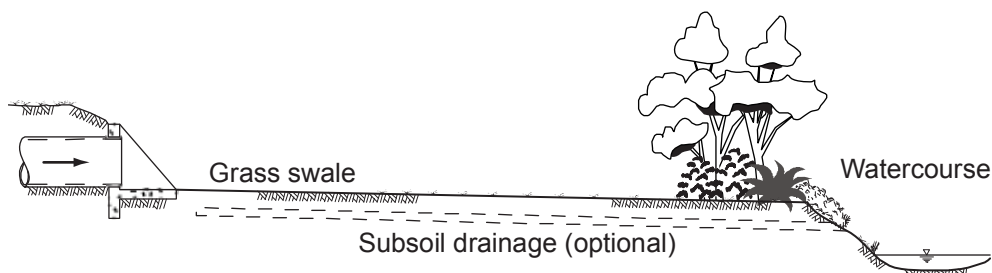

Figure 8.7 – Discharge to swale or spoon drain

Design considerations include:

- The outlet's invert level should be at least 50 mm above the design invert of the grass swale to allow for normal grass growth.

- The depth*velocity product (d.V) within the swale should not exceed 0.4 to 0.6 (depending on safety risk) during a 2% AEP discharge.

- The hydraulic analysis must consider the total flow within the swale, including flows that enter the swale as overland flow.

- Subsoil drainage—incorporating suitable pervious bed materials—may be required to minimise long-term soil saturation along the swale invert to facilitate regular maintenance activities (e.g. grass cutting).

- Final discharge from the swale into a waterway or open channel must incorporate adequate scour protection. Scour protection may include a loose rock chute or stepped spillway. In general, this scour protection should extend at least five times the nominal flow depth upstream of the chute crest to protect the crest of the channel bank from erosion (refer to Figure 8.8).
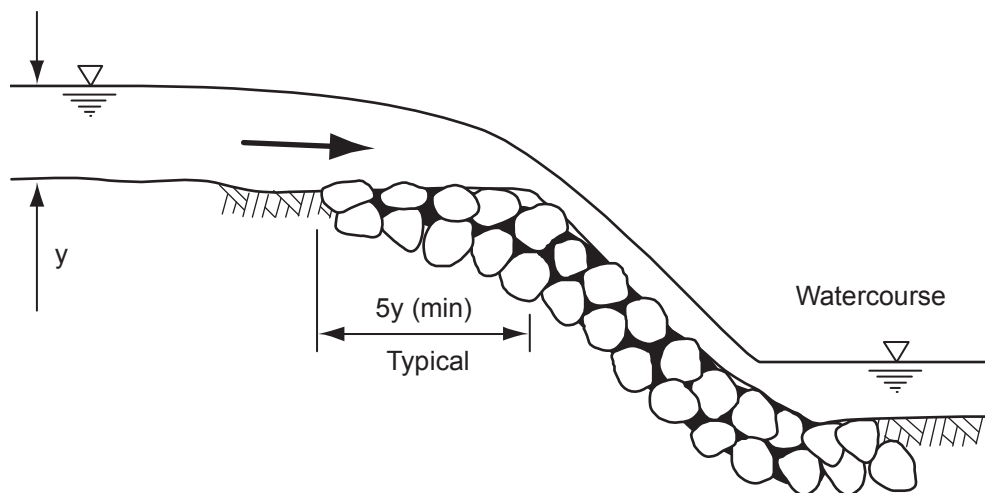
Figure 8.8 – Recommended scour protection at crest of drop chutes

8.5.3 Partial discharge via a surcharge chamber

In the past, surcharge chambers were commonly used when stormwater systems discharged through a park or open space (Figure 8.9). In such cases a lower drainage standard is often accepted through the park, thus allowing a reduction is pipe and forcing the need for a surcharge chamber.
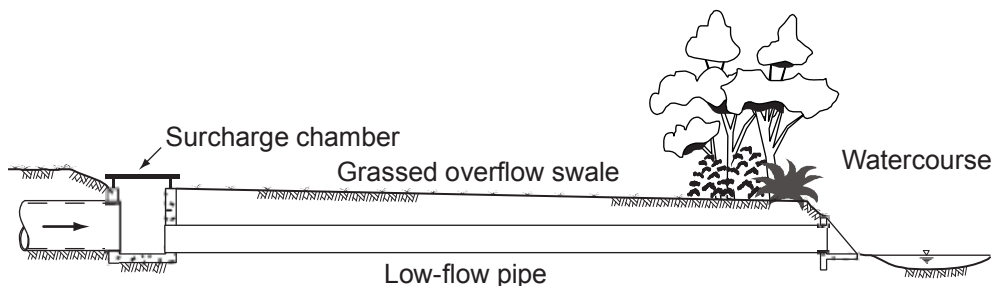
Figure 8.9 – Partial discharge through a surcharge chamber

Prior to incorporating a surcharge chamber into a drainage design, the following should be considered:

- The potential for a person—that has been swept into the upstream drainage system—being trapped inside the surcharge chamber and unable to exit through the chamber or the outlet pipe.

- Potential upstream flooding problems caused by debris blockage of the outlet screen.

- Structural integrity of the outlet screen and concrete coping, and its ability to withstand high outflow velocities and high bursting pressures caused by debris blockages.

- Safe maintenance access to allow removal of debris trapped within the surcharge chamber.

Design of the low-flow outlet pipe discharging from a surcharge chamber should consider the following:

- Minimum desirable 63% AEP (1 in 1 year) flow capacity (refer to local government for preferred minimum standard).

- Pipe capacity should be sufficient to avoid an undesirable depth*velocity product (d.V) within the above overland flow path.

- Minimum desirable 63% AEP (1 in 1 year) peak flow velocity of 1.0 and 1.2 m/s for full pipe flow and partial flow conditions respectively.

- Pipes diameters of 600 mm or smaller are generally less likely to attract exploration by children.

- Maintenance de-silting is generally considered most difficult for 600 to 900 mm diameter pipes (this issue depends on the type of equipment available to the asset manager).

8.5.4   Discharge to constructed outlet channels

In this section, reference is made to the design of stormwater systems that discharge through a constructed drainage channel that connects the outlet to a larger channel or waterway. Such conditions may exist when an outlet structure is recessed well into the bank of a waterway as shown in Figure 8.10.
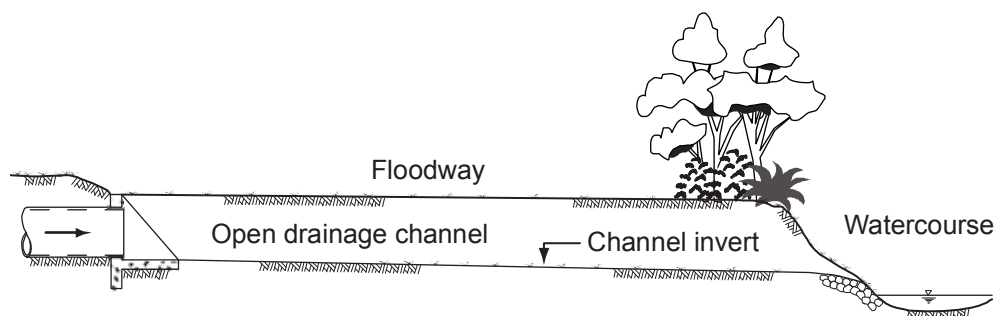
Figure 8.10 – Discharge into constructed outlet channel

Design of constructed outlet channels should consider the following:

- Drainage channels constructed through parks must consider safety issues associated with users of the park, including clear visibility of the open drain by people driving/riding through the park.

- Suitable access should be provided, either across or around the drain, for maintenance vehicles, including grass-cutting equipment.

- Wherever practical, the principles of Natural Channel Design should be incorporated into the design of the outlet channel (refer to Chapter 9 – *Open channel hydraulics*).

- Any potential hydraulic impact on flood waters passing down the floodplain transverse to the outlet channel. This is usually only a concern if a vegetative barrier (riparian zone) is established along the banks of the constructed outlet channel.

- The hydraulic capacity of the outlet channel should be sufficient to convey the expected pipe discharge under normal maintenance conditions.

- The design roughness of the outlet channel must be consistent with the expected long-term vegetative conditions of the channel.

- If heavy reed growth is expected within the low-flow channel, and such reed growth could adversely affect discharge from the stormwater pipe, then consideration should be given to the inclusion of an elevated bypass channel as shown in Figure 8.11. Typically reed growth is not expected to be a major problem if the discharging pipe has a diameter of 1200 mm or greater. A similar channel design can be developed for tidal channels that experience heavy mangrove growth as shown in Figure 8.4.

Bypass channels maintained free of reeds and woody vegetation
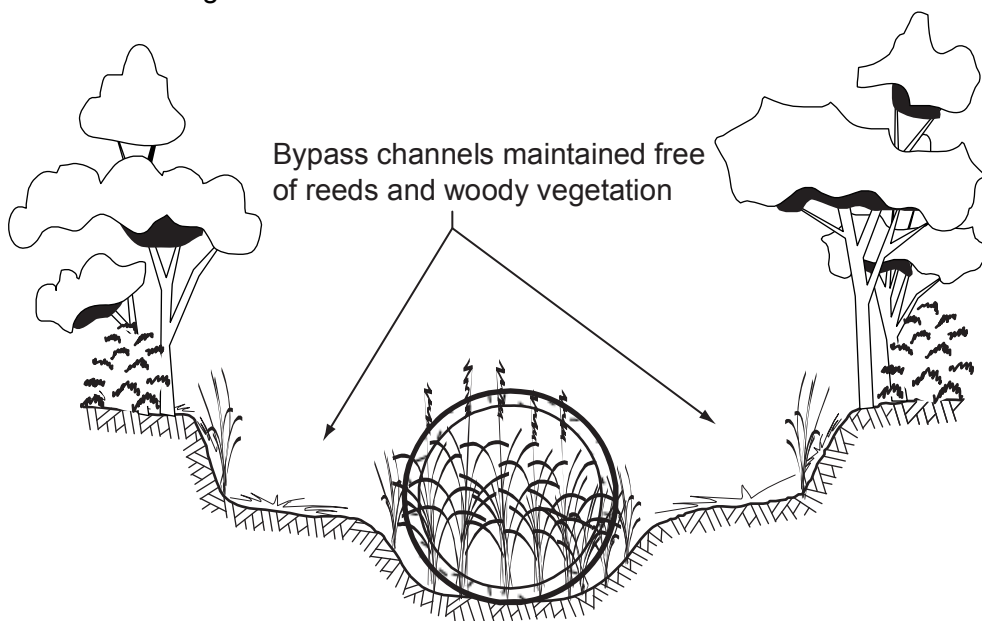
Figure 8.11 – Outlet channel with benching to allow flow bypassing of a heavily vegetated low-flow channel

8.5.5 Discharge to waterways

Reference is made here to the design of outlets that discharge directly into a watercourse as shown in Figure 8.12.

Floodway

Watercourse

Pipe

D

Fall to bed
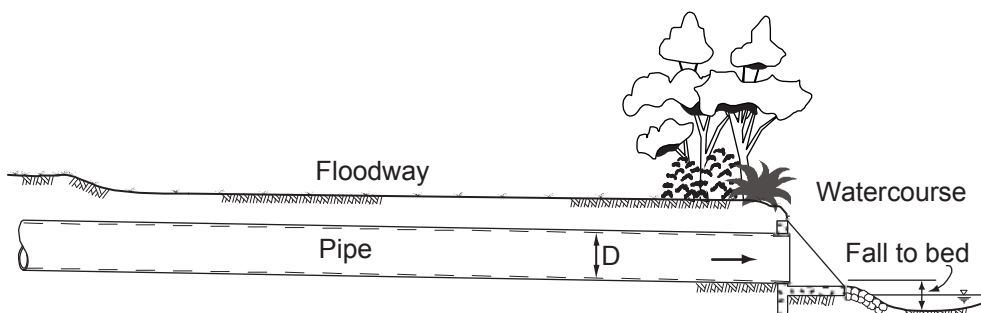
Figure 8.12 – Discharge directly into a watercourse

The design of stormwater outlets that discharge directly into a waterway channel should consider the following:

- The minimum and maximum invert elevations relative to the receiving channel bed level for earth, vegetated or otherwise erodible channels as presented in Table 8.5.1.

Table 8.5.1 – Minimum and maximum desirable elevation of pipe outlets above receiving water bed level for ephemeral waterways [1]

Pipe diameter (mm)	Minimum desirable elevation (m)	Maximum desirable elevation (m) based on $0.274/D^{0.5}$ [2]
300	0.30	0.50
450	0.30	0.41
525	0.30	0.38
600	0.30	0.35
750	0.30	0.32
825	0.30	0.30
900	N/A	0.29
1050	N/A	0.27
1200	N/A	0.25
1500	N/A	0.22
1800	N/A	0.20
2100	N/A	0.19

Notes (Table 8.5.1):

[1]   Sourced from Brisbane City Council (2003).

[2]   D = pipe diameter in metres.

8.5.6   Discharge to lakes

The design of stormwater outlets into lakes should consider the following issues. Consideration of these issues will in general be subject to the requirements of the local authority.

- Submerged outlets should be avoided for reasons of maintenance, including inspections and de-silting operations.

- Maintenance considerations—such as the safety of maintenance officers, and access for de-silting operations—must be considered before designing a submerged outlet.

- Ideally, a solid, possibly partially submerged, outlet apron should be provided as a stable entry platform for maintenance activities. Advice should be obtained from both the local government and the proposed asset owner (if not the local government).

- To help disguise or hide the outlet, consideration may be given to the placement of the outlet under an observation deck or other lake side structure.

- Consideration should be given to the installation of pollution control systems to control gross pollutants and sediment.

8.6   Backflow control systems

Stormwater pipes and drains can be subject to backflow in circumstances where flood levels within the receiving water rise above the water level within the pipe or drain. Backflows can be the result of normal tidal action, in which case it is mostly considered a desirable, natural feature of the drainage system, or a result of river flooding. Backflow prevention devices are used when it is desirable to limit the degree of backflow or likelihood of backflows. The most common types of backflow prevention devices include flap gates, duckbill valves and mechanically operated gates.

Backflow prevention devices can be used for the following reasons:

- to reduce the risk of coastal sediment/sand inflows into tidal drains
- to reduce saltwater intrusion into established freshwater habitats
- to reduce the frequency and/or severity of property flooding resulting from king tides
- to reduce the frequency and/or severity of backwater flooding of low-lying land adjacent floodplains
- to reduce the floodwater inundation of flood-prone land protected by flood control levees.

The operation of backflow prevention devices is notoriously problematic. The use of such devices should not occur without appropriate consideration of both the potential benefits and problems. Issues that should be considered include:

- potential barrier to essential movement of fish and other aquatic creatures
- potential for partial blockage and failure to close due to sediment, debris o the growth of marine organisms
- potential for partial blockage by stormwater debris passing down the stormwater drain, and the resulting impacts on local flooding
- increased risk of the breeding of mosquitoes and other biting insects
- potential for the backflow prevention device to increase the risk of erosion, either at the stormwater outlet or on the face of an associated levee bank, due to an increase in severity and frequency of overtopping flows.

Besides those issues listed above, the following should also be considered during the detail design process:

- control of backflow flooding resulting from floodwaters passing along electrical conduits, sewer lines and wildlife movement tunnels
- the need for real-time flood level monitoring that could potentially alert authorities of unacceptable water level differences across the backflow device altering the authority to a potential blockage or maintenance issue
- development of a monitoring and maintenance program that could be used during post flood analysis to demonstrate that the device was adequately maintained.

8.7   Outlet energy dissipation

8.7.1   General

Energy dissipation at stormwater outlets is usually required to achieve the following:

- control of bed scour
- control of erosion caused by a submerged 'outlet jet'.

The control of 'bed scour' is usually achieved by the development of a thick, low velocity, boundary layer, usually through the introduction of erosion resistant bed roughness (e.g. a rock pad).

The control of 'outlet jetting' is usually achieved by:

- reducing the outlet jet velocity prior to discharge (e.g. expansion chamber)
- reducing the outlet jet velocity post discharge (e.g. impact structures)
- splitting the outlet jet into several smaller jets (e.g. some types of impact structures)
- recessing the outlet into the bank to allow natural dissipation of the outlet jet prior to the jet impacting upon a waterway bank.

It is noted that the effective travel length of an outlet jet is related to the diameter or thickness of the jet; therefore, if the diameter of the jet can be reduced (e.g. by splitting the jet) then the effective travel length of the jet will be reduced.

Bank erosion is likely to result from the impact of a submerged outlet jet if all of the following conditions exist:

- tailwater levels are above the centre of the outlet
- the average velocity at the outlet exceeds those velocities presented in Table 8.6.1
- the distance between the outlet and the opposing bank is less than approximately 10 times the equivalent pipe diameter for a single outlet, or 13 times the equivalent pipe diameter for a multi-cell outlet.

Table 8.6.1 – Typical bank scour velocities

Bank condition	Typical bank scour velocity [1] (m/s)
Non vegetated banks:	
Highly erodible sandy-loam soils	0.5
Moderately erodible clay-loam soils	0.6
Lean clayey soils	0.6 to 1.2
Heavy clayey soils	0.7 to 1.5
Poorly vegetated banks:	
Banks with sparse groundcover	1.0 to 1.5 [2]
Well vegetated, erosion-resistant soils:	
Grassed banks	2.0
Banks with thick shrub and tree cover	2.5
Banks with a good, healthy coverage of fibrous-rooted herb layer plants such as *Lomandra*	3.0

Notes (Table 8.6.1):

[1]   Average jetting velocity impacting on a channel bank.

[2]   Depending on soil erodibility.

8.7.2   Attributes of various energy dissipaters

The attributes of various energy dissipaters and flow expansion devices are presented below.

(a)   Rock pad outlet
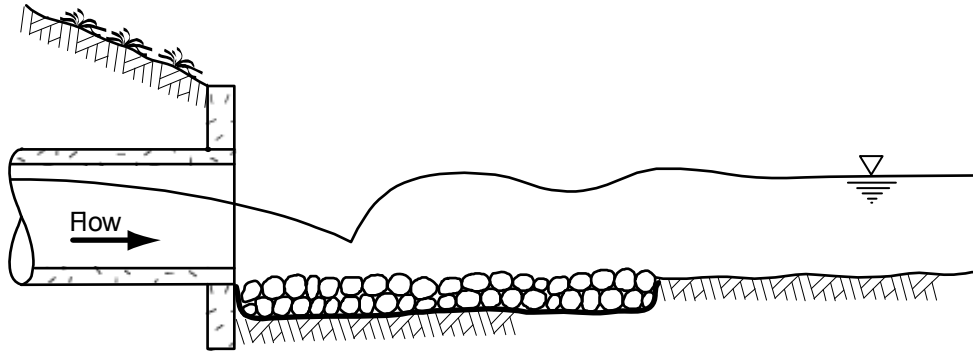
Figure 8.13 – Rock pad outlet

|	
Function:	Energy dissipation; boundary layer development; control of bed scour
Form of energy loss:	Bed friction
Tailwater conditions:	Designs exist for both high and low tailwater conditions
Jet control:	Control of plunging jet only (i.e. low tailwater condition)
Bed scour control:	Good control of bed scour can be achieved
Debris effects:	Low debris hazard
Safety issues:	Low safety hazard
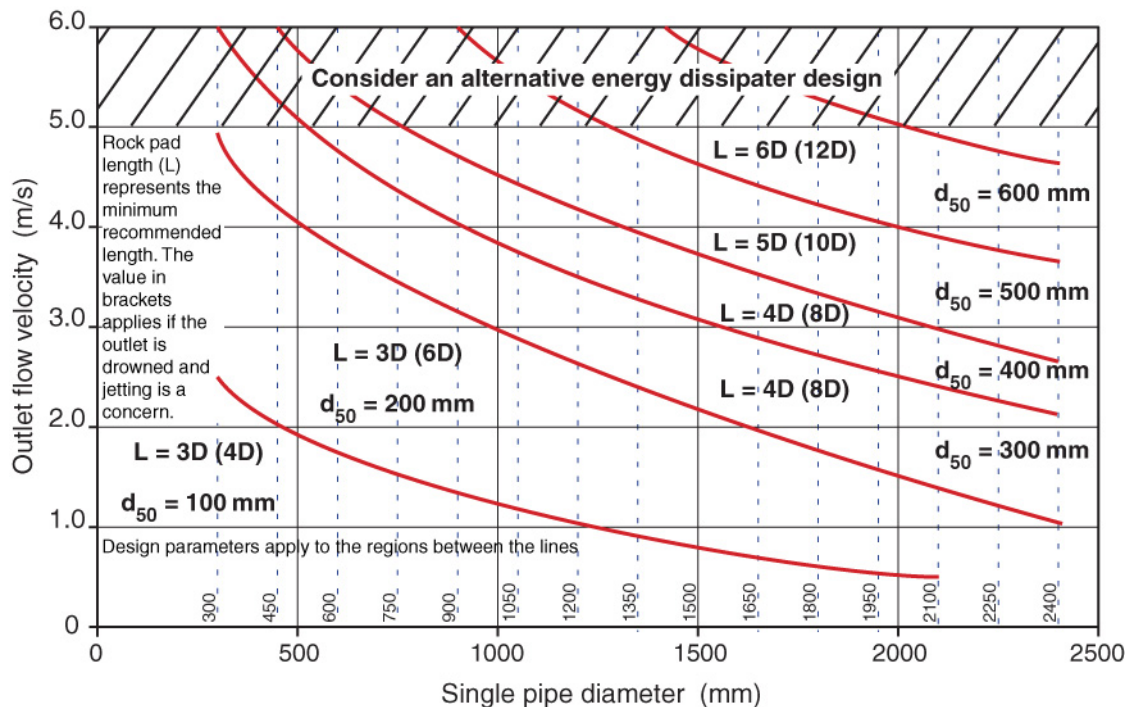
Figure 8.14 – Sizing of rock pads for single pipe outlets

The recommended minimum rock size (d_{50}) and length (L) of rock protection downstream of outlets may be determined from Figure 8.14. Rock pad sizing for multiple pipe outlets is provided in Figure 10.12 (as for multiple cell culvert outlets).

The minimum recommended width of the rock pad is defined as:

• Immediately downstream of the outlet: the width of the outlet apron, or the width of the outlet plus 0.6 m (if there is no apron).

• At the downstream end of the rock pad: the above width plus 0.4 times the length of the rock pad (L) as shown in Figure 8.15.

If the width of the outlet channel is less than the recommended width of the rock protection, then rock protection should extend up the banks to either the height of the pipe's obvert or to the design tailwater level.
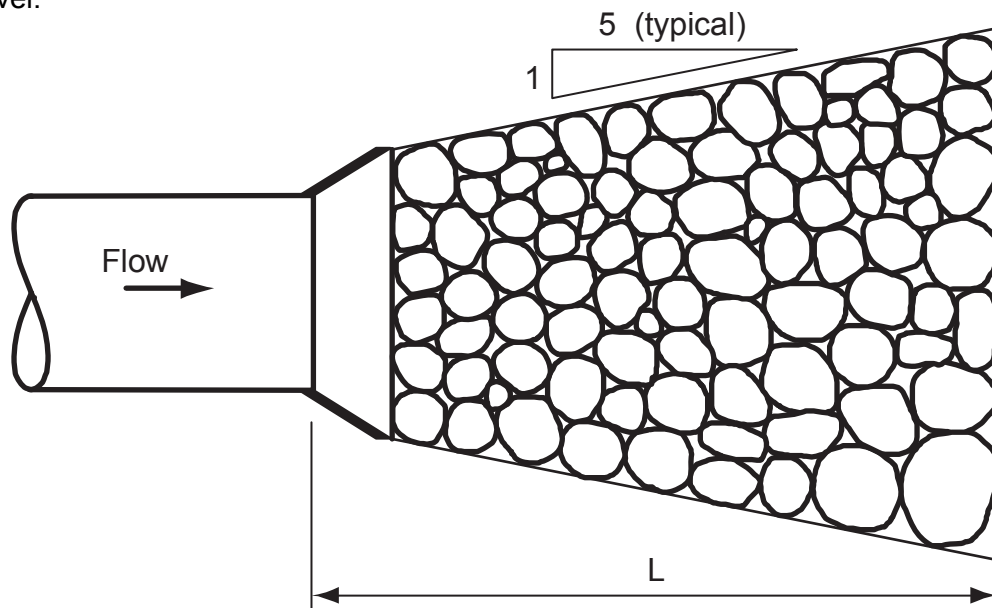

Figure 8.15 – Typical layout of a rock pad outlet structure

(b)   Rock mattress outlet
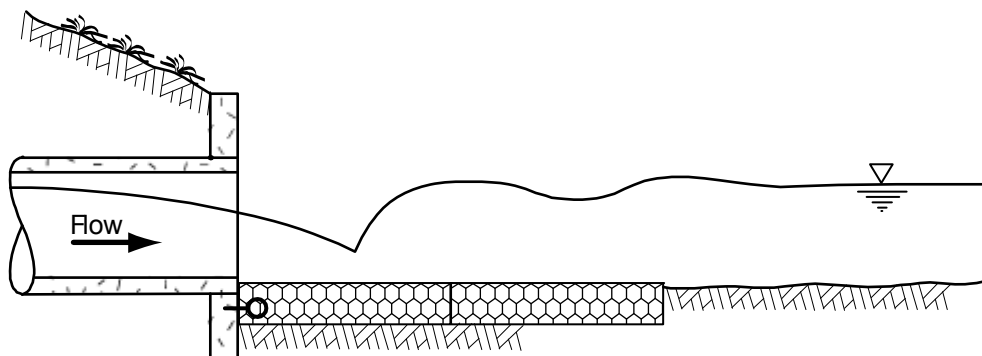
**Figure 8.16 – Rock mattress outlet**

Design reference: Valletine, Hattersely & Cornish (1961), Queensland Transport (1975)
Function: Prevent undermining head wall
Form of energy loss: Bed friction
Tailwater conditions: Effective at low tailwater only
Jet control: Control of plunging jet (i.e. low tailwater conditions); minimum control of submerged outlet jet

Bed scour control: Bed scour will still likely occur downstream of the rock mattress
Debris effects: Low debris hazard. Wire may be damaged by debris or high sediment flows
Safety issues: Low safety hazard, broken wire may represent a minor safety risk

(c)   Forced hydraulic jump basin
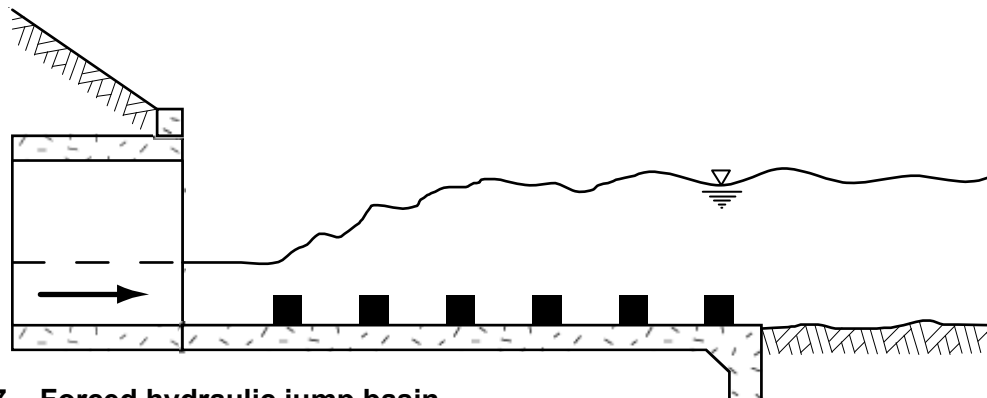
Figure 8.17 – Forced hydraulic jump basin

Design reference: U.S. Dept. of Transport (1983)
Function: Energy dissipation, boundary layer development, forced hydraulic jump
Form of energy loss: Bed friction, and hydraulic jump
Tailwater conditions: Tailwater requirements exist but are flexible, generally suitable for a range of tailwater conditions
Jet control: Provides minimum control of outlet jet unless an effective hydraulic jump forms
Bed scour control: Relatively good control of bed scour
Debris effects: Medium debris hazard
Safety issues: Medium safety hazard

(d)   Hydraulic jump chambers
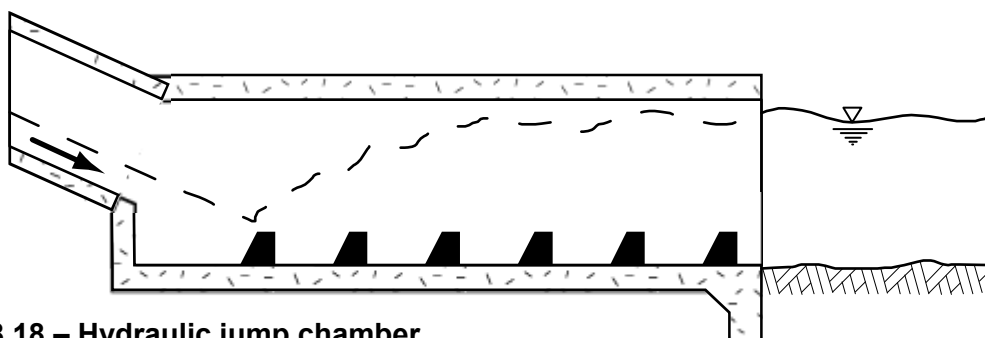
Figure 8.18 – Hydraulic jump chamber

Design reference: Korom, Sarikelle & Simon (1990)
Function: Energy dissipation, hydraulic jump control
Form of energy loss: Bed friction and forced hydraulic jump
Tailwater conditions: Maximum tailwater requirements exist, no minimum tailwater condition
Jet control: Jet control exists if an effective hydraulic jump is formed, otherwise the jet may pass through with minimum energy loss
Bed scour control: Minor bed scour may still occur downstream of the chamber, thus rock may be required
Debris effects: Low to medium debris hazard, but may be difficult to de-silt
Safety issues: Medium safety hazard

(e)   Riprap basin
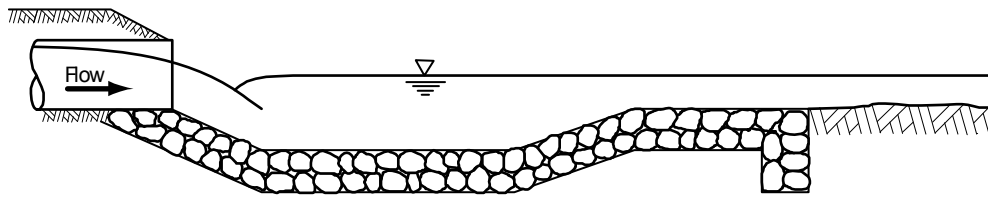
Figure 8.19 – Riprap basin

Design reference: ASCE (1992), U.S. Dept. of Transport (1983)
Function: Energy dissipation
Form of energy loss: Plunge pool
Tailwater conditions: Effective for tailwater levels less than 3/4 incoming jet height
Jet control: Good control of plunging jet, but minimal control of submerged jet
Bed scour control: Bed scour caused by a high velocity submerged jet can still occur
                         downstream of the structure
Debris effects: Low debris hazard
Safety issues: Low to medium safety hazard. Long-term pooling may occur unless the
                         plunge pool is raised above the channel thus allowing the pool to free drain
                         through a low-flow outlet slot

**(f)   Single pipe plunge pool outlet structure**
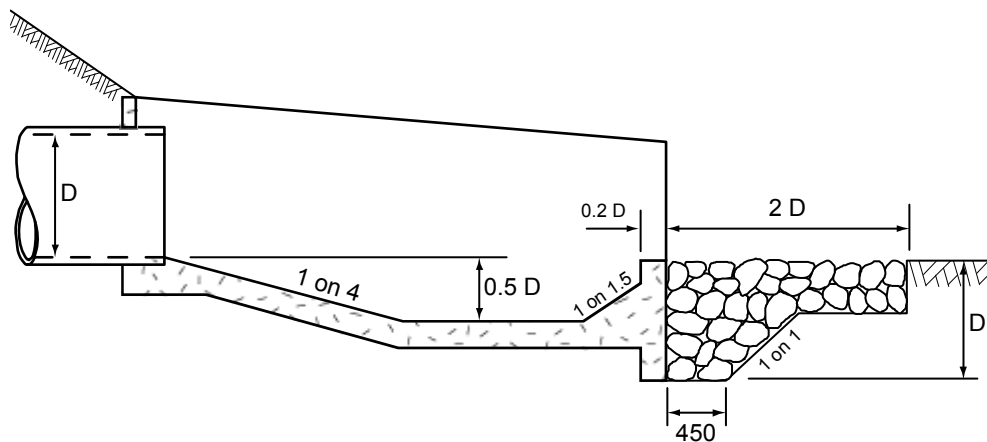
Figure 8.20 – Single pipe plunge pool outlet structure

Design reference: Argue (1960), Queensland Transport (1975)
Function: Energy dissipation, hydraulic jump control, flow expansion
Form of energy loss: Plunge pool and forced hydraulic jump
Tailwater conditions: Effective at low tailwater conditions
Jet control: Minimal control of high velocity submerged jets, but good control of
                         plunging jets
Bed scour control: Downstream rock protection is required if bed scour is to be controlled.
Debris effects: Low debris hazard
Safety issues: Medium to high safety hazard

**(g)   Twin pipe plunge pool outlet structure**
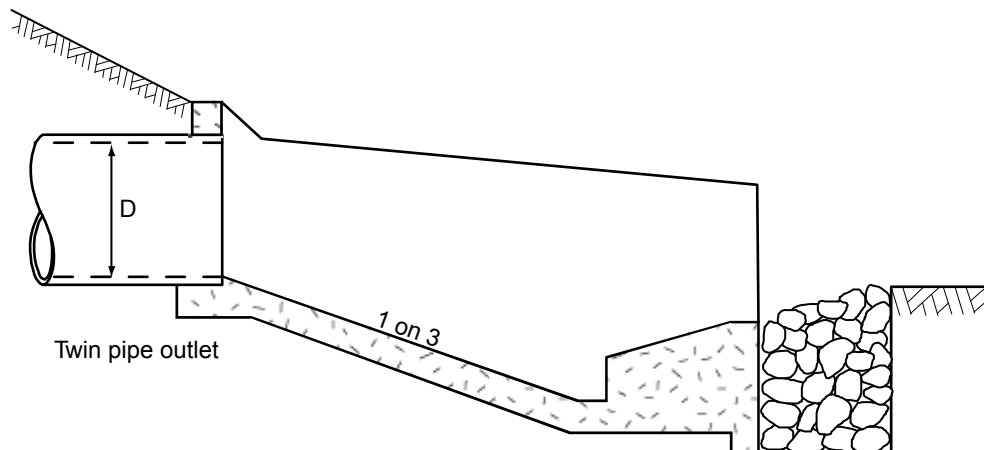
Figure 8.21 – Twin pipe plunge pool outlet structure

Design reference: O'Loughlin (1960), Queensland Transport (1975)
Function: Energy dissipation, hydraulic jump control, flow expansion
Form of energy loss: Plunge pool and forced hydraulic jump
Tailwater conditions: Effective at low tailwater conditions
Jet control: Minimal control of high velocity submerged jets, but good control of
                       plunging jets
Bed scour control: Downstream rock protection is required if bed scour is to be controlled.
Debris effects: Low debris hazard
Safety issues: Medium to high safety hazard

(h)   USBR Type VI impact basin
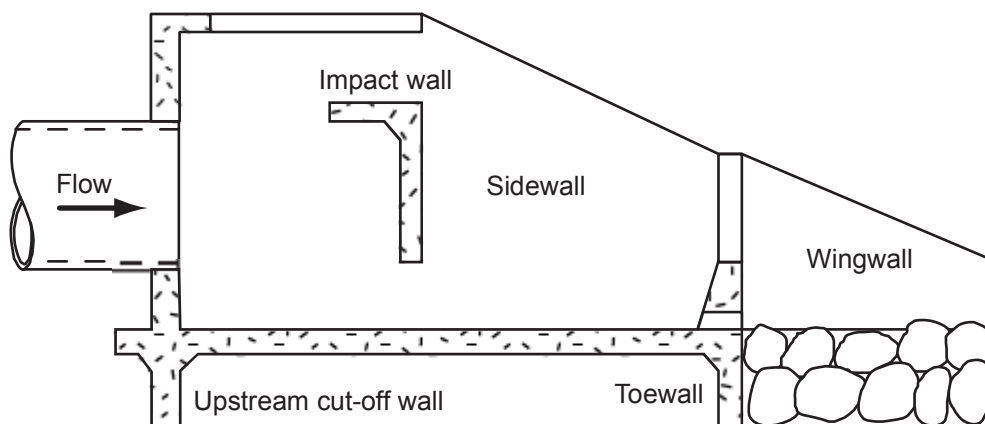
Figure 8.22 – USBR Type VI impact basin

Design reference: Peterka (1984), Meredith (1975), U.S. Soil Conservation (-), Rice & Kadavy
                       (1991), U.S. Dept. of Transport (1983), Standing Committee on Rivers and
                       Catchments (1991)
Function: Energy dissipation
Form of energy loss: Impact structure
Tailwater conditions: No tailwater requirements
Jet control: Control of high velocity outlet jet
Bed scour control: Bed scour will still occur and may require downstream rock protection
Debris effects: High debris hazard
Safety issues: Extreme safety hazard

(i)   Contra Costa energy dissipater
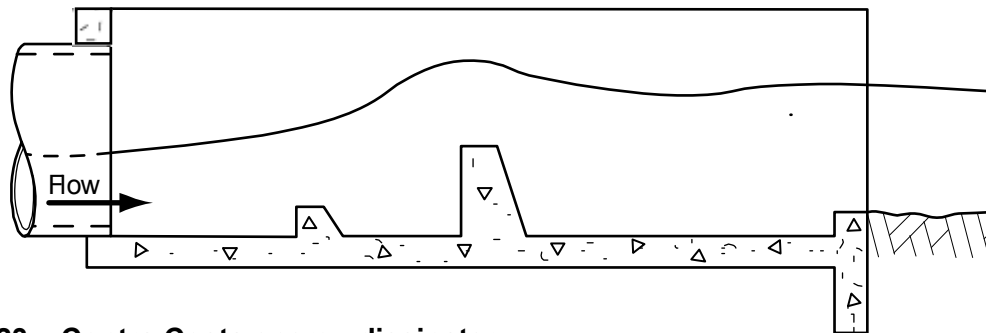
Figure 8.23 – Contra Costa energy dissipater

Design reference: Keim (1962), U.S. Dept. of Transport (1983)
Function: Energy dissipation
Form of energy loss: Impact structure and forced hydraulic jump
Tailwater conditions: Minimal tailwater requirements
Jet control: Good control of outlet jet if pipe flow is less than half full
Bed scour control: Bed scour will still occur and downstream rock protection may be required
Debris effects: Low to medium debris hazard
Safety issues: High to extreme safety hazard

(j)   Impact columns
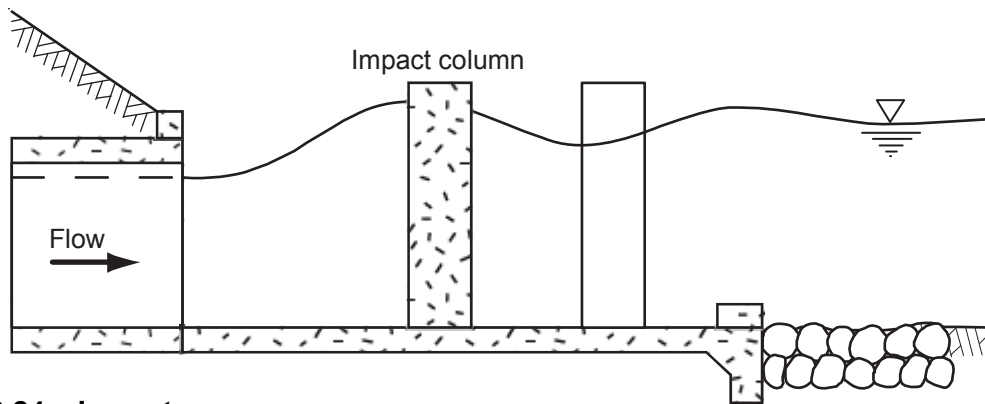
Figure 8.24 – Impact columns

Design reference: Brisbane City Council (2003), Smith & Yu (1966)
Function: Energy dissipation and flow expansion
Form of energy loss: Impact structure
Tailwater conditions: Suitable for high or low tailwater conditions
Jet control: Effective control of outlet jet
Bed scour control: Some control of bed scour
Debris effects: Medium to high debris hazard
Safety issues: Extreme safety hazard

9. Open channel hydraulics

9.1 General

This chapter provides guidelines on the design of constructed drainage channels, including the design of hard-lined, grassed and vegetated channels. Discussion has not been provided on the management of natural waterways and floodplains.

Guidance on the management and rehabilitation of urban waterways may be found in Brisbane City Council (1997, 2000). Guidelines on the design of waterway crossings (i.e. bridges, culverts, causeways, fords) are provided in Chapter 10 – *Waterway crossings*.

9.2 Planning issues

(a) Legislative requirements

Under the *Water Act 2000*, the Queensland Government may require approval of in-stream works (i.e. works below the top of the high bank), possibly including the acquisition of a 'Water Licence' or 'Riverine Protection Permit'. Some works may be conducted under the department's (DNRM) self-assessable guidelines; however, these guidelines do **not** include those requirements required by Queensland Fisheries (Department of Agriculture, Fisheries and Forestry). Fisheries also have an extensive selection of self-assessable codes dealing with a range of in-stream works and disturbances. If the works are to be carried out within tidal waters, then approval may be required from the Department of Environment and Heritage Protection.

As in all cases, the acquisition of an 'approval' from one government department does **not** mean that further approvals are not required before works can commence within a watercourse. The onus is on the designer / project manager to determine the extent of approvals required to conduct in-stream works.

(b) Retention of natural waterways

Consideration should be given to the retention of existing natural channels in the following circumstances:

- waterways identified as important within a Waterway Corridor Plan, Catchment Management Plan, or similar strategic plan
- waterways defined as fish corridors by Queensland Fisheries
- natural waterways with well-defined bed and banks, and associated floodways.

It is impractical to expect a pristine 'natural' channel to remain unchanged once the catchment has been urbanised. The degree of physical change experienced within a pristine waterway is dependent on a number of factors, but most importantly to the degree of change to the natural water cycle.

In circumstances where the existing channel is either heavily modified or degraded, then consideration should be given to the rehabilitation of the channel to a condition consistent with the proposed hydrologic and ecological conditions of the catchment.

Wherever practical, residential properties should not back directly onto vegetated channels, rather these waterway corridors should be viewed as a 'feature' of the urban landscape. The benefits of

providing an open buffer or grassed floodway (figures 9.6 and 9.7) between residential properties and urban waterways include:

- fire control
- maintenance access
- provision of public access to the waterway
- promotion of public usage of the waterway area
- public safety and crime control
- greater public ownership of the waterway
- reduced waterway pollution
- reduced dumping of grass clippings and garden waste over back fences into the waterway
- reduced community concerns regarding bushfire control and the management of problematic wildlife, such as snakes
- enhanced property values
- higher residential densities through the integration of park and open space requirements into waterway reserves.

(c)   Selection of channel type

Factors to be considered when choosing the configuration of a constructed open channel include:

- existing and likely future channel conditions immediately upstream and downstream of the channel in question
- existing and long-term (i.e. full catchment development) hydrologic conditions, including pollutant and sediment loadings
- local site constraints such as width restrictions, land ownership, existing services, location of natural and constructed features
- recognised environmental values
- aesthetics and landscaping of the overbank environment
- likely impact of channel surcharge (i.e. overbank flows)
- long-term ecological requirements of the channel, including aquatic and terrestrial habitat and corridor values
- existing and likely future community expectations
- heritage values relating to the waterway, especially those protected by the *Aboriginal Cultural Heritage Act 2003*
- safety risks to the public and maintenance personnel
- maintenance access requirements.

If current knowledge does not allow the confident selection of a drainage channel type that is consistent with the ecological needs of the area, then the channel design should, as a minimum:

- provide a drainage system that satisfies known requirements
- provide appropriate consideration of the likely future rehabilitation requirements of the channel, and where practical, provide sufficient easement width to allow for possible channel rehabilitation, including appropriate allowance for maintenance access.

This Manual recognises the following types of 'constructed' drainage channels:

C1   Hard lined (e.g. concrete) drainage channels
C2   Grassed trapezoidal channels with low-flow pipe
C3   Grassed trapezoidal channels with low-flow channel
C4   Vegetated trapezoidal channels
C5   Vegetated trapezoidal channels with low-flow channel
C6   Two-stage vegetated drainage channel and floodway
C7   Multi-stage vegetated drainage channel with low-flow channel and floodway

General attributes or features of these drainage channels are presented below as well as in Chapter 13 of *Australian Runoff Quality* (ARQ, 2005). Table 9.2.1 provides a general guide to the selection of channel configuration based on catchment area, sediment control and fauna requirements. Table 9.2.1 is not presented as mandatory design standard. This information has been provided as an example of possible catchment area limitations a local government may wish to consider when establishing its 'local' drainage design standard.

Table 9.2.1 – Typical attributes of various constructed drainage channels

	Description	Typical catchment area [1]	Tolerance to sediment flow	Fish passage corridor potential	Terrestrial passage corridor potential
C1	Hard lined channel	< 30ha	High	No	No
C2	Grass channel with low-flow pipe	< 30ha	Medium to high [2]	No	Limited
C3	Grass channel with low-flow channel	< 30ha	Medium to high	Limited	Limited
C4	Vegetated channel [3]	< 30ha	Low	Possible	Yes
C5	Vegetated channel with low-flow channel	30 to 60ha	Low	Yes	Yes
C6	Vegetated channel and floodway	> 60ha	Low	Yes	Yes
C7	Vegetated channel with low-flow channel and floodway	> 60ha	Low	Yes	Yes

Notes (Table 9.2.1):

[1]   Typical urban catchment areas for South East Queensland presented as a guide only. Actual values can be highly variable depending on local environmental requirements and hydrological conditions.

[2]   Low-flow pipes may experience sedimentation problems, including complete blockage of the pipe unless appropriately designed using normal pipeline drainage requirements. Only smooth-wall pipes should be used if the low-flow pipe has open inlets. Corrugated agricultural drainage pipes should only be used as a sub-surface drainage system with no open inlets.

[3]   Maximum channel flow rate is around 32 m³/s based on an average velocity of 1.5 m/s, a bank slope of 1 in 3, and freeboard of 300 mm.
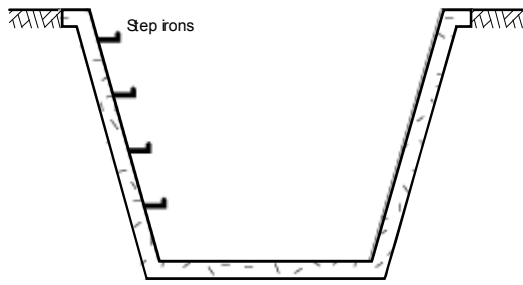
Figure 9.1 – C1: Hard lined channel

Features of a hard-lined channel:
- Poor water quality benefits
- Negligible ecological benefits
- High safety risks
- High hydraulic efficiency
- Low space requirements
- Typically 1% to 2% AEP capacity plus freeboard
- High thermal pollutant input
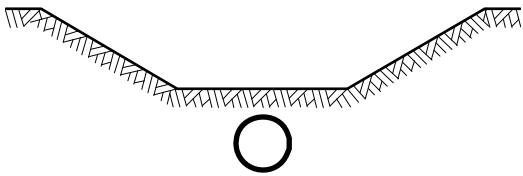
Figure 9.2 – C2: Grass channel with low-flow pipe

Features of a grassed channel:
- Limited water quality benefits
- Poor ecological benefits, but can provide limited terrestrial corridor values
- Good hydraulic efficiency
- Bank slopes of 1 in 6, but no steeper than 1 in 4
- Low-flow pipes can be susceptible to sediment blockage.
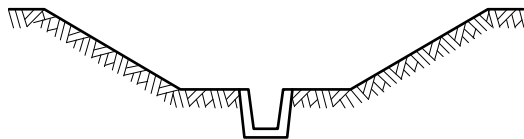
Figure 9.3 – C3: Grass channel with low-flow channel

Features of a grassed channel with open low-flow channel:
- Limited water quality benefits
- Poor ecological benefits, but can provide limited terrestrial corridor values
- Good hydraulic efficiency
- Bank slopes of 1 in 6, but no steeper than 1 in 4
- Low-flow channel can be susceptible to sediment blockage and weed growth
- Channel adjacent to the low-flow channel can be subject to waterlogging and/or erosion

**Vegetated channel with no formal low-flow channel:**
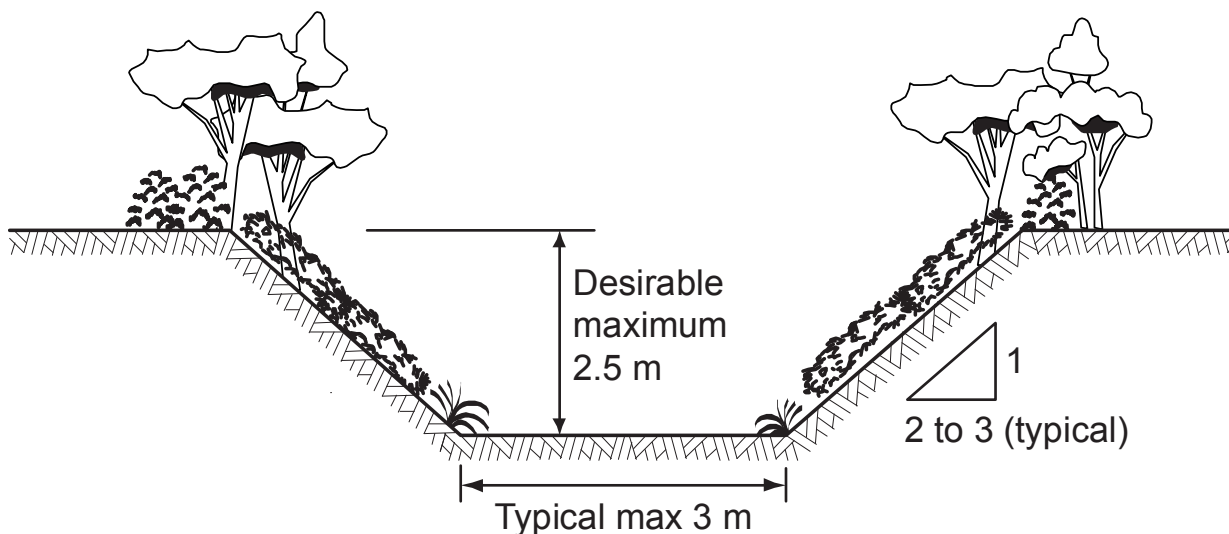
Figure 9.4 – C4: Vegetated channel with no formal low-flow channel

Typical features of a type C4 channel include:

- Channel banks vegetated with appropriate shrubs and understorey plants to provide desired hydraulic conveyance, public safety and wildlife habitat. Large trees generally should not be placed within the channel, unless located in an area of low velocity. Channel vegetation should not be dominated by grasses, even though some grasses and other ground covers will be required for scour control.

- The full bed width acts as the low-flow channel and may incorporate a pool-riffle system. Maintaining suitable bed conditions over the long-term may become impractical once the bed width exceeds 3 m, but exceptions do exist.

- Desirable maximum depth of around 2.5 m. Maximum channel depth may be limited by maintenance requirements.

- Rock protection is usually required along the toe of the banks.

- Public safety is generally addressed through the appropriate design and maintenance of bank and overbank (riparian) vegetation.

- Bank slopes of 1 in 2 to 1 in 3 (V:H) may be used provided appropriate bank vegetation is established to address public safety issues (i.e. woody vegetation with limited grass cover). Bank slopes steeper than 1 in 2 are generally not recommended except on the outside of bends and then only when adequate protection is provided against bank erosion.

- Desirable overbank maintenance berm width of 4.5 m on at least one side of channel.

- These channels can act as a terrestrial corridor and fauna habitat.

- If the channel is required to provide aquatic wildlife habitat, then:
  - Suitable bed conditions must be provided along the length of the channel.
  - Wide (>2 m) flat channel beds that provide shallow, uniform depth, low-flow conditions are generally not suitable. Instead, consideration should be given to channel types C5–C7.
  - Bank slopes steeper than 1 in 3 may be required to allow overbank trees to provide necessary shading of the channel bed for water temperature control.

**Vegetated trapezoidal channel with low-flow channel:**
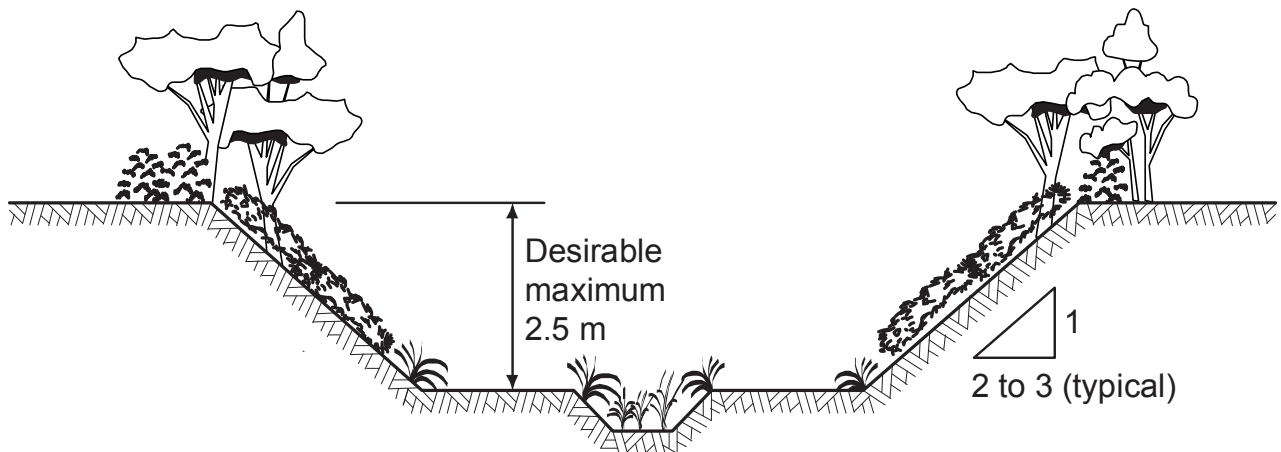
Figure 9.5 – C5: Vegetated trapezoidal channel with low-flow channel

Typical features of a type C5 channel include:

- A well-defined low-flow channel forms part of the channel bed to assist in fish passage and control soil moisture levels across the remaining channel bed. The low-flow channel may meander across the channel bed and may incorporate a pool-riffle system.

- Channel banks vegetated with appropriate shrubs and understorey plants to provide desired hydraulic conveyance, public safety and wildlife habitat. Large trees generally should not be placed within the channel, unless located in an area of low velocity. Channel vegetation should not be dominated by grasses, even though some grasses and other ground covers will be required for scour control.

- Desirable maximum channel depth is 2.5 m, otherwise excessive erosion and vegetation damage occurs during high flows and/or excessive sedimentation occurs during low flows.

- Bank slope requirements as for type C4 channel.

- The low-flow channel usually requires rock stabilisation to maintain stability. Rock protection may also be required along the toe of the banks.

- Vegetative shading of the low-flow channel is highly desirable.

- Woody vegetation may need to be limited to the channel banks if hydraulic capacity is critical. This can result in the development of an **undesirable**, high-maintenance channel requiring regular vegetation clearing.

- Desirable overbank maintenance berm width of 4.5 m on at least one side of channel, but typically both sides.

- Water quality improvements can occur during low flows, but very limited treatment during flood flows.

- The channel can act as a terrestrial corridor and fauna habitat.

- If the channel is required to provide aquatic wildlife habitat, then:
  - suitable bed conditions must be provided along the length of the channel
  - edge planting along the low-flow channel must provide adequate shading of the channel for water temperature control

- Formal public access may be provided into the channel, but is not recommended.

**Two-stage vegetated channel and floodway:**
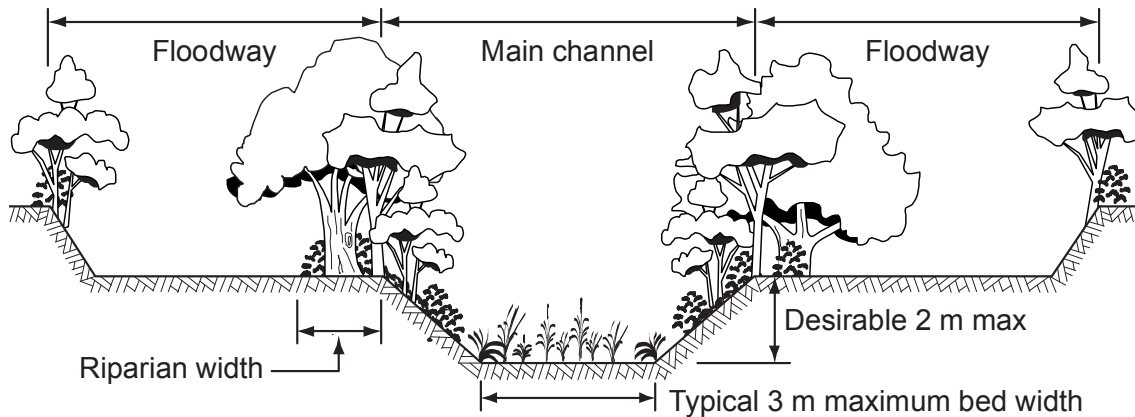
Figure 9.6 – C6: Two-stage vegetated channel and floodway

Typical features of the main channel of a type C6 constructed waterway include:

- Main channel capacity typically 63% (1 in 1 year) to 10% AEP (1 in 10 year). A main channel capacity in excess of 10% AEP should be avoided.

- Desirable maximum main channel depth of 2 m relative to top of the lower bank (i.e. base of floodway).

- The main channel is usually designed as a low-maintenance, heavily vegetated, closed canopy system.

- The full bed width acts as the low-flow channel. This usually becomes impractical in ephemeral streams once the bed width exceeds around 3 m.

- Water quality improvements can occur during low flows, but very limited treatment of flood flows.

- May provide aquatic passage if the channel bed has suitable aquatic features.

- Main channel may meander across the floodway to improve habitat diversity within the channel and to reduce the effective channel gradient; however, this meandering should not adversely affect the passage of floodwaters.

Typical features of the floodway of a type C6 constructed waterway include:

- Large space requirements.

- Typically 1% AEP floodway capacity plus freeboard (as applicable).

- An open (grassed) floodway allows better overbank flood flow; however, fully or partially vegetated floodways should be preserved wherever practical.

- Minimum desirable overbank riparian width is 5 m from the top of the lower bank. In ideal circumstances, overbank riparian widths of 15 to 60 m are recommended depending on the size and function of the waterway.

- Desirable maintenance berm width of 4.5 m each side of the channel.

- Minimum floodway cross slope of 1 in 80 to prevent waterlogging problems and allow regular maintenance mowing.

- These channels can act as a terrestrial corridor and fauna habitat. An open, grassed floodway can be used as a deterrent to prevent snakes entering residential properties.

- Formal public access and movement corridor may be provided along the floodway; however, riparian vegetation should be preserved as a buffer between public access areas and the vegetated channel to protect wildlife habitat values.

**Multi-stage vegetated channel with low-flow channel:**
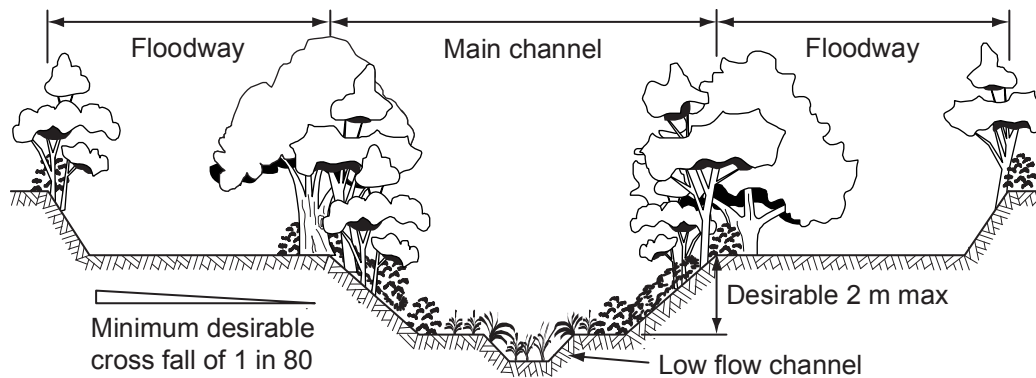
Figure 9.7 – C7: Multi-stage vegetated channel with low-flow channel

Typical features of the main channel of a type C7 constructed waterway include:

- Separate low-flow channel may be required to maintain desirable fish passage conditions and to control the spread of aquatic vegetation (e.g. reeds) across the channel bed (typically when the bed width exceeds 3 m).
- Low-flow channel capacity usually less than 1 in 1 year AEP.
- Main channel capacity typically 63% (1 in 1 year) to 10% AEP (1 in 10 year). A channel capacity in excess of 10% AEP should be avoided.
- Desirable maximum main channel depth of 2 m relative to top of the lower bank (i.e. base of floodway).
- Main channel is usually designed as a low-maintenance, heavily vegetated, closed canopy system.
- Water quality improvements can occur during periods of low flows.
- May provide aquatic passage if low-flow channel has suitable aquatic features.
- Main channel may meander across the floodway to improve habitat diversity within the channel and to reduce the effective channel gradient; however, this meandering should not adversely affect the passage of floodwaters.

Typical features of the floodway of a class C7 constructed waterway include:

- Large space requirements.
- Typically 1% AEP floodway capacity plus freeboard (as applicable).
- An open (grassed) floodway allows better overbank flood flow; however, fully or partially vegetated floodways should be preserved wherever practical.
- Minimum desirable overbank riparian width is 5 m from the top of the lower bank. In ideal circumstances, overbank riparian widths of 15 to 60 m are recommended depending on the size and function of the waterway.
- Desirable maintenance berm width of 4.5 m each side of the channel.
- Minimum floodway cross slope of 1 in 80 to prevent waterlogging problems and allow regular maintenance mowing.
- Can act as a terrestrial corridor and fauna habitat. An open, grassed floodway can be used as a deterrent to prevent snakes entering residential properties.
- Formal public access and movement corridor may be provided along the floodway; however, riparian vegetation should be preserved as a buffer between public access areas and the vegetated channel to protect wildlife habitat values.

9.3   Open channel hydraulics

9.3.1   Hydraulic analysis

Uniform flow conditions rarely occur within vegetated drainage channels, the general conditions being that of gradually varied flow including possible fluctuations between subcritical flow and supercritical flow at isolated locations along the channel. It is the responsibility of the designer to identify variations in flow conditions that are likely to occur along the channel and to design the channel accordingly.

The procedures and recommendations presented in this chapter refer to the design of typical open channels operating primarily under subcritical flow conditions. Open channels operating under or approaching supercritical conditions should be avoided. Where such situations cannot be avoided, specialist design knowledge may be required. It is generally considered desirable to limit the Froude Number in open channels to 0.9 wherever practical to avoid the formation of unstable flow conditions.

Backwater analysis is usually required to establish a complete water surface profile. It is the designer's responsibility to be familiar with the computational procedures and limitations of any numerical modelling programs used.

9.3.2   Design flow

Discussion on the selection of the design storm frequency for open channels is provided in section 7.3. The 1% AEP is commonly adopted for the design of major waterways and drainage paths where it is difficult to predict actual flow conditions (e.g. channels subject to complicated 3D hydraulics) and for vegetated channels where the surface roughness can vary significantly throughout its design life.

The likely effects of channel flows resulting from a storm event in excess of the major design storm should be considered and the consequences discussed with the local government (refer to sections 7.2.4 & 7.3.3). Designers should refer to the requirements of the Government's State Planning Policies with respect to the design of floodways.

9.3.3   Starting tailwater level

Tailwater levels for the hydraulic assessment of drainage channels may be determined from a variety of sources. Where appropriate, the local authority may supply starting water levels at a downstream location based upon previous catchment modelling.

Guidelines on the selection of appropriate tailwater conditions for tidal and non-tidal channel outlets are provided in Chapter 8 – *Stormwater outlets*.

In locations where the reliability of the tailwater level is questionable, the hydraulic analysis should be extended downstream a sufficient distance to minimise its influence on the backwater analysis. A sensitivity analysis should be performed on a range of possible tailwater levels to confirm the downstream extent of the hydraulic analysis.

At locations where the proposed drainage channel discharges into a larger waterway, the effects of coincident flooding must be considered. In such cases, the joint probability of simultaneous runoff events occurring in both catchments needs to be assessed. This may require the adoption of appropriate aerial reduction factors as discussed in ARR (1998). Discussion on coincident flooding is provided in section 8.3.4 of this Manual.

9.3.4 Channel freeboard

In open channel design, the term channel 'freeboard' generally refers to the vertical distance between the design water surface elevation and the top of the channel bank as shown in Figure 9.8. However, 'freeboard' is more commonly specified relative to adjacent floor levels, meaning that design flows may be allowed to surcharge above the main channel banks depending on the depth and width of the overbank floodway.

Drainage channels can surcharge as a result of a number of factors including design errors, construction problems, minor wave action (caused by wind, flow turbulence and lateral inflows) channel sedimentation, or minor seasonal change to vegetation roughness. The existence of a channel freeboard should **not** be used as an excuse to ignore those design conditions that would reasonably be expected to influence the calculated design water surface elevation.

Channel freeboard (relative to the top-of-bank) is incorporated only if the consequences of channel surcharge are considered undesirable, such as flooding of adjacent land and buildings. Table 9.3.1 provides recommendations on channel freeboard. Alternatively, the local authority may agree to a set freeboard based on a risk assessment.

Table 9.3.1 – Recommended channel freeboard [1]

Recommendation	Condition
Adopt maximum of conditions (a), (b) or (c)	(a) 300 mm (b) 20% of channel depth (c) flow velocity head

Note (Table 9.3.1):

[1] Specified freeboard is relative to the top-of-bank (Figure 9.8), and is only relevant in cases where channel surcharge during the major design storm is undesirable. Otherwise, freeboards specified within section 7.3 (overland flow) or within the Governments State Planning Policies (floodplains) shall apply.
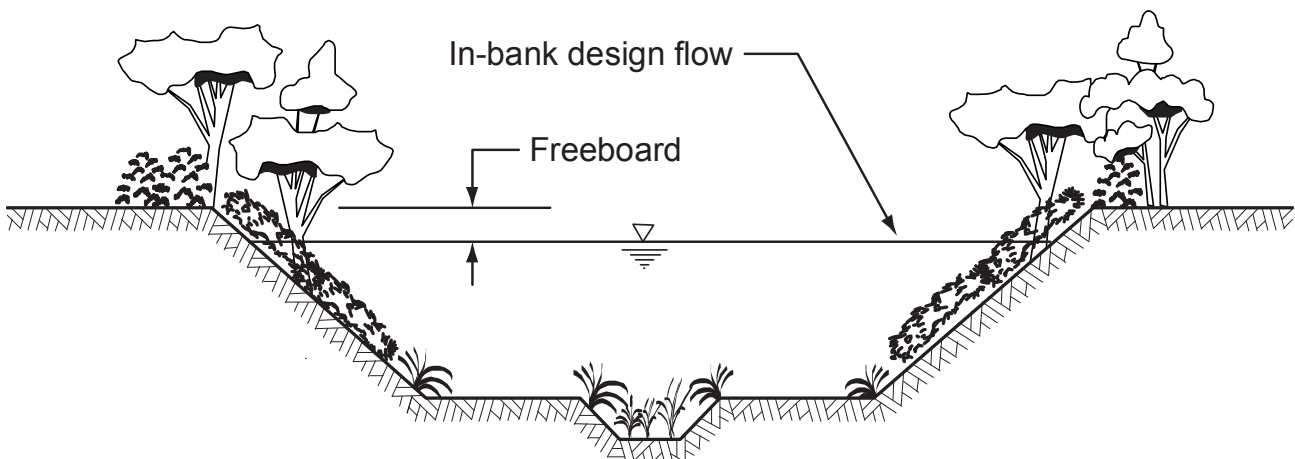

Figure 9.8 – Channel freeboard

9.3.5   Use of Manning's equation

(a)   General

Manning's equation (equation 9.1) is most commonly used throughout Australia for the analysis of uniform flow conditions within constructed channels.

$$V = (1/n)\ R^{2/3}\ S^{1/2} \qquad\qquad (9.1)$$

where:

V = average flow velocity (m/s)
n = Manning's roughness
R = hydraulic radius = A/P (m)
A = effective channel flow area (m^2)
P = wetted perimeter (m)
S = channel slope (uniform flow conditions) (m/m)

(b)   Selection of Manning's roughness

The choice of an appropriate value for the Manning's roughness coefficient is critical and requires a considerable degree of judgement. In addition to the following discussion, designers are referred to Book 7 of ARR (1998) for discussion on the selection of Manning's roughness.

Appropriate consideration should be given to the following factors when selecting a design channel roughness value:

(i)   The hydraulic capacity of a channel should be based on the expected channel conditions just prior to normal channel maintenance (i.e. prior to clearing, weeding, grass cutting).

(ii)   To minimise the ongoing cost of channel maintenance and flood control activities within vegetated channels, local governments are encouraged to nominate 'minimum design roughness values' for selected vegetative conditions. Table 9.3.2 provides guidelines on the selection of minimum design roughness values for the assessment of maximum design water levels; however, these values should be adjusted for local conditions.

Table 9.3.2 – Typical minimum design roughness values for vegetated channels

Vegetation and maintenance conditions	Manning's (n)
Bank and overbank vegetation (not grasses) with little or no vines and little or no ongoing maintenance.	0.15
Bank and overbank vegetation (not grasses) with vines and little or no ongoing maintenance.	0.2

(iii) Consideration shall also be given to expected 'flow velocities' that will occur during periods of low-roughness, i.e. immediately after construction and after regular channel maintenance. Typically, the assessment of low-roughness flow velocities is based on a more frequent design storm such as the 39% AEP (1 in 2 year) design storm rather than the Major Design Storm.

(iv) The Manning's roughness used in the design of irregular, non-uniform channels must not be based solely on the assessed surface roughness, but also on the influence of the following factors:

o   degree of channel and surface irregularity

- o   rate of variation of channel cross-section
- o   degree of in-channel obstructions (e.g. boulders and logs)
- o   density and type of vegetation
- o   degree of meandering.

Appendix C of Brisbane City Council (2000a) provides guidelines on the relative impact of the above variables.

(v)   The presence of a significant sediment flow will increase the effective channel roughness.

(vi) Manning's roughness generally decreases with increasing flow depth. Roughness values provided in design charts such as Chow (1959) may need to be adjusted for actual flow depths. Tables 9.3.3 and 9.3.4 provide typical roughness values for rock-lined and grass channels under various flow depth.

(vii) For rock lined channels and chutes, consideration must be given to the likelihood of the rocks eventually being covered with vegetation. This vegetation may increase or decrease the effective channel roughness. Thus the channel's maximum flow velocity and hydraulic capacity requirements may need to be analysed for different roughness values.

Table 9.3.3 – Manning's roughness of rock lined channels with shallow flow

d_{50} = (mm)	$d_{50}/d_{90} = 0.5$				$d_{50}/d_{90} = 0.8$			
	200	300	400	500	200	300	400	500
R (m)	Manning's roughness (n)				Manning's roughness (n)			
0.2	0.10	0.14	0.17	0.21	0.06	0.08	0.09	0.11
0.3	0.08	0.11	0.14	0.16	0.05	0.06	0.08	0.09
0.4	0.07	0.09	0.12	0.14	0.04	0.05	0.07	0.08
0.5	0.06	0.08	0.10	0.12	0.04	0.05	0.06	0.07
0.6	0.06	0.08	0.09	0.11	0.04	0.05	0.05	0.06
0.8	0.05	0.07	0.08	0.09	0.04	0.04	0.05	0.06
1.0	0.04	0.06	0.07	0.08	0.03	0.04	0.05	0.05

The roughness values presented in Table 9.4.2 have been developed from equation 9.2 (modification of Witheridge, 2002). Equation 9.2 may be used to estimate the Manning's roughness 'n' of rock lined channels in shallow water.

$$n = \frac{(d_{90})^{1/6}}{26(1 - 0.359^{m})} \qquad (9.2)$$

where:

m = $[(R/d_{90})(d_{50}/d_{90})]^{0.7}$

R =   Hydraulic radius of flow over rocks [m]

d_{50} =   mean rock size for which 50% of rocks are smaller [m]

d_{90} =   mean rock size for which 90% of rocks are smaller [m]

In 'natural' gravel-based streams the factor d_{50}/d_{90} is typically in the range 0.2 to 0.5, while in constructed channels where imported graded rock is used, the ratio is more likely to be in the range 0.5 to 0.8.

Table 9.3.4 – Manning's roughness for grassed channels (50–150 mm blade length)

Hydraulic radius (m)	Swale slope (%)					
	0.1	0.2	0.5	1.0	2.0	5.0
0.1	—	—	—	0.105	0.081	0.046
0.2	—	0.091	0.068	0.057	0.043	0.030
0.3	0.078	0.064	0.053	0.043	0.031	0.030
0.4	0.063	0.054	0.044	0.033	0.030	0.030
0.5	0.056	0.050	0.038	0.030	0.030	0.030
0.6	0.051	0.047	0.034	0.030	0.030	0.030
0.8	0.047	0.044	0.030	0.030	0.030	0.030
1.0	0.044	0.044	0.030	0.030	0.030	0.030
>1.2	0.030	0.030	0.030	0.030	0.030	0.030

Note (Table 9.3.4):

1. Manning's values determined from vegetation retardance Chart-D (Department of Main Roads, 2002a). Values are presented to three significant figures for convenience. This should not imply the values are accurate to three significant figures. A Manning's roughness of 0.03 is adopted for hydraulic radius greater than 1.2 m in accordance with recommendations of original research; however, this may not always be appropriate. Vegetation retardance charts for grass length <50 mm, 150–250 mm, 280–610 mm and >750 mm may be found in Department of Main Roads (2002a).

The following references provide information on the selection of Manning's roughness:

1. Argue (1986) Table 6.1, Books 7 and 8 of ARR (1998), Henderson (1966) Table 4.2, Chow (1959) Table 5.6.

2. Photographs and descriptions of channels with known roughness coefficients, e.g. Brisbane City Council (2000a), Chow (1959), Barnes (1967), French (1985), Hicks & Mason (1991) and Arcement & Schneider (1989).

   *Caution: Hicks & Mason (1991) provide roughness values usually relating to low-flow conditions, **not** to bankful or overbank conditions presented in the photos. Arcement & Schneider (1989) provide roughness values for vegetated floodplains in the USA; however, the supplied photos show the vegetation in winter conditions (i.e. low leaf matter) even though the roughness values refer to summer conditions (i.e. dense leaf and vine matter).*

3. Equations to derive Manning's roughness coefficients for complex channels may be found in Brisbane City Council (2000a), Book 7 of ARR (1998), Chow (1959) Table 5.5 and French (1985).

(c)   Channels of composite roughness

A composite roughness value may need to be determined when surface roughness varies significantly within a channel cross-section. Variations in in-channel and overbank (floodway) roughness should be treated separately. Guidelines on the determination of composite roughness values are provided in Book 7 of ARR (1998). Alternatively, composite roughness values may be determined by establishing uniform flow conditions within a 1D numerical hydraulic model.

The equation used for determining composite Manning's roughness values is:

$$n = P.R^{5/3} / \Sigma[P_i.R_i^{5/3}/n_i] = (A^{5/3}/P^{2/3}) / \Sigma(A_i^{5/3}/n_i.P_i^{2/3}) \qquad (9.3)$$

where:

n = equivalent composite Manning's roughness coefficient for entire cross-section
P = wetted perimeter of whole cross-section
R = hydraulic radius of whole cross-section = A/P
n_i = Manning's roughness coefficient for segment i
P_i = wetted perimeter of segment i
R_i = hydraulic radius of segment i

It is recommended that use of equation 9.3 be restricted to simple channel sections. Overbank flow conditions should not be incorporated into the composite roughness, but should be treated as part of a compound cross-section.

(d)   Compound cross-sections

A 'complex' channel cross-section consisting of a deep channel and adjacent shallow floodways may be referred to as a compound cross-section. The hydraulic analysis of compound cross-sections is described in Book 7 of ARR (1998).

Compound cross-sections are usually best analysed using numerical hydraulic models.

(e)   Effective wetted perimeter for channels with wide floodways

The composite channel roughness for channels with wide floodways should be used in conjunction with an effective channel wetted perimeter P^* which can be determined from equation 9.4 or 9.5 whichever is the lesser.

$$P^* = A^{5/2} / [\Sigma(A_i^{5/3} / P_i^{2/3})]^{3/2} \qquad (9.4)$$

$$P^* = P \qquad (9.5)$$

where:

P^* = effective channel wetted perimeter
A = area of whole cross-section
A_i = area of segment i

(f)   Sensitivity analyses

The designer should ensure that the parameters adopted in the design of an open channel system accurately represent the range of anticipated conditions that could reasonably be expected to occur throughout the design life of the drainage system. Specifically, it is important to assess the hydraulic capacity of vegetated channels for both the lowest and highest likely values of channel roughness (as discussed in section 9.3.5 (b) above).

In open channel design, a sensitivity analysis generally includes modelling the system for a range of assumed Manning's roughness coefficients, or modelling the system with a modified cross-sectional shape to account for the effects of sedimentation and/or scour.

9.3.6 Energy losses at channel transitions and channel bends

(a) Energy losses associated with channel transitions

Energy losses occur within a channel as a result of changes in cross-sectional shape. As a general rule, the loss associated with flow expansion is larger than that associated with flow contraction, whilst losses associated with abrupt transitions are greater than losses associated with gradual transitions.

Recommended configurations for gradual transitions consist of maximum contraction rates of about 1 on 1, and maximum expansion rates of about 1 on 4. The transition loss is determined by applying a transition loss coefficient to the 'absolute' value of the difference in velocity head between sections upstream and downstream of the transition (equation 9.6). Recommended values for loss factors are presented in Table 9.3.5.

Table 9.3.5 – Channel transition energy loss coefficients (C_u)

Transition type	Contraction coefficient	Expansion coefficient
Gradual channel transition	0.1	0.3
Typical bridge transition	0.3	0.5
Square edged abrupt transition	0.6	0.8

$$h_t = C_u. \, ABS \, [(V_1^2/2g) - (V_2^2/2g)] \tag{9.6}$$

where:

h_t = transition head loss
C_u = transition energy loss coefficient
V_1 = average flow velocity upstream of transition
V_2 = average flow velocity downstream of transition
ABS = means absolute value

(b) Energy losses associated with channel bends

The presence of a bend in an open channel creates a backwater effect similar to that associated with a channel obstruction. The magnitude of head losses associated with channel bends is a function of a number of parameters including bend radius, channel width, flow depth, Froude number, and relative change in direction.

As discussed in Chow (1959), Henderson (1966) and French (1985) experimental work has been undertaken on this subject, but the results are far from conclusive. Equation 9.7 is derived from work undertaken by Mockmore (1944) and based on experimental data taken from artificial and natural channels with changes in direction ranging between 90 and 180 degrees. Results obtained using this equation should be considered conservative, given that comparisons with alternative experimental data indicate discrepancies of between 300 and 400 percent in certain situations. The following equation is nevertheless recommended for use in estimating head losses associated with bends in open channels:

$$h_b = (2B/R_c).(V^2/2g) \tag{9.7}$$

where:

h_b = channel bend head loss
B = channel width
V = average flow velocity
g = acceleration due to gravity
R_c = centreline radius of bend

Note: *Equation 9.7 is applicable for channel bends with changes in direction of between 90 and 180 degrees. For bends with changes in direction of between 0 and 90 degrees, linear interpolation is recommended.*

(c)   Superelevation associated with channel bends

When flow moves around a channel bend, a rise in the water surface elevation occurs along the outer radius of the bend, whilst a corresponding lowering in the water surface elevation occurs along the inner radius of the bend. This difference in water levels is known as the superelevation, and in some cases may be an important factor in channel design.

A comprehensive procedure for calculating channel superelevation, based on the assumption of theoretical free-vortex velocity distribution, is presented in Chow (1959). Designers, however, may choose to calculate superelevation using a less accurate, but simpler procedure, based on the application of Newton's second law of motion to the centrifugal flow around the channel bend. By applying Newton's second law of motion to each streamline of the flow as it travels around the bend, it is possible to demonstrate that the transverse water surface profile across the channel is a logarithmic curve, and that the superelevation can be estimated using equation 9.8.

$$h_{sup} = (2.3V^2/g).log_{10} (R_o/R_i) \tag{9.8}$$

which may also be expressed as:

$$h_{sup} = [2log_e (R_o/R_i)].(V^2/2g) \tag{9.9}$$

where:

h_{sup} = superelevation of the water surface across the channel (difference in level)
V = average flow velocity
g = acceleration due to gravity
R_o = outer radius of bend
R_i = inner radius of bend

9.4   Constructed channels with hard linings

9.4.1   General

Channels lined with concrete, stone pitching and rock mattresses are typical of the type of channel included in this category. Channels of this type are often used in situations where severe easement width restrictions exist, or where the channel gradient is steep resulting in high flow velocities.

9.4.2   Contraction and expansion joints

Contraction and expansion joints should be provided within concrete lined or stone pitched channels to allow articulation and to accept minor temperature and environmental movements, thereby minimising the risk of cracking and subsequent undermining and failure.

If a hydraulic jump is likely to move over a construction/expansion joint, then extra joint reinforcing may be required to prevent displacement of the concrete slabs. Such displacement of the concrete is caused by a rapid change in hydraulic pressure—as much as 40% of approaching velocity head—under the slab resulting from the hydraulic jump moving over the slab. Technical guidelines are provided in Peterka (1984).

9.4.3   Step irons

Step irons should be provided in concrete lined and stone pitched channels, where the channel side slope is steeper than 1 in 2 (1V in 2H) and the channel depth exceeds 0.9 m. Step irons should be spaced at 0.3 m vertical interval. The maximum longitudinal spacing between step irons should not exceed 60 m.

9.4.4   Pressure relief weep holes

Pressure relief weep holes should be provided in channels lined with impervious material such as concrete, grouted stone pitching, and grouted rock mattresses, both within the channel invert and within the channel side slopes. The extent and density of pressure relief weep holes should be sufficient to prevent hydraulic uplift of the channel, and should satisfy the conditions of the local authority.

9.4.5   Treatment of channel inverts

The inverts of channels lined with concrete or stone pitching should have a nominal invert vee of at least 1 in 10, such that low flows remain concentrated along a single location within the channel invert. This invert vee may be either centrally located or offset to one side of the channel invert.

9.4.6   Lateral protection and cut-off walls

The lining of hard faced channels should extend horizontally beyond the top edge of the channel, at least 0.45 m wide on both sides. This horizontal strip of hard-faced material will assist in providing scour protection against lateral inflows to the channel, and in prevention of undermining.

Vertical cut-off walls are required at the upstream and downstream extents of the lined channel. These cut-off walls should be provided along the channel invert and up the channel side slopes. The required depth of cut-off walls is dependent on a number of factors including channel flow rate, flow velocity, and type of natural material upstream and downstream of the lined section.

Designers should consult Chiu & Rahmann (1980) and Peterka (1984) for procedures concerning the determination of required cut-off wall depths. A minimum depth of cut-off wall penetration of 0.6 m is recommended unless otherwise directed by the local authority.

9.4.7   Downstream scour protection

Designers should ensure that scour beyond the downstream end of lined channels is prevented, or at least reduced to an acceptable level.

Bed scour typically occurs immediately downstream of hydraulically-smooth channels for one or more of the following reasons:

(i)   Average flow velocity of the water discharging from the hard-lined channels is excessive for the downstream surface conditions.

(ii) The 'smooth' upstream channel produces a thin boundary layer that attracts high flow velocities and shear stresses close to the surface of the channel lining. This smooth-wall velocity profile is inconsistent with the velocity profile required within the downstream channel. Such flow conditions are shown in Figure 9.9.

To avoid the scour problems discussed in (ii) above, it is desirable to pass the discharging water over a roughened surface before releasing it into a vegetated channel. This is normally achieved by placing a rock scour pad at the exit of the smooth-bed channel. It should be noted that it is the 'length' and 'roughness' of the rock pad that is critical, thus the use of rock mattress outlet pads—which are hydraulically-smooth compared to large loose rock—can be problematic.
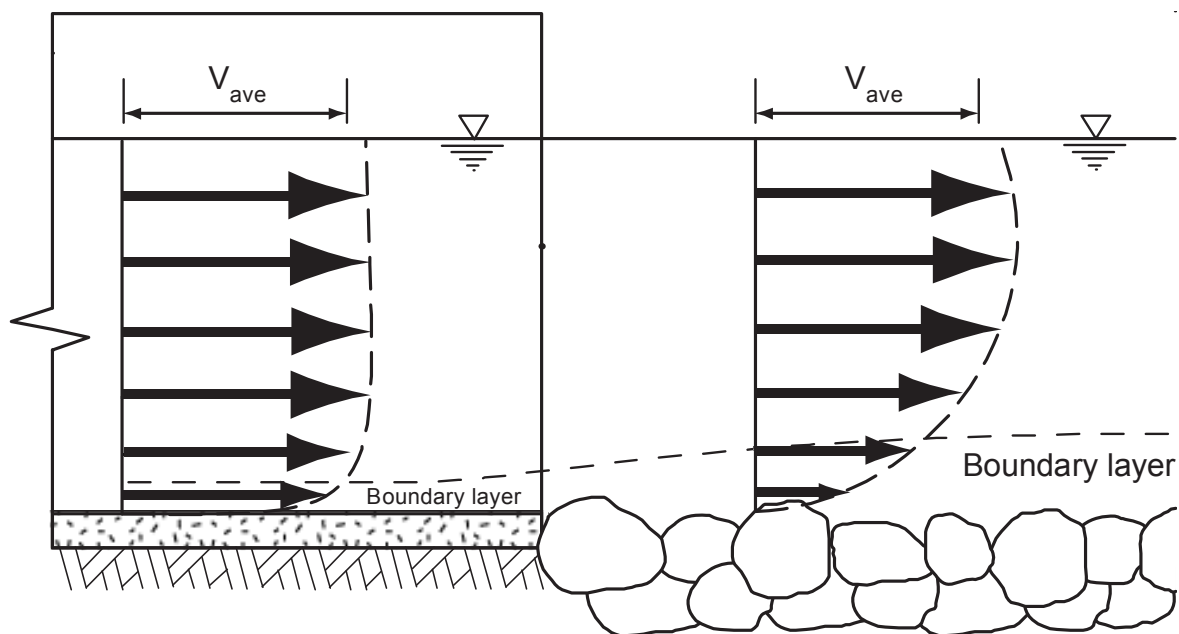

Figure 9.9 – Boundary layer conditions for flow passing from a smooth channel surface onto a rough channel surface

9.4.8 Rock mattress channels

In many environments, the long-term success of hydraulic structures formed from gabions and/or rock mattresses depends on the successful establishment of vegetation over the wire baskets. In most aquatic environments significant damage to the wire mesh is inevitable even if the wire is galvanised and plastic coated. This damage can be caused by the movement of woody debris or sediment flow. The exception may be in semi-arid areas where the design life of non-vegetated rock mattresses is generally considered acceptable.

Certain acidic or polluted aquatic environments may also cause rapid deterioration of wire baskets, in which case it will be essential for a desirable vegetative cover to be promptly established over the baskets.

The vegetation used to cover wire baskets should be aesthetically pleasing, self maintaining in a manner consistent with the channel's nominated design roughness, have a root system capable of binding the rock fill together, be environmentally sensitive, and where possible, native to the area.

If a successful vegetative cover cannot be established, then the maintenance of rock mattress channels can be labour intensive and their operational life can be significantly reduced.

If left non-vegetated, gabions and rock mattresses can be colonised by vine species transported by stormwater runoff from upstream residential landscapes. Consideration should be given to the risk of these vines migrating into adjacent bushland.

9.5 Constructed channels with soft linings

Fully vegetated channels, or channels lined with grass are typical of the types of channel included in the soft-lining category.

9.5.1 Reducing flow velocities in channels with soft linings

Design flow velocities in channels with soft linings should be limited to the maximum permissible flow velocity for the surface material. Flow velocities in channels may be reduced by either:

- increasing the channel roughness (e.g. through appropriate plant selection)
- reducing flow depth (e.g. wide, shallow channels)
- reducing channel slope (e.g. increase channel length, or use of grade control structures).

Numerous design and operational problems exist when grade control (drop) structures are incorporated into vegetated channels; therefore, the need for drop structures should be avoided or at least minimised.

9.5.2 Recommended maximum average flow velocities

The permissible velocity for flow passing through heavy vegetation (i.e. non-grassed floodways) is not well documented. It is recognised that the permissible design velocity for vegetated channels should allow for some degree of 'natural' vegetation damage, as long as sufficient recovery time exists between such damaging flows. In the absence of local guidelines, permissible velocities for flow passing through heavy vegetation may be determined from Table 9.5.1.

Table 9.5.1 – Suggested permissible flow velocities for water passing through/over vegetation [1]

Manning's roughness of vegetated segment	Suggested permissible average flow velocity during 2% AEP flow [2]
n = 0.03	Refer to Table 9.5.3
n = 0.06	1.7 m/s
n = 0.09	1.5 m/s
n = 0.15	1.0 m/s

Notes (Table 9.5.1):

[1]   Sourced from Brisbane City Council (2000a).

[2]   Permissible flow velocity may need to be reduced if soils are considered highly erosive.

For channels with composite shape or composite surface cover, the average flow velocity for each channel segment should be determined and compared with the permissible values.

Recommended permissible design flow velocities for consolidated bare earth and grassed channels are presented in DNRM (2004) and IECA (2008) and reproduced below in Table 9.5.2.

Table 9.5.2 – Maximum permissible velocities for consolidated bare earth channels and grassed channels [1]

Channel gradient (%)	Percentage of stable vegetal cover [2]			
	0 [3]	50	70	100
Erosion resistant soils				
1	0.7	1.6	2.1	2.8
2	0.6	1.4	1.8	2.5
3	0.5	1.3	1.7	2.4
4		1.3	1.6	2.3
5		1.2	1.6	2.2
6			1.5	2.1
8			1.5	2.0
10			1.4	1.9
15			1.3	1.8
20			1.3	1.7
Easily eroded soils				
1	0.5	1.2	1.5	2.1
2	0.5	1.1	1.4	1.9
3	0.4	1.0	1.3	1.8
4		1.0	1.2	1.7
5		0.9	1.2	1.6
6			1.1	1.6
8			1.1	1.5
10			1.1	1.5
15			1.0	1.4
20			0.9	1.3

Notes (Table 9.5.2):

[1]   Adapted from DNRM (2004).

[2]   Designers should assess the percentage of stable vegetal cover likely to persist under design flow conditions. However it should be assumed that under average conditions the following species are not likely to provide more than the percentage of stable vegetal cover indicated:

- Kikuyu, Pangola and well maintained Couch species – 100%
- Rhodes Grass, poorly maintained Couch species – 70%
- Native species, tussock grasses – 50%

[2]   Applies to surface consolidated, but not cultivated.

9.5.3   Recommended maximum channel side slopes

Guidelines on recommended maximum bank slopes are provided in Table 9.5.3. Maximum bank slopes for grass-lined channels should preferably be 1 in 6 (1V:6H), with an absolute maximum of 1 in 4 (1V:4H).

Table 9.5.3 – Suggested maximum bank gradient [1]

(V:H)	Bank description
1:1	• Earth banks stabilised with selectively placed rock (boulders)
1:2	• Good, erosion resistant clay or clay-loam soils with a healthy, deep-rooted bank vegetation formed by suitable riparian groundcover species, shrubs and trees • Earth banks protected with dumped rock
1:3	• Sandy-loam soil with groundcover vegetation or a closed canopy channel with shaded banks and sparse bank vegetation
1:4	• Grassed or vegetated banks on sandy soils • Maximum gradient for mowable grass banks (refer to local government)
1:6	• Desirable gradient for mowable grassed banks

Note (Table 9.5.3):

[1]   Use as a general guide only—final bank slope should be based on bank stability and revegetation requirements.

9.5.4   Treatment of channel inverts

Soft lined channels may need to be provided with a low-flow channel that may be narrower than the overall channel bed. The purpose of the low-flow channel is to allow for free drainage of the channel invert, thereby minimising the occurrence of waterlogging that may present a maintenance and/or mosquito breeding problem.

The low-flow channel may be either centrally located, offset to one side of the channel centreline, or may meander within the channel. In natural channels the typical sinuosity of these low-flow channels is around 1.02 to 1.08 (Brisbane City Council, 2000a).

Designers should consult the local authority to ascertain the design requirements for low-flow drainage, and should also refer to the recommendations presented in section 9.8.

9.5.5   Tidal channels

Where grassed channels are exposed to salt water environs, such as channels discharging to estuaries or the ocean, then the chosen grass lining should be of a salt resistant variety, otherwise a hard facing should be adopted as the channel lining.

Mangrove infestation of such channels is common and can result in high ongoing maintenance costs. If mangrove growth is expected along a constructed channel, and such growth could adversely affect the hydraulic capacity of the channel, then consideration should be given to the inclusion of an elevated grass-lined bypass channel as shown in Figure 9.10.
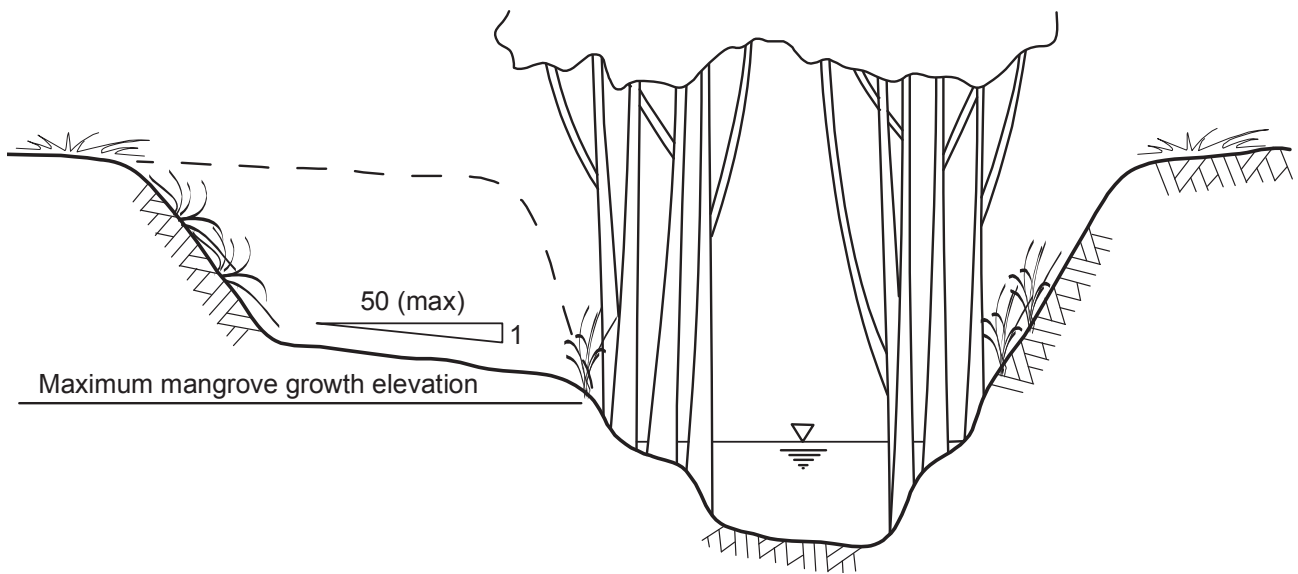
Figure 9.10 – Introduction of salt-grass bypass channel to minimise the hydraulic impact of mangroves

9.6　Natural channel design

Natural Channel Design (NCD) is a waterway design concept based on the planning, design, construction and maintenance of a waterway channel that is compatible with current and future hydrologic, ecological and social requirements for the catchment. Urban drainage channels that are designed using the principles of NCD may also be referred to as 'vegetated channels'.

Guidelines on Natural Channel Design may be found in Brisbane City Council (2000a). Designers are cautioned in the use of some 'River Morphology' guidelines when designing minor urban drainage systems. Even though constructed drainage channels, creeks and rivers all obey the same basic laws of physics, there are subtle differences that exist in the design of these systems and it is the responsibility of the designer to be aware of these differences.

Urban waterways can operate very differently from natural waterways even if the urban waterway is a remnant natural system. When designing new drainage channels, or rehabilitating existing channels, it is important for the designer to appreciate the potential functions and differences of the 'urban' versus 'natural' waterway as summarised in Table 9.6.1.

Vegetated channels designed using the principles of NCD are normally incorporated into urban drainage systems in the following circumstances:

• drainage channels that form part of a wildlife corridor

• when it is desirable to rehabilitate a constructed drainage channel, or a heavily modified natural channel

• when it is desirable to introduce natural features into the design of an urban waterway.

Constructed waterways within modified catchments (rural, urban, industrial and commercial) should incorporate the principles of Natural Channel Design wherever practical. As a general guide, within South East Queensland a minimum of 30 ha urban and 50 ha bushland is required to provide sufficient runoff to maintain desirable low-flow water quality within minor permanent pools (Brisbane City Council, 2000a). Outside South East Queensland it will be necessary to base design information on local data.

Channel designs should function as self-sustaining ecosystems, and their design should aim to minimise construction impacts and long-term maintenance requirements. The planning of urban waterways must consider their potential interactions with adjacent ecosystems. These interactions may be important on a sub-regional basis and may not always correspond to water catchment boundaries.

The long-term viability of vegetated channels depends on the control of in-channel vegetation growth (including weed control and aquatic plants) and on the control of sediment inflow. Therefore, designers should ensure the following:

- permanent sediment controls exist upstream of Natural Channel Design systems if sediment inflow is expected to be significant
- adequate floodway width is provided such that a shade-producing riparian zone can be developed each side of the channel without adversely affecting property flooding.

The adoption of NCD features into an urban drainage channel **must** be preceded by the adoption and appropriate enforcement of 'erosion and sediment control' (ESC) principles throughout the catchment. Without effective sediment control measures, the on-going maintenance requirements (i.e. de-silting) of the drainage channel will be incompatible with the vegetative features of a natural channel design.

It is noted that following initial construction, vegetated channels normally experience a settling-in period (typically 2 to 5 years) during which time the risks of hydraulic failure or significant channel erosion are significantly higher than when the vegetation is fully established. Proponents of NCD should acknowledge these erosion risks as an integral part of this design philosophy.

Tasks to be completed during the investigation and planning of a Natural Channel Design include:

- identifying State and local government requirements
- identifying community and local government's expectations
- identifying local issues and concerns, such as fauna requirements, flood risk, weed control and mosquito control
- identifying site constraints, such as floodway width restrictions, location of services, bed rock and valued stands of vegetation
- identifying the key geomorphological characteristics of the catchment including base flow rate, bed form (i.e. either a clay-based, sand-based, or gravel-based system) and soil type (e.g. erosion-resistant soils, dispersive soils).

Table 9.6.1 – Operational differences between 'natural' and 'urban' waterways

Function	Natural waterways in 'natural' catchments	Vegetated waterways in 'urban' catchments
Treatment of stormwater runoff	• In most natural catchments, overbank runoff contains few pollutants except possibly organic matter and dissolved animal faeces • Most organic matter is filtered from the runoff as the stormwater passes as *sheet flow* through the riparian zone • The major pollutant passing down the waterway is likely to be sediment originating from natural channel erosion and channel migration	• Urban runoff contains a wide variety of pollutants • Most stormwater runoff enters streams via stormwater pipes or open channel drainage • Very little water enters urban waterways as *sheet flow* and thus riparian zones are unable to provide their normal function of *filtration* • Urban stormwater should be pre-treated (filtered) prior to entering waterways

Aquatic passage	• Aquatic passage is required in natural streams for:
	  (i) migration
	  (ii) breeding
	  (iii) allowing aquatic life to return to upstream habitats following periods of high-velocity flows
		• Aquatic passage is needed for the same reasons as in natural streams
		
		• Aquatic passage is needed to help control mosquito breeding
		
		• There is usually increased downstream displacement of aquatic life by increased stream flows

Terrestrial passage	• A natural waterway may play only a supplementary role in the provision of terrestrial passage throughout the overall catchment	• In urbanised catchments, the waterways are likely to act as the major terrestrial corridors as well as the primary habitat
Riparian vegetation	• Due to the high filtration of stormwater flows entering natural waterways, leaf fall from overhanging riparian vegetation can become an important food source for aquatic life	• Due to the very high concentrations of organic matter entering urban waterways, most urban waterways are eutrophic, and thus aquatic life does not depend on riparian leaf fall
Snag management	• In-stream snags are needed to collect and retain floating organic matter, thus allowing it to decay and provide an essential food source	• In-stream snags generally do not need to be designed to trap and retain organic matter because urban streams usually experience excessive inflow of organic matter • Snags may still be needed to provide resting sites for amphibious and terrestrial wildlife

9.7   Other design considerations (all channels)

The following issues need to be given appropriate consideration in the design of all constructed channels; however, not all issues will be applicable for all constructed channels.

9.7.1   Safety issues

Urban waterways and stormwater drainage systems can represent a significant safety risk during storms and times of flood. These risks may be associated with a person deliberately entering a drain or waterway, or as a result of an accidental slip or fall. Discussion of safety issues is provided in Chapter 12 – *Safety issues*.

9.7.2   Access and maintenance berms

It is recommended that the overall easement/reserve width for an open channel should allow for an access/maintenance berm of at least 4.5 m width on at least one side of the main channel.

Maintenance berms may be located within the main channel (not desirable in Natural Channel Design) if it is necessary to provide access for mowing or debris removal, or if it is important to obtain maximum hydraulic efficiency within a specified easement width. Some authorities may require in-channel maintenance berms to be benched into the channel bank at an elevation above the 63% AEP (1 in 1 year) flow. In any case, the primary objective must be to provide suitable access to all areas requiring regular maintenance.

Where access and maintenance cannot be achieved for the whole channel from one side, it may be necessary to provide an access/maintenance berm on both sides of the channel. Notwithstanding the above provisions, a 1.5 m wide safety/access strip should be provided along any side of the channel not provided with a access/maintenance berm.

Designers should consult with the relevant local authority regarding the provision and location of access/maintenance berms and safety berms, to ensure all requirements are satisfied.

9.7.3   Fish passage

Fish passage requirements are generally only an issue for consideration in the following circumstances:

* Fish habitats identified by Department of Agriculture, Fisheries and Forestry (DAFF), the local government, or within a Stormwater/Catchment Management Plan. Designers should refer to the waterway maps provided on Queensland Fisheries web site.

* Natural waterways containing permanent or near-permanent water, either pooled or flowing.

* Constructed channels developed using the principles of Natural Channel Design—fish passage normally being required to control mosquito breeding within the low-flow channel.

* Any waterway, watercourse, or constructed drainage channel containing desirable aquatic life that requires suitable aquatic passage conditions to ensure its sustainability.

Common components of aquatic habitats and passages include:
* continuity of aquatic corridor
* low-flow channels through culverts
* pool and riffle systems (within medium-gradient alluvial streams)
* permanent water habitat pools
* shading of water's edge and habitat pools
* aquatic plants and lower-bank, water's edge plants

- coarse bed substrate consistent with local, natural waterway
- habitat diversity
- backwater areas and shelter from high-velocity flood flows
- in-stream logs and boulders
- low flows of high water quality.

Table 9.7.1 provides recommendations on the preferred crossing type for waterway crossings over various fish habitats. In general the preference would be a bridge (preferred), arch, culvert, ford and finally a causeway as the least preferred option.

Table 9.7.1 – Recommended waterway crossings of fish habitats [1]

Characteristics of watercourse type	Recommended crossing type
Major fish habitat: Permanently flowing river or named permanent or intermittent flowing stream, creek or watercourse containing threatened fish species.	Bridge, arch structure or tunnel
Moderate fish habitat: Named permanent or intermittent stream, creek or watercourse with clearly defined bed and banks with semi-permanent to permanent waters in pools or in connected wetland areas. Marine or freshwater aquatic vegetation is present. Known fish habitat and/or fish observed inhabiting the area.	Bridge, arch structure, culvert or ford
Minimal fish habitat: Named or unnamed watercourse with intermittent flow, but has potential refuge, breeding or feeding areas for some aquatic fauna (e.g. fish, yabbies). Semi-permanent pools form within the watercourse or adjacent wetlands after a rain event. Otherwise, any minor watercourse that interconnects with wetlands or recognised aquatic habitats.	Culvert or ford
Unlikely fish habitat: Named or unnamed watercourse with intermittent flow following rain events only, little or no defined drainage channel, little or no flow or free standing water or pools after rain events (e.g. dry gullies or shallow floodplain depressions with no permanent wetland aquatic flora present). No aquatic vegetation present within the channel.	Culvert, causeway or ford

Note (Table 9.7.1):

[1]   Developed from Fairfull and Witheridge (2003).

Grade control structures (e.g. riffles, chutes and rock weirs) should be limited to a maximum overall fall height of 0.5 m wherever practical; otherwise the structure may need to incorporate a separate fishway. It is noted that traditional open slot fish ladders should **not** be incorporated into grade control structures, chutes or culverts due to their inability to manage bed load sediments.

Any works that could potentially interfere with the passage of fish will require approval from Queensland Fisheries (DAFF). Guidelines on the fish passage requirements may be obtained from Fisheries. Guidelines on fish passage at waterway crossings may be obtained from Kapitzke (2010), Witheridge (2002) and Fairfull & Witheridge (2003). Revised guidelines for the design of culvert fishways are currently (2013) in production through Ocean Watch (Bundaberg).

9.7.4   Terrestrial passage

In urban as well as rural residential areas, terrestrial wildlife corridors are often limited to the locations of waterway corridors; however, important terrestrial habitats also exist within hilltop bushland reserves, often located well away from a waterway. Urban drainage corridors can often be used as wildlife corridors linking bushland reserves with waterways. The development of a council-wide Wildlife Corridor Map (refer to section 2.9.2) can provide a valuable planning tool for urban development and the design of drainage corridors.

Common components of terrestrial waterway habitats and movement corridors include:

- habitat areas
- resting/roosting logs and boulders
- movement links between habitat areas
- continuity of corridor along one or both sides of the channel
- ability to cross the channel (dry crossings, log bridges, pipe crossings)
- open floodways to separate wildlife habitats from residential areas
- isolation from human movement corridors
- 'dry' passageways under bridges and culverts.

Wherever practical, open grassed floodways should be used to separate residential, industrial, and commercial properties from riparian vegetation for bushfire control, the protection of wildlife habitats, and limiting the movement of undesirable wildlife into residential properties.

Guidelines on the integration of terrestrial passage into waterway crossings may be obtained from the Department of Main Roads (2000b).

9.7.5   Connectivity

Typically, the incorporation of natural features into the design of open channels should not aim to produce isolated areas of 'nature' in the middle of urban landscapes. The integrity of natural areas and their connectivity is integral to the ecological value and sustainability of these areas.

The linear nature of aquatic ecosystems must recognise the need for continuous and functional corridors to support the natural processes associated with flora and fauna (both aquatic and terrestrial) and sediment transport.

9.7.6   Human movement corridors

Where practical, human movement corridors (e.g. footpaths and bikeways) should be limited to floodplains, floodways and other overbank areas **outside** the primary riparian vegetation zone. Channel access points may be desirable at regular intervals along vegetated channels; however, the development of in-channel pathways can significantly affect the daily movement, feeding and activities of wildlife.

The construction of boardwalks along waterways is **strongly** discouraged. Though highly successful when placed in low velocity environments such as wetlands, waterway boardwalks commonly fail for the following reasons:

- The shading caused by the boardwalk suppresses vegetation under the boardwalk. This can result in severe gully erosion as high velocity flows find it easier to pass under the boardwalk than pass through adjacent vegetation.
- Turbulence resulting from high velocity flows passing around the support posts can initiate erosion under the boardwalk.

9.7.7　Open channel drop structures

The adaptation of standard spillway-type energy dissipaters (Hager 1992, Peterka 1984, and USBR 1987) to open channel drop structures can result in numerous design problems. The most common problems are associated with safety issues and scour control downstream of non-rectangular drop structures.

To avoid the three-dimensional flow problems commonly associated with trapezoidal drop structures, it is recommended that only drop structures of rectangular cross-sections should be used unless the design is supported by physical modelling. It is noted, however, that these three-dimensional flow problems can also occur when a rectangular drop structure is located within a wide floodway.

The following design requirements are recommended for open channel drop structures:

- Maximum fall of 1.5 m for vertical and near-vertical drops. Preferred fall range is 0.75 to 1.3 m.

- Maximum desirable fall of 0.5 m in locations where fish passage is required; otherwise appropriate fish passage conditions need to be installed into or around the drop structure.

- Appropriate measures are taken to stabilise the hydraulic jump within the structure (i.e. to prevent the hydraulic jump moving away from the structure as tailwater levels fall).

- The drop structure, including the energy dissipation zone, should be free draining if permanent pooling would cause offensive water, a safety risk, or the breeding of mosquito or biting insects.

9.7.8　In-stream lakes and wetlands

Constructed lakes and wetlands can introduce significant fish passage barriers into a waterway. By their nature, lakes and wetlands are water bodies with very low hydraulic gradients. When introduced to a medium or even low gradient waterway, a hydraulic discontinuity is usually established at the lake's inlet and/or outlet, either in the form of a drop structure or weir. Even though lakes and wetlands can represent significant aquatic habitats, designers should minimise the adverse impacts these water bodies have on essential aquatic and terrestrial passage along the waterway.

Guidelines on the preferred edging treatment of lakes and wetlands are provided within the Water-by-Design suite of guidelines. Steeper slopes may be used on those banks lined with suitable woody vegetation, provided a person can readily egress from the water body.

9.7.9　Design and construction through acid sulfate soils

The following discussion relates to open channels, including constructed open drains, trenches (including runnels for mosquito control), swales and canals.

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

In areas that have a high probability of containing ASS, local government planning strategies should, as far as practical, give preference to land uses that avoid or minimise the disturbance of ASS. Land uses such as extractive industries, golf courses, marinas, canal estates, and land uses with car parking or storage areas below ground level which are likely to result in significant amounts of excavation, filling (or even de-watering), should be avoided in high probability areas. However, where the ASS occur at significant depth, the previously mentioned land uses may be appropriate if they are unlikely to result in the disturbance of ASS layers. This issue is further

explained in the State Planning Policy 2/02 *Guideline: Planning and Managing Development involving Acid Sulfate Soils* (due for replacement in 2014)*.*

Alternative uses such as open space or wildlife corridors may be allocated to areas with high sulfide concentration.

It is preferable to maintain groundwater levels in a steady state. Works to be avoided include:

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

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

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

In situations where the ASS investigation has identified high levels of sulfides in the soil, the design of new drainage works must incorporate appropriate management principles, such as (Dear et al. 2002):

1. Disturbance of ASS to be avoided wherever possible.

2. Where disturbance of ASS is unavoidable, preferred management strategies are:
   - minimisation of disturbance
   - neutralisation
   - hydraulic separation of sulfides either on its own or in conjunction with dredging
   - strategic re-burial (reinterment).
   Other management measures may be considered but must not pose unacceptably high risks.

3. Appropriate consideration of alternative development sites, and/or alternative sites to locate drains, roads, pipelines and other underground services.

4. In situations where avoidance of all ASS is not possible, drainage should be designed so areas with the highest levels of sulfide are either not disturbed (preferred), or minimally disturbed (where non-disturbance is not practical), and overall ASS disturbance is minimised.

5. Wherever practical, drains are designed so that they do not penetrate the acid sulfate soil layers, and preferably the acid sulfate soils are at least 0.5 m below the channel invert.

6. Drainage designs must allow construction and ongoing maintenance works to be performed in accordance with best practice environmental management as defined in documents such as the Queensland Acid Sulfate Soil Technical Manual (Dear, et al., 2002), *Environmental Protection Act 1994*, *Fisheries Act 1994*, *Coastal Protection and Management Act 1995*.

7. Neutralising agents may need to be incorporated into the lining of constructed drainage channels to aid the neutralisation of acidic stormwater runoff, and to neutralise acidic water entering from acidified groundwater inflows. It is inappropriate to apply neutralising agents into natural watercourses or water bodies unless carefully planned and approved. This is particularly important for waters where pH-sensitive wildlife may be present such as in naturally acidic coastal wetlands e.g. wallum.

8. Drainage designs should not rely on receiving marine, estuarine, brackish or fresh waters as a primary means of diluting and/or neutralising ASS or associated contaminated waters.

9. Larger drainage works (e.g. greater than 5000 tonnes of soil disturbance) should be staged to ensure that the disturbance is manageable.

In addition, the design of drainage systems in new urban development should give preference to:

- firstly, open channels with inverts at least 0.5 m above ASS layers

- secondly, open channels with inverts above ASS layers

- thirdly, piped drainage systems that discharge directly to open waterways that will allow maximum dilution of acid waters

- fourthly, piped drainage systems that discharge directly to existing open channels or waterways, i.e. do not require the construction of new open channels that may intersect ASS layers or cause a lowering of surrounding groundwater levels.

The application of Water Sensitive Urban Design into potential ASS regions should give preference to systems that:

- maintain natural stormwater infiltration into the soil

- maintain or increase local groundwater levels

- avoid the need for groundwater to be used as a source of non-potable water.

A detailed management plan (Dear, et al., 2002) will be required:

- for disturbances greater than 1000 tonnes

  or

- where the proposed works are likely to alter the groundwater table of the area or where the site is close to an environmentally sensitive area (even if less than 5 tonnes of lime treatment are required).

Environmental Management Plans (EM Plans) may be requested by the local government to support a drainage proposal, or prepared by a proponent who wishes to demonstrate their general environmental duty effectively. As of 2006, there is no statutory mechanism for approval of EM Plans, although they may be given legal standing by incorporation into a development approval through a condition of the approval (e.g. under the *Sustainable Planning Act 2009*).

The Department of Environment and Heritage Protection (EHP) can require an Environmental Management Program (EM Program) to be submitted for assessment under certain circumstances. EM Programs are a statutory tool under Part 3 of the *Environmental Protection Act 1994* and may be approved, approved with conditions, or refused by the EHP.

9.8 Low-flow channels

9.8.1 General

The primary functions of low-flow channels within open channels include:

- allowing the efficient drainage of the greater channel or floodway area to minimise the risk of undesirable waterlogging of the soil

- controlling erosion along the invert of large drainage channels

- providing a hydraulic regime that allows the flushing of regular sediment flows towards specified in-stream sediment traps

- providing necessary ecological features (e.g. habitat and passage) within waterway habitats.

Low-flow channels should be designed such that:

- stormwater does not unnecessarily pond within stormwater outlets

- ephemeral open channels freely drain if stagnant or offensive waters would otherwise occur

- all reasonable efforts are taken to maintain suitable water quality within constructed pools either by:
  - o maintaining suitable low-flow water quality and/or
  - o maintaining desirable aquatic life within pools to prevent the development of offensive waters, and to minimise the development of habitats suitable for the breeding or habitation of 'biting insects'.

9.8.2 Recommended design capacity

The design capacity of the low-flow channel or drainage system will depend on the requirements of the relevant local government.

Whilst no specific recommendation can be made, Table 9.8.1 details practice previously adopted by some local governments for constructed channels that are not intended to incorporate 'natural' features (i.e. not applicable to Natural Channel Design works).

Table 9.8.1 – Low-flow channels within grassed or hard-lined channels

Authority	Design capacity	Minimum channel size
Brisbane City Council	0.01 m^3/s per ha of contributing catchment	Base width = 2.0 m Depth = 0.45 m Side slopes = 1 on 1
Logan City Council [1]	3 mm/hr average rainfall intensity falling on the catchment encompassed by the 30 minute isochrone	Similar to BCC practice and required to contain a 'Bobcat' or similar machine for cleaning and maintenance

Note (Table 9.8.1):

[1]   Based on a procedure proposed by Leighton (1988).

Brisbane City Council (2000a) has published the approximate base flow discharges for natural streams (based on flow rates originally developed for NSW streams). Equation 9.10 has been developed from the design chart presented in the above publication.

$$log_{10}(Q) = 0.886 \, log_{10}(A) + 2.66 \, log_{10}(P_A) - 8.18 \qquad (9.10)$$

where:

Q = Base flow (L/s)

A = Catchment area (ha)

PA = Average annual rainfall depth (mm)

Low-flow channels within Natural Channel Design systems should be based on one of the following conditions, as appropriate for the site:

- continuity of the existing upstream and downstream low-flow channels

- a channel of sufficient capacity to contain the highest observed seasonal (i.e. not average annual) trickle flow plus 150 mm freeboard

- where field observations are not practical, a channel of sufficient capacity to contain the base flow obtained from equation 9.10 plus 150 mm freeboard.

Excluding those regions of a low-flow channel that incorporate a pool–riffle system, a low-flow channel should have a minimum depth of 300 mm.

9.8.3 Design considerations

The recommended configuration of the low-flow drainage system within an open drain (i.e. channel with limited 'natural' features) consists of a concrete-lined open channel section of either trapezoidal or vee shape. The low-flow drainage system could also consist of an underground pipe system connected to regularly-spaced grated inlets. Such underground systems are prone to debris blockage at the inlet grates, and sediment blockage of the underground pipes, as the maximum available invert gradient is generally quite flat, (usually flatter than the open channel). Such underground systems should therefore be graded to be self-cleaning in order to minimise blockages.

Low-flow channels for vegetated open channels that are intended to incorporate 'natural' features must be compatible with the functions and character of the waterway.

9.8.4 Edge protection for low-flow channels

The edges of a concrete lined low-flow channel require special attention. In the majority of cases, local flow velocities within the low-flow section will be substantially higher than corresponding flow velocities within the adjacent soft faced section, because of the different surface roughness. At the interface between the concrete lined low-flow section and the adjacent soft faced section, local flow velocities may be sufficiently high to cause scour.

A row of flexible rock mattresses, protective open mesh geotextile or similar may therefore need to be installed along both sides of the concrete lined low-flow section. In any case, the edge treatment must have adequate hydraulic roughness to produce a local boundary layer equivalent to the adjacent vegetative surface.

9.8.5 Attributes of various low-flow channels

(a) Open earth low-flow channel

Sketch Only
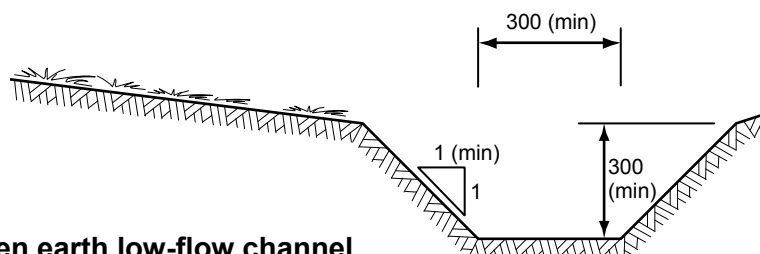
Figure 9.11 – Open earth low-flow channel

Attributes (Figure 9.11):

- Highly susceptible to erosion.

- Most earth channels will eventually become vegetated with grasses, reeds, or weeds unless heavily shaded.

- In many environments these channels are being maintained by annual spraying with herbicides. Such practices are generally not considered to be ecologically sustainable.

- Generally **not** recommended unless there is a demonstrated history of successful application within the proposed channel environment.

(b) Vegetated low-flow channel
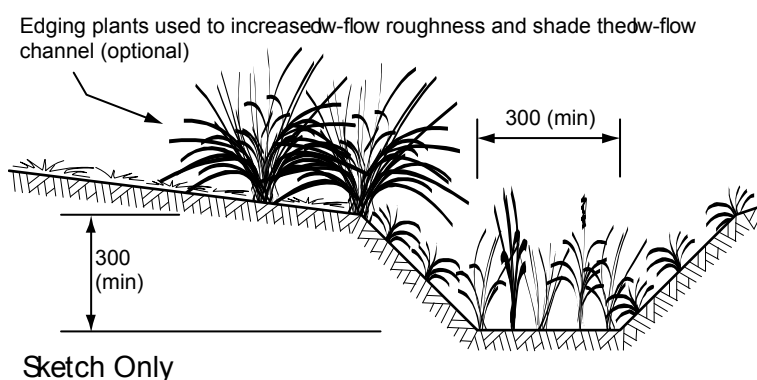
Figure 9.12 – Vegetated low-flow channel

Attributes (Figure 9.12):

- Highly susceptible to erosion along the banks of the low-flow channel if the bed of the channel becomes overgrown with inflexible wetland plants (e.g. reeds).

- These channels generally require medium to heavy shading by riparian vegetation to control weed and reed growth.

- Can be susceptible to mosquito breeding.

- Typical permissible flow velocities in the range of 1.4 to 2.0 m/s for erosive to erosion-resistant soils respectively.

- In many environments these channels are being maintained by annual spraying with herbicides. Such practices are generally not considered to be ecologically sustainable.

- Generally **not** recommended unless there is a demonstrated history of successful application within the proposed channel environment.

(c)   Vegetated rock-lined low-flow channel

Edging plants (e.g. Lomandra) used to provide a mowing edge (optional)

300 (min)

300 (min)

Sketch Only
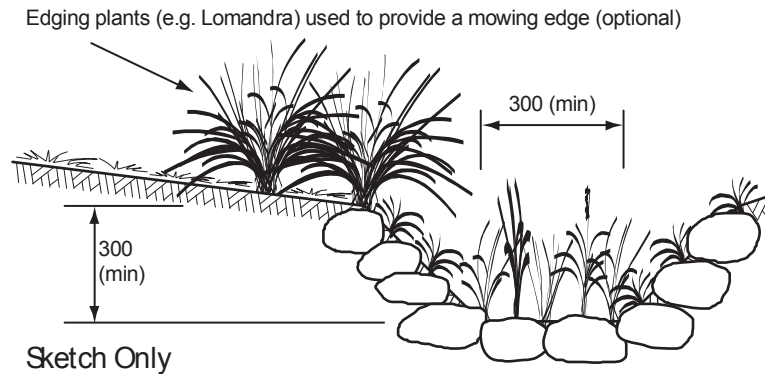
Figure 9.13 – Vegetation rock-lined low-flow channel

Attributes (Figure 9.13):

- Generally a high degree of stability.

- If excessive weed/reed growth occurs on the bed, then the rocks help to prevent bank erosion.

- Very difficult to de-weed without disturbing the rocks.

- Very difficult to de-silt without disturbing the rocks, thus suitable only for channels subject to low sediment flows.

- Can be used in open or closed canopy environments, but best results are achieved with partial shading.

- Typical minimum rock size, d_{50} = 100 mm with good vegetation cover, or 200 mm for poorly vegetated channels. Note; small rocks (say < 100 mm) will generally not be stable without vegetation being established around the rocks, and may also be subject to displacement by children.

(d)   Non-vegetated, rock-lined low-flow channel

Sketch Only

300 (min)

2 (typical)

1

300 (min)

Edging plants to povide a mowing edge and reduce fbw velocities in the low-flow channel (optional)
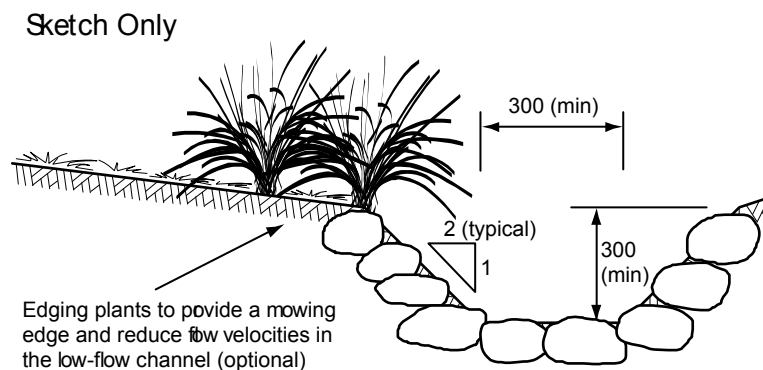
Figure 9.14 – Non-vegetated, rock-lined low-flow channel

Attributes (Figure 9.14):

- Medium to high degree of stability.

- Channels are prone to weed/reed infestation unless heavily shaded by riparian vegetation, otherwise these channels may appear 'poorly maintained'.

- Very difficult to de-silt without disturbing the rocks, thus suitable only for channels subject to low sediment flows.

- Nominal rock size typically based on $d_{50} = 40.V^2$, with minimum rock size being 200 mm and the velocity 'V' being the average flow velocity (m/s) within the low-flow channel.

(e)   Grouted rock low-flow channel
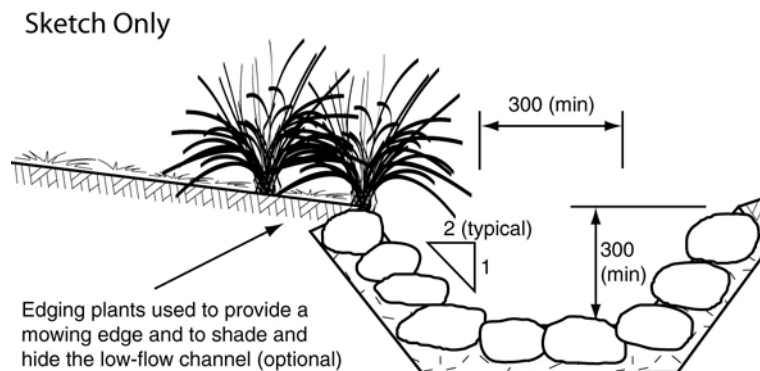
Figure 9.15 – Grouted rock low-flow channel

Attributes (Figure 9.15):
- High degree of stability.
- High velocity flushing can be used to control sedimentation.
- Prone to high water temperatures unless shaded.
- Generally little ecological benefit; however, if large rocks are used, sediment and minor weed growth may establish in the cavities between the rocks resulting in some habitat and low-flow water quality benefits if partially shaded.
- Used in constructed channels where ecological considerations are low.
- Failures (e.g. tunnel erosion) are common when placed over dispersive soils unless the soil is first covered with a layer of non-dispersive soil.

(f)   Pool-riffle system with earth, rock or vegetated low-flow channel
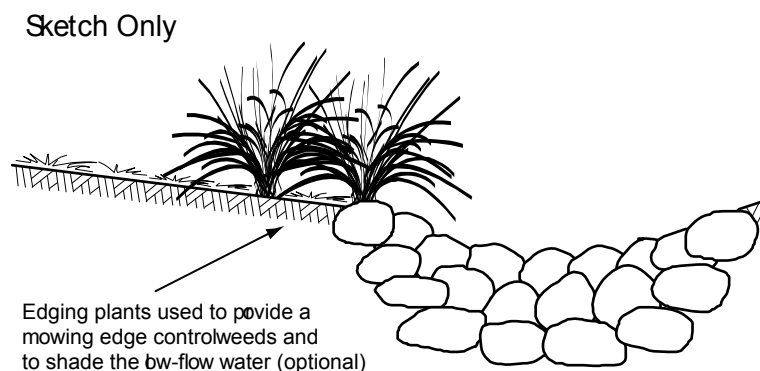
Figure 9.16 – Pool-riffle system within earth, rock or vegetated low-flow channel

Attributes (Figure 9.16):

- Medium to high low-flow water quality benefits.

- Medium to high ecological benefits.

- Most successfully used on mild gradient channels 1 in 50 to 1 in 100 for earth channels, or up to 1 in 20 for fully rock-lined low-flow channels.

- May require catchment areas of at least 30 ha (urban) or 50 ha (bushland) (within South East Queensland) to obtain sufficient dry-weather base flows to maintain good water quality within the pools.

- Highly susceptible to weed/reed invasion if located downstream of wetlands.

- Used in closed canopy or partially shaded environments, and thus are normally associated with Natural Channel Designs, or channels with good riparian cover.

(g)   Gabion or rock mattress low-flow channel

Edging plants to shade channel and provide mowing edge (optional)

Sketch Only
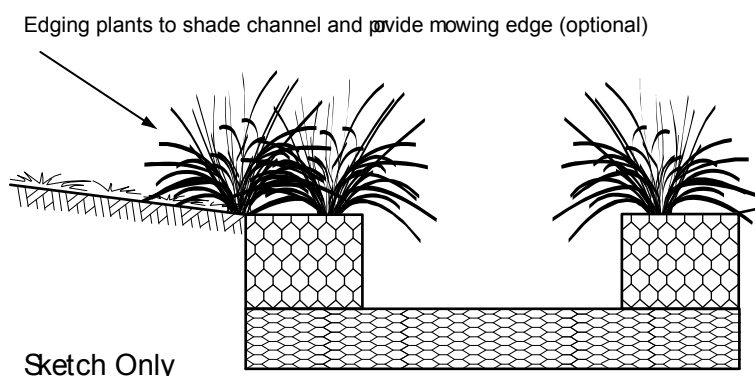
Figure 9.17 – Gabion or rock mattress low-flow channel

Attributes (Figure 9.17):

- In aquatic environments, the wire baskets can have a short to medium design life (even if zinc and plastic coated) unless successfully covered with vegetation.

- Very difficult to de-weed or de-silt.

- Weed or vine invasion is common unless fully vegetated with preferred species at time of construction.

- Channels can look 'weedy' or 'poorly maintained'.

- Used in open canopy channels where sufficient light exists to establish a vegetative cover over the wire mesh. Not recommended within a fully closed canopy.

- Generally **not** recommended due to high construction cost and high maintenance requirements. Should not be used in areas of high sediment flow.

(h)   Concrete low-flow channel

Scour control required in the form of a rough surface typically a strong, thick groundcover (e.g. Lomandra) or grass reinforced with rock or synthetics

Sketch Only

1000 (min)   1000 (min)

50 (typical)

Subsoil drainage (option A)

300 (min)

No-fines concrete block to drain soil (option B)

100 (min)
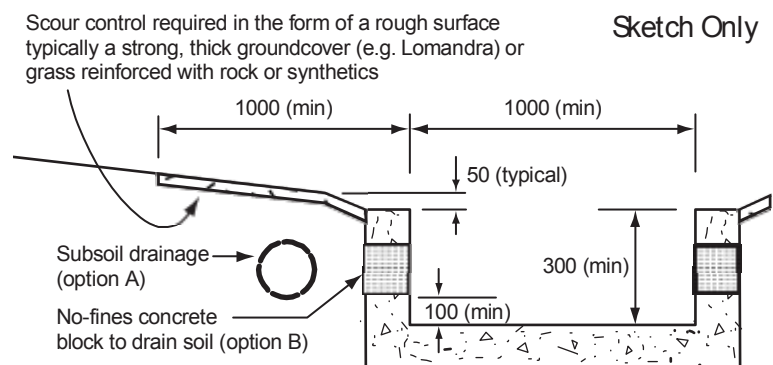
Figure 9.18 – Concrete low-flow channel

Attributes (Figure 9.18):

- Relatively easy to maintain and de-silt.
- Very poor water quality attributes including high temperatures.
- Very poor ecological attributes.
- Some degree of subsoil drainage is required each side of the concrete channel.
- Erosion problems typically occur immediately adjacent the smooth concrete surface unless protected by rock, reinforced grass or similar.
- Typically used on catchment areas less than 30 ha, but can cause water quality problems (e.g. high low-flow water temperatures) adversely affecting downstream waterways.

(i)   Grass swale

Sketch Only

450 (min)

Soakage pit and low-flow pipe system only required if adequate subsoil drainage does not exist

100   Sandy loam

Heavy to medium clay soil   Sand   800 (min)

Gravel

Low-flow pipe   Subsoil drainage pipe
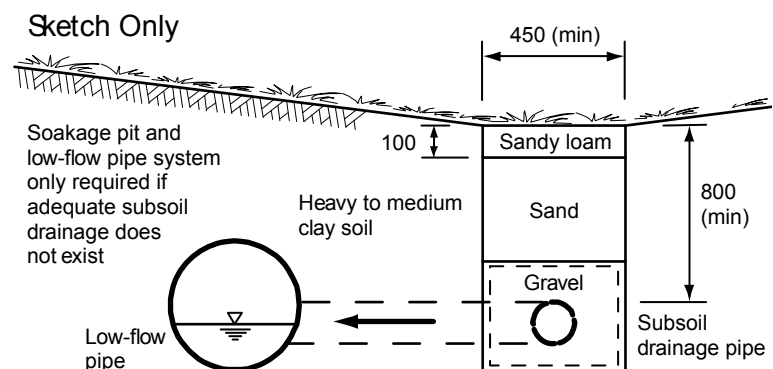
Figure 9.19 – Grass swale (typical system for heavy to medium clayey soils shown)

Attributes (Figure 9.19):

- High low-flow water quality if all low-flows are filtered by the subsoil drainage systems before entering the low-flow pipe; otherwise poor water quality attributes if water is allowed to flow directly into the low-flow pipe.
- The low-flow pipe must be able to freely discharge into a stormwater pipe or open channel.
- Easy to maintain (mow) grass swales if an effective and sustainable subsoil drainage system is established along the swale invert.
- Few ecological benefits other than the water quality benefits.
- Ideally used on grassed swales with gradients of 2 to 4%. Stormwater treatment benefits begin to decrease on gradients steeper than 4%.
- Typical 2% AEP design flow velocity is 1.4 m/s, 1.8 m/s and 2.0 m/s for high, moderate and low-erosive soils respectively.

9.9   Use of rock in drainage channels

9.9.1   General

The following design guidelines refer to the sizing rock used in the construction of:

- rock stabilised drainage channels (section 9.9.2)
- rock-lined batter chutes (section 9.9.3)
- bank stabilisation within waterway channels (section 9.9.4)
- waterway riffles and fishway ramps (section 9.9.5)
- watercourse and gully chutes (section 9.9.5)
- small dam spillways (section 9.9.5)
- culvert and stormwater outlet structures (section 9.9.6)
- scour protection immediately downstream of energy dissipaters (section 9.9.7).

A detailed discussion on the development of the following rock sizing equations is provided in Witheridge (2012a) and the various associated fact sheets. Guidance on the determination of the effective Manning's roughness coefficient (n) for non-vegetated rock-lined surfaces can be found in section 9.3.5 of this Manual.

The following rock sizing equations are based on 'loose rock' placement. It is known that vegetated rock surfaces have significantly enhanced hydraulic stability compared to loose (open void) rock; however, this vegetation can either increase or decrease the effective channel roughness, thus affecting the channel flow velocity. Unfortunately, minimal research currently exists on the sizing of rock for use in vegetated channels.

Crushed (angular) rock is generally more stable than natural rounded rock. A 36% increase in rock size is recommended for rounded rock. This size correction is exercised through the use of the K_1 factor.

The rock should be durable and resistant to weathering, and should be proportioned so that neither the breadth nor the thickness of a single rock is less than one-third its length. The maximum rock size (say d_{90}) generally should not exceed twice the nominal (mean) rock size (d_{50}), but in some cases a maximum rock size of 1.5 times the average rock size may be specified.

Typically the thickness of the rock layer should be sufficient to allow at least two overlapping layers of the nominal rock size. A single layer of rock may, however, be appropriate if vegetation is established on the rock. In order to allow at least two layers of rock, the minimum thickness of rock protection (T) can be approximated by the values presented in Table 9.9.1.

Table 9.9.1 – Typical thickness (T) of two rock layers

Thickness (T)	Size distribution (d_{50}/d_{90})	Description
1.4 d_{50}	1.0	Highly uniform rock size
1.6 d_{50}	0.8	Typical upper limit of quarry rock
1.8 d_{50}	0.67	Recommended lower limit of distribution
2.1 d_{50}	0.5	Typical lower limit of quarry rock

The rock must be placed over a layer of suitably graded filter rock or geotextile filter cloth (minimum 'bidim A24' or the equivalent). The geotextile filter cloth must have sufficient strength and must be suitably overlapped to withstand the initial placement of the rock.

A subgrade filter is generally **not** required if the voids are filled with soil and the rock-lined surface is vegetated. Vegetating rock-lined drains can significantly increase the stability of the rock; however, it can also reduce the drain's hydraulic capacity. Designers should seek 'local' expert advice before establishing vegetation within rock-lined drainage structures

If the rock is placed on a dispersive (e.g. sodic) soil, then prior to placing the filter cloth, the exposed bank **must** first be covered with a layer of non-dispersive soil, typically minimum 200 mm thickness, but preferably 300 mm.

The symbols used in the following design equations are defined below.

A	=	equation constant that typically varies with the degree of turbulence, rock texture (e.g. rounded or fractured) and the required factor of safety
B	=	constant that typically varies with the degree of turbulence
C_u	=	coefficient of uniformity in relation to the distribution of rock size = d_{60}/d_{10}
d	=	nominal rock diameter (m)
d_x	=	nominal rock size (diameter) of which X% of the rocks are smaller (m)
g	=	acceleration due to gravity (m/s^2)
K	=	equation constant based on flow conditions (degree of local turbulence) refer to Table 9.9.3
K_1	=	correction factor for rock shape = 1.0 for angular (fractured) rock, 1.36 for rounded rock (i.e. smooth, spherical rock)
K_2	=	correction factor for rock grading
q	=	flow per unit width down the embankment (m^3/s/m)
s_r	=	specific gravity of rock (e.g. sandstone 2.1–2.4; granite 2.5–3.1, typically 2.6; limestone 2.6; basalt 2.7–3.2)
S	=	slope (m/m)
S_e	=	slope of energy line (m/m)
S_o	=	bed slope (m/m)
SF	=	factor of safety as applied to rock size *'d'*
T	=	Total thickness of rock lining (m)
V	=	design flow velocity (m/s)
V_{avg}	=	the average channel velocity (m/s)
V_o	=	depth-average flow velocity based on **uniform** flow down a slope, S_o (m/s)
y	=	depth of flow to bed at a given location (m)

9.9.2   Rock sizing for the lining of drainage channels

Table 9.9.2 provides the recommended design equation for sizing rock used in the lining of drainage channels. This equation represents a modification of the equation originally presented by Isbash (1936).

A 36% increase in rock size is recommended for **'rounded rock'** (i.e. K_1 = 1.36).

Table 9.9.2 – Recommended rock sizing equation for non-vegetated rock-lined drains

Bed slope (%)	Rock sizing equation
Velocity-based equation for low-gradient drainage channels S_o < 5%	Low gradient, uniform and non-uniform flow conditions $$d_{50} = \frac{K_1 . V^2}{2.g.K^2(s_r - 1)} \qquad (9.11)$$ K = 1.1 for low-turbulent deepwater flow, 1.0 for low-turbulent shallow water flow, and 0.86 for highly turbulent flow (also see Table 9.9.3)

Table 9.9.3 provides recommended 'K-values' for use in equations 9.11. Table 9.9.3 indicates that as the channel slope increases and the flow becomes more turbulent, the required K-value decreases, which is consistent with the recommendations of Isbash (1936).

Table 9.9.3 – Recommended K-values for use in rock sizing equations [1]

Bed slope (%)	< 1.0	2.0	3.0	4.0	5.0	6.0	8.0	> 10.0
K =	1.09	1.01	0.96	0.92	0.89	0.86	0.83	0.80
Flow conditions	Low turbulence					Highly turbulent (whitewater)		

Note (Table 9.9.3):

[1]   Tabulated results are applicable to uniform flow conditions and Manning's n based on equation 9.2.

Equation 9.11 reduces to the commonly used equation 9.12 for 'angular' rock, which is appropriate for rock with a specific gravity, s_r = 2.6.

$$d_{50} = 0.04 \ V^2 \qquad (9.12)$$

9.9.3   Rock sizing for the lining of batter chutes

Batter chutes are used to provide drainage down steep embankments. The critical design components of a batter chute are: (1) flow entry into the chute, (2) the maximum allowable flow velocity down the face of the chute, and (3) the dissipation of energy at the base of the chute.

Table 9.9.4 provides the recommended design equations for sizing rock placed on batter chutes. Table 9.9.5 provides guidance on the selection of an appropriate safety factor (SF) for batter chutes.

Table 9.9.4 – Recommended rock sizing equations for rock-lined batter chutes

Batter slope (%)	Rock sizing equations	
Preferred equation: $S_o < 50\%$	Wide, uniform flow conditions, $S_e = S_o$ $$d_{50} = \frac{1.27 . SF . K_1 . K_2 . S_o^{0.5} . q^{0.5} . y^{0.25}}{(s_r - 1)}$$	(9.13)
Simplified, velocity-based equation: $S_o < 33\%$	Uniform flow conditions, $S_e = S_o$ $$d_{50} = \frac{SF . K_1 . K_2 . V^2}{(A - B . \ln(S_o)) . (s_r - 1)}$$ For $SF = 1.2$: $A = 3.95$, $B = 4.97$ For $SF = 1.5$: $A = 2.44$, $B = 4.60$	(9.14)

Table 9.9.5 – Recommended safety factor for use in determining rock size

Safety factor (*SF*)	Recommended site conditions
1.2	• Low-risk structures (most batter chutes) • Failure of structure is most unlikely to cause loss of life or irreversible property damage • Permanent batter chutes with all voids filled with soil and pocket planted
1.5	• High-risk structures • Failure of structure may cause loss of life or irreversible property damage • Temporary structures that have a high risk of experiencing the design discharge while the voids remain open

9.9.4   Rock sizing for the stabilisation of channel banks

Rock is often used in the stabilisation of constructed drainage channels as well as natural waterways subject to the erosive effects of urbanisation. The critical design parameter is the rock size (d_{50}). Secondary design parameters are the rock type (relating to density, s_r), colour, durability, shape (angular or rounded), and grading (coefficient of uniformity, $C_u = d_{60}/d_{10}$).

Rock size is primarily dependent on the local flow velocity *(V)*, nominated safety factor *(SF)*, rock shape, density and grading; and to a lesser degree, bank slope.

Equation 9.11 can be used to size rock placed on the bed and bank of waterway channels. Bank slope is generally not critical unless the slope is steeper than 1:2 (V:H). The same equation can be used for sizing rock placed on bank slopes of 1:1.5 provided a 25% increase in rock size is specified.

Critical in the sizing of rock in waterway channels is the selection of an appropriate 'design flow velocity' *(V)*. When selecting a design flow velocity the following recommendations should be considered.

- In straight heavily-vegetated watercourses the flow velocity adjacent to the bank may be only a small percentage of the average channel velocity. Such locations generally do not experience severe bank erosion, consequently they are not usually in the need of rock protection.

- In most cases where bank erosion has occurred, there is usually some local condition (e.g. locally induced turbulence or a channel bend) that results in flow velocities adjacent to the bank being equal to, or greater than, the average channel velocity. Thus, 'average flow velocities' as determined by numerical modelling should always be used with caution in sizing rock.

- It should be noted that flood mapping exercises often involve hydraulic modelling based on 'worst case' channel roughness (i.e. full vegetation cover even if such vegetation does not currently exist). Such modelling exercises produce 'worst case' flood levels, but also the lowest channel velocity conditions. Consequently, the flow velocities used in the design of rock protection **may** need to be based on relatively smooth ('As constructed') bank roughness conditions rather than the long-term high roughness conditions.

- For the sizing of rock on the bed of a waterway channel, adopt a design velocity (V) equal to the average channel velocity (V_{ave}).

- In straight sections of a constructed, **uniform** cross-section channel, adopt a design **'bank'** flow velocity equal to the average channel velocity.

- In straight sections of a irregular cross-section, heavily-vegetated watercourse, adopt a design **'bank'** flow velocity 0.67 times the average channel velocity:

$$V = 0.67 \; V_{ave} \tag{9.15}$$

- For impinging bank velocities (e.g. on the outside of a bend) within a meandering channel, adopt a design **'bank'** flow velocity 1.33 times the average channel velocity:

$$V = 1.33 \; V_{ave} \tag{9.16}$$

9.9.5   Rock sizing for the design of constructed waterway riffles

A riffle is an isolated section of channel bed where the steepness of the bed allows for the exposure of the bed rocks and gravels during periods of low flow, often resulting in the formation of whitewater conditions. In pure hydraulic terms, riffles are the same as rock chutes; however, their small size and low gradient means the design procedures used for sizing the rock are normally different from those used in the design of batter chutes and spillways. A rock-lined fish ramp is also a type of rock chute, and in essence, also a riffle.

One of the biggest differences between the rock used in the construction of rock chutes (which are normally used in the stabilisation of gullies) and the rocks used in the construction of fishways and riffles is the required distribution of rock sizes. Rock chutes are normally constructed from rocks of a near uniform size, with special interest being paid to the larger rocks. On the other hand, constructed riffles and other fishways need to be constructed from rocks containing sufficient quantities of small rocks to minimise interflow (flow through the rock voids). This means that special attention must be given to the distribution of rock sizes.

In circumstances where the constructed riffle is required to simulate 'natural' bed conditions, and the riffle is located in a waterway that contains natural pool–riffle systems, then the rocks used in construction of the riffle should match the size distribution of the natural riffle systems. For constructed riffles that are required to be stable during major flows, the following rock specifications should be considered.

Table 9.9.6 provides a suggested distribution rock sizes for 'constructed' riffles.

Table 9.9.6 – Recommended distribution of rock size for constructed riffles

Rock size ratio	Distribution value	Rock size ratio	Distribution value
d_{100}/d_{50}	2.0	d_{40}/d_{50}	0.65
d_{90}/d_{50}	1.8	d_{33}/d_{50} [1]	0.50
d_{75}/d_{50}	1.5	d_{25}/d_{50} [1]	0.45
d_{65}/d_{50}	1.3	d_{10}/d_{50}	0.20

Note (Table 9.9.6):

[1]   Critical for the fish-friendly operation of the riffle.

Riffle rock size is determined as the largest of the following two flow conditions:

- Low-flow condition: This hydraulic check requires the determination of the maximum flow velocity that occurs on the riffle prior to the riffle being drown-out by downstream flow conditions. This usually requires numerical modelling of the stream for a range of flow conditions. Minimum, mean rock size is determined from equation 9.14.

- High-flow conditions: This hydraulic check requires the nomination of the maximum stream flow during which the riffle rock is required to be stable, e.g. the 1 in 10 year or 1 in 50 year discharge. This flow condition is then modelled to determine the maximum depth-average flow velocity passing over the riffle. Minimum, mean rock size for these high flow conditions may be determined from equation 9.11.

In the 'high-flow' case it is important that the depth-average velocity—as determined from the hydraulic modelling—is representative of the actual flow velocities over the riffle, **not** the flow velocity averaged across the full cross-section, which may be influenced by bank roughness or overbank flow conditions.

9.9.6   Rock sizing for the stabilisation of waterway and gully chutes, and minor dam spillways

A waterway chute is a stabilised section of channel bed used to control bed erosion while maintaining desirable fish passage conditions in a manner similar to a natural riffle. These structures may also be referred to as 'grade control structures' or 'rock ramps'.

A gully chute is a steep drainage channel, typically of uniform cross-section, used to stabilise head-cut erosion and/or flow into, or out of, a drainage gully.

Both of these types of structures are highly susceptible to structural failure, either through the direct displacement of rocks by water flow, or the undermining as a result of downstream erosion or flows bypassing either around or under the rocks.

The following design steps have been provided as a general guide only.

- Determine the effective unit discharge *(q)* within the deepest section of the chute crest. If the chute contains a trickle-flow channel, then it will be necessary to determine the effective unit discharge over the deepest section of this channel during the designated design event.

- Based on the unit flow rate *(q)* and the gradient of the chute (S_o), **estimate** the rock size (d_{50}) using an appropriate simplified velocity-based equation. Alternatively, select a trial rock size from tables 9.9.9 or 9.9.10 (for a safety factor of 1.2 and 1.5 respectively).

- Compute water surface profile analysis on the face of the chute and within the energy dissipation basin below. Compute separately the main drop and the drop through the trickle

channel (if any) using a Manning's roughness (equation 9.2) based on the estimated rock size and hydraulic radius. Do not simply assume normal depth will be achieved down the face of the chute. Note; the rock sizes presented within tables 9.9.9 and 9.9.10 assume uniform flow conditions.

- Determine the location of the hydraulic jump. If the tailwater elevation is expected to be greater than the crest elevation, then adopt a conservative design by testing the chute for a 'lower than normal' tailwater condition.

- It should be noted that the critical design parameters used within the rock-sizing equations (i.e. velocity, depth and energy slope) are normally based on the flow conditions just prior to formation of the hydraulic jump.

- Separate numerical analyses of the main channel and trickle flow channel conditions are performed because the hydraulic jump will normally be pushed further downstream in the region of the trickle flow channel. The hydraulic jump in the main channel is usually located either at the toe of the chute or on the face of the chute. However, flow energy from the trickle flow channel can push the jump well into the energy dissipation basin. Scour protection will normally be required for distance of five to six times the tailwater depth downstream of the formation of the hydraulic jump.

- Using equations 9.13 or 9.14 (as appropriate), iterate an acceptable solution for the mean rock size (d_{50}) based on verification of reasonable assumptions for the trial rock size and Manning's roughness. If the chute is partially drowned, then determine rock size based on equation 9.17.

- Design appropriate rock stabilisation upstream of the chute's crest. Typically such protection measures extend upstream of the crest a distance of up to five times the depth of the approaching flow.

Table 9.9.7 – Recommended rock sizing equation for partially drowned waterway chutes

Bed slope (%)	Rock sizing equation	
Partially drowned chutes: $S_o < 50\%$	Steep gradient, non-uniform flow conditions, $S_e \neq S_o$ $$d_{50} = \frac{1.27 . SF . K_1 . K_2 . S_o^{0.5} . V^{2.5} . y^{0.75}}{V_o^{2.0}(s_r - 1)}$$	(9.17)

Chute gradients flatter than 6:1 (H:V) are inherently much more stable, safer and fish-friendly in comparison to steeper bed slopes.

The maximum recommended rock size is d_{50} = 600 mm due to the difficulties of both obtaining larger rock, and placing such rock within natural waterways. It is noted that such large rock will likely prevent the formation of desirable fish passage conditions.

For reasons of stability, the total chute drop height should not be greater than 1.2 m; however, fish passage requirements generally limit the fall of individual drops to no more than 0.5 m.

Tables 9.9.9 and 9.9.10 provide mean rock size (rounded up to the next 0.1 m unit) for 'angular' rock, for a factor of safety of both 1.2 and 1.5. These tables are based on equation 9.13 and are best used in the design of long gully chutes on which uniform flow conditions are achieved. The use of a design equation based on unit flow rate (q) instead of average flow velocity (V) is preferred because it avoids errors associated with the selection of Manning's roughness.

Flow depths provided in tables 9.9.9 and 9.9.10 represent 'average' flow depths. Actual flow depths are expected to be highly variable due to turbulent (whitewater) flow conditions.

Note: Most chutes fail as a result of rock displacement; therefore, it is critical to size the rocks using rock properties (e.g. rock density, size distribution and shape) that are truly representative of the rocks that will actually be used within the structure.

Table 9.9.8 – Recommended safety factor for use in designing waterway and gully chutes

Safety factor (SF)	Recommended usage	Example site conditions
1.2	- Low risk structure - Failure of structure is most unlikely to cause loss of life or irreversible property damage - Permanent rock chutes with all voids filled with soil and pocket planted	- Waterway chutes where failure of the structure is likely to result in easily repairable soil erosion - Waterway chutes that are likely to experience significant sedimentation and vegetation growth before experiencing the high flows - Temporary (< 2 yrs) gully chutes with a design storm of 1 in 10 years of greater
1.5	- High risk structures - Failure of structure may cause loss of life or irreversible property damage - Temporary structures that have a high risk of experiencing the design discharge while the voids remain open (i.e. prior to sediment settling within and stabilising the voids between individual rocks)	- Gully chutes where failure of the structure may cause severe gully erosion - Waterway chutes where failure of the structure may cause severe gully erosion or damage to important infrastructure

Table 9.9.9 – Waterway chutes: uniform flow conditions, s_r = 2.4, d_{50}/d_{90} = 0.5, 'SF = 1.2'

Unit flow rate (m³/s/m)	Bed slope = 1:6		Bed slope = 1:4		Bed slope = 1:3		Bed slope = 1:2	
	y (m)	d_{50} (m)	y (m)	d_{50} (m)	y (m)	d_{50} (m)	y (m)	d_{50} (m)
0.1	0.09	**0.10**	0.09	**0.10**	0.09	**0.20**	0.09	**0.20**
0.2	0.15	**0.20**	0.14	**0.20**	0.14	**0.20**	0.14	**0.30**
0.3	0.20	**0.20**	0.19	**0.20**	0.19	**0.30**	0.18	**0.30**
0.4	0.24	**0.20**	0.23	**0.30**	0.23	**0.30**	0.22	**0.40**
0.5	0.27	**0.30**	0.27	**0.30**	0.26	**0.40**	0.26	**0.40**
0.6	0.31	**0.30**	0.30	**0.40**	0.30	**0.40**	0.29	**0.50**
0.8	0.37	**0.40**	0.37	**0.40**	0.36	**0.50**	0.35	**0.60**
1.0	0.44	**0.40**	0.42	**0.50**	0.42	**0.60**	0.41	**0.70**
1.2	0.49	**0.50**	0.48	**0.50**	0.47	**0.60**	0.46	**0.70**
1.4	0.54	**0.50**	0.53	**0.60**	0.52	**0.70**	0.51	**0.80**
1.6	0.60	**0.50**	0.58	**0.70**	0.57	**0.70**	0.56	**0.90**
1.8	0.64	**0.60**	0.63	**0.70**	0.62	**0.80**	0.60	**1.00**
2.0	0.69	**0.60**	0.67	**0.70**	0.66	**0.90**	0.65	**1.00**
3.0	0.90	**0.80**	0.88	**1.00**	0.87	**1.10**	0.85	**1.30**
4.0	1.10	**1.00**	1.07	**1.20**	1.05	**1.30**	1.02	**1.60**
5.0	1.27	**1.10**	1.24	**1.30**	1.22	**1.50**	1.19	**1.80**
Unit flow rate (m³/s/m)	**Bed slope = 1:30**		**Bed slope = 1:20**		**Bed slope = 1:15**		**Bed slope = 1:10**	
	y (m)	d_{50} (m)	y (m)	d_{50} (m)	y (m)	d_{50} (m)	y (m)	d_{50} (m)
0.1	0.11	**0.05**	0.10	**0.05**	0.10	**0.10**	0.10	**0.10**
0.2	0.17	**0.10**	0.16	**0.10**	0.16	**0.10**	0.15	**0.10**
0.3	0.22	**0.10**	0.21	**0.10**	0.21	**0.20**	0.20	**0.20**
0.4	0.27	**0.10**	0.26	**0.20**	0.25	**0.20**	0.25	**0.20**
0.5	0.31	**0.20**	0.30	**0.20**	0.29	**0.20**	0.28	**0.20**
0.6	0.35	**0.20**	0.34	**0.20**	0.33	**0.20**	0.32	**0.30**
0.8	0.43	**0.20**	0.41	**0.20**	0.40	**0.30**	0.39	**0.30**
1.0	0.50	**0.20**	0.48	**0.30**	0.47	**0.30**	0.45	**0.30**
1.2	0.56	**0.20**	0.54	**0.30**	0.53	**0.30**	0.51	**0.40**
1.4	0.62	**0.30**	0.60	**0.30**	0.58	**0.30**	0.56	**0.40**
1.6	0.68	**0.30**	0.65	**0.30**	0.64	**0.40**	0.62	**0.40**
1.8	0.73	**0.30**	0.71	**0.30**	0.69	**0.40**	0.67	**0.50**
2.0	0.79	**0.30**	0.76	**0.40**	0.74	**0.40**	0.72	**0.50**
3.0	1.03	**0.40**	0.99	**0.50**	0.97	**0.50**	0.94	**0.60**
4.0	1.25	**0.50**	1.20	**0.60**	1.17	**0.60**	1.14	**0.80**
5.0	1.45	**0.50**	1.40	**0.60**	1.36	**0.70**	1.32	**0.90**

Table 9.9.10 – Waterway chutes: uniform flow conditions, s_r = 2.4, d_{50}/d_{90} = 0.5, 'SF = 1.5'

Unit flow rate (m³/s/m)	Bed slope = 1:6		Bed slope = 1:4		Bed slope = 1:3		Bed slope = 1:2	
	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)
0.1	0.10	**0.10**	0.10	**0.20**	0.10	**0.20**	0.10	**0.20**
0.2	0.16	**0.20**	0.16	**0.20**	0.15	**0.30**	0.15	**0.30**
0.3	0.21	**0.30**	0.21	**0.30**	0.20	**0.30**	0.20	**0.40**
0.4	0.26	**0.30**	0.25	**0.40**	0.25	**0.40**	0.24	**0.50**
0.5	0.30	**0.30**	0.29	**0.40**	0.28	**0.50**	0.28	**0.50**
0.6	0.34	**0.40**	0.33	**0.40**	0.32	**0.50**	0.31	**0.60**
0.8	0.41	**0.40**	0.40	**0.50**	0.39	**0.60**	0.38	**0.70**
1.0	0.47	**0.50**	0.46	**0.60**	0.45	**0.70**	0.44	**0.80**
1.2	0.53	**0.60**	0.52	**0.70**	0.51	**0.80**	0.50	**0.90**
1.4	0.59	**0.60**	0.58	**0.80**	0.57	**0.90**	0.55	**1.00**
1.6	0.64	**0.70**	0.63	**0.80**	0.62	**0.90**	0.60	**1.10**
1.8	0.70	**0.70**	0.68	**0.90**	0.67	**1.00**	0.65	**1.20**
2.0	0.75	**0.80**	0.73	**0.90**	0.72	**1.10**	0.70	**1.30**
3.0	0.98	**1.00**	0.96	**1.20**	0.94	**1.40**	0.92	**1.70**
4.0	1.19	**1.20**	1.16	**1.50**	1.14	**1.70**	1.11	**2.00**
5.0	1.38	**1.40**	1.34	**1.70**	1.32	**1.90**	1.29	**2.30**
Unit flow rate (m³/s/m)	Bed slope = 1:30		Bed slope = 1:20		Bed slope = 1:15		Bed slope = 1:10	
	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)	y (m)	d₅₀ (m)
0.1	0.11	**0.05**	0.11	**0.10**	0.11	**0.10**	0.11	**0.10**
0.2	0.18	**0.10**	0.18	**0.10**	0.17	**0.20**	0.17	**0.20**
0.3	0.24	**0.10**	0.23	**0.20**	0.23	**0.20**	0.22	**0.20**
0.4	0.29	**0.20**	0.28	**0.20**	0.27	**0.20**	0.26	**0.20**
0.5	0.34	**0.20**	0.32	**0.20**	0.32	**0.20**	0.31	**0.30**
0.6	0.38	**0.20**	0.37	**0.20**	0.36	**0.30**	0.35	**0.30**
0.8	0.46	**0.20**	0.44	**0.30**	0.43	**0.30**	0.42	**0.40**
1.0	0.53	**0.30**	0.51	**0.30**	0.50	**0.30**	0.49	**0.40**
1.2	0.60	**0.30**	0.58	**0.30**	0.57	**0.40**	0.55	**0.50**
1.4	0.67	**0.30**	0.64	**0.40**	0.63	**0.40**	0.61	**0.50**
1.6	0.73	**0.30**	0.70	**0.40**	0.69	**0.50**	0.67	**0.50**
1.8	0.79	**0.40**	0.76	**0.40**	0.74	**0.50**	0.72	**0.60**
2.0	0.85	**0.40**	0.82	**0.50**	0.80	**0.50**	0.77	**0.60**
3.0	1.11	**0.50**	1.07	**0.60**	1.04	**0.70**	1.01	**0.80**
4.0	1.34	**0.60**	1.30	**0.70**	1.27	**0.80**	1.23	**1.00**
5.0	1.56	**0.70**	1.50	**0.80**	1.47	**0.90**	1.43	**1.10**

9.9.7 Rock sizing for the design of outlet structures

The term 'outlet structure' refers to a wide range of outlet control devices including rock pads, rock mattress aprons, and various impact-type energy dissipaters. Outlet structures are typically used as energy dissipaters on the outlets of stormwater pipes, batter chutes and waterway culverts.

The standard outlet structure consists of a flat pad containing medium-sized rock. The sizing of the rock and the dimensions of the rock pad depend on the hydraulic conditions that exist at the leading edge of the outlet structure. Both rock size and pad dimensions vary depending on whether the outlet consists of a single pipe or multiple pipes/cells.

A rock size and pad length design chart for the sizing of single pipe outlets is provided in Figure 8.13 (Chapter 8 – *Stormwater outlets*).

A rock size and pad length design chart for the sizing of multiple pipe/cell outlets is provided in Figure 10.12 (Chapter 10 – *Waterway crossings*).

9.9.8 Rock sizing for the design of energy dissipaters

Most energy dissipaters contain two zones where rock stabilisation may be used. Zone 1 is the primary energy dissipation zone where turbulence and energy losses are the greatest. Zone 2 is the area immediately downstream of Zone 1 where flows are allowed to return to normal 'uniform' flow conditions prior to entering the receiving channel.

Concrete is more commonly used in the Zone 1, however rock has also been used. In some cases the stability of the rock is increased by filling all voids with grout (e.g. grouted plunge pool energy dissipaters). Rock is most commonly used for scour control within Zone 2.

Equation 9.18 is recommended for the sizing rock placed within the zone of highly turbulent water immediately **downstream** of the end sill of an energy dissipater (i.e. within Zone 2). This equation is based on the recommendations of Bos, Reploge, and Clemmens (1984).

$$d_{40} = 0.038 \, V^{2.26} \tag{9.18}$$

If it is assumed that $d_{40}/d_{50} = 0.75$, then equation 9.18 may be converted to:

$$d_{50} = 0.050 \, V^{2.26} \tag{9.19}$$

If it is assumed that equation 9.18 was based on rock of a specific gravity (s_r) of 2.6, then:

$$d_{50} = \frac{0.08.V^{2.26}}{(s_r - 1)} \tag{9.20}$$

Alternatively, equation 9.21 (based on Isbash, 1936) may be used for sizing riprap **downstream** of stilling basins (i.e. within Zone 2):

$$d_{50} = \frac{K_1.V^2}{14.5(s_r - 1)} \tag{9.21}$$

10. Waterway crossings

10.1 Bridge crossings

10.1.1 General

Bridges are generally the preferred means of crossing an open channel or urban waterway, particularly in the following circumstances:

- where the road elevation is well-above the stream's bed level
- fish passage is required along the waterway for a threatened fish species (refer to Table 9.7.1)
- waterways identified as 'red' zones within Queensland Fisheries (DAFF) mapping

  or

- there is a high degree of environmental sensitivity associated with the waterway and/or its banks.

The design of a bridge is a complex matter requiring input from a multi-disciplinary team including suitably qualified engineers. Design guidelines may be obtained from Department of Main Roads (2000a & 2002) and AustRoads (1994, 2005). Guidelines on scour control around bridge structures are provided in AustRoads (1994, 2005) and Witheridge (2002). The impact of a bridge and its approaches on flood levels in major/extreme events may also need to be assessed through specialist floodplain modelling.

10.1.2 Blockage factors

Blockage considerations for bridges are discussed with AustRoads (1994, 2005) and Engineers Australia (2012). The Engineers Australia report provides a detailed risk assessment process for determining the likelihood of debris blockage and the design consequences. In the absence of a site-specific risk assessment, Table 10.1.1 provides suggested blockage factors for bridges.

Table 10.1.1 – Suggested blockage factors for bridges [1]

Bridge conditions	Blockage factor	
	Design value	Severe storm
Clear opening height < 3 m	[2]	100%
Clear opening height > 3 m	0%	[3]
Central piers	[4]	[4]

Notes (Table 10.1.1):

[1]   Developed from Engineers Australia (2012)

[2]   Typical event blockage depends on risk of debris rafts and large floating debris.

[3]   Blockage considerations are normally managed by assuming 100% blockage of handrails and traffic barriers, plus expected debris matter wrapped around central piers.

[4]   Typical event blockage depends on risk of debris wrapped around central piers. Typically, the larger the piers, the lower the risk normally associated with debris wrapped around piers.

10.1.3 Hydraulics of scupper pipe outflow channels

Roadways represent a major source of stormwater pollution. Stormwater runoff from bridges should be collected and filtered through riparian vegetation and/or other appropriate treatment measures to ensure compliance with water quality objectives before being released into the waterway.

Stormwater runoff from bridges is typically collected by scupper pipes and discharged into a 'side flow channel' or stormwater pipe attached to the bridge deck. Backwater analysis of a side flow channel may be based on the hydraulics of a lateral spillway channel. A lateral spillway channel is an open channel which receives lateral inflow along its length. Benefield et al. (1984) describes the hydraulics of three channel flow conditions as shown in figures 10.1 to 10.3.
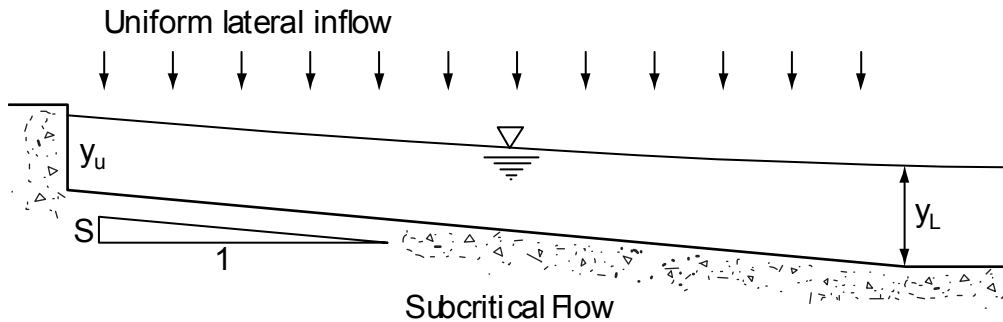
Figure 10.1 – Subcritical flow with subcritical tailwater

For a rectangular channel with downstream subcritical flow depth (y_L), the upstream water depth (y_u) may be estimated using equation 10.1.

$$y_u = \left[\frac{2(y_c)^3}{y_L} + \left(y_L - \frac{S.L}{3} \right)^2 \right]^{0.5} - \frac{2.S.L}{3} \qquad (10.1)$$

where:

y_u = upstream water depth (m)
y_c = effective critical water depth at end of channel where total flow, $Q = q.L$
Q = total flow rate (m³/s)
q = lateral inflow rate per unit length (m³/s/m)
L = length of channel over which lateral inflow occurs (m)
y_L = flow depth at downstream end (m)
S = channel slope (m/m)
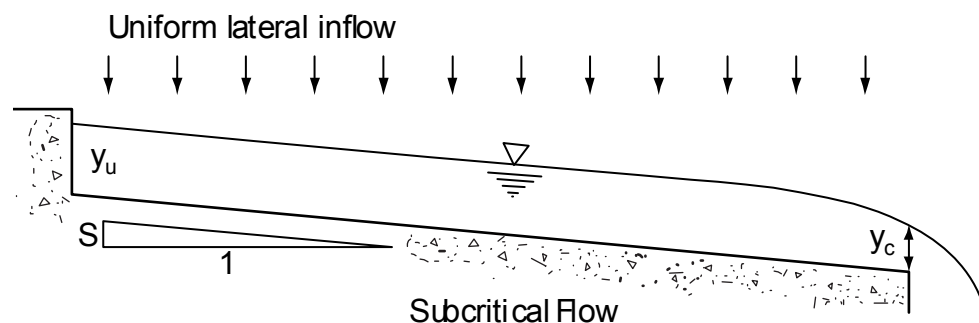
Figure 10.2 – Subcritical flow with critical depth at tailwater

For a rectangular channel with critical depth (y_c) at the downstream end of lateral inflow, the upstream water depth (y_u) may be estimated using equation 10.2.

$$y_u = \left[2(y_c)^2 + \left(y_c - \frac{S.L}{3} \right)^2 \right]^{0.5} - \frac{2.S.L}{3} \qquad (10.2)$$

For zero channel slope ($S = 0$): $y_u = 1.73\ y_c$
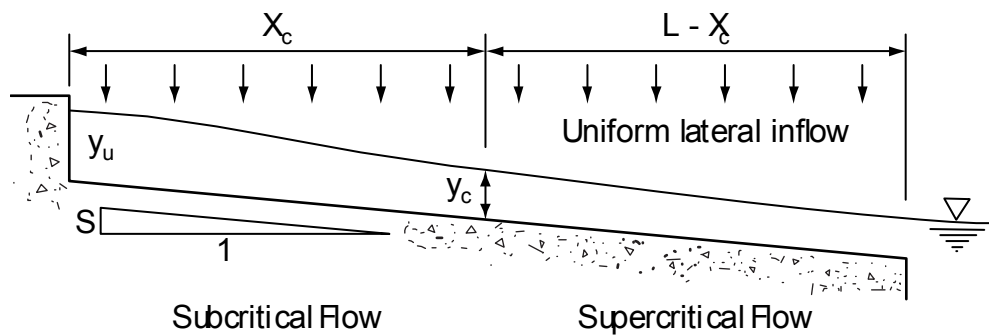
Figure 10.3 – Combined subcritical and supercritical flow

For a trapezoidal channel with critical depth occurring within the length of the channel, the location of critical depth may be determined from equation 10.3.

$$y_u = \left[2(y_c)^2 + \left(y_c - \frac{S.X_c}{3} \right)^2 \right]^{0.5} - \frac{2.S.X_c}{3} \qquad (10.3)$$

where:

   X_c =   length of channel containing subcritical flow (m)

$$X_c = \left(\frac{g.A^3}{q^2.B} \right)^{0.5} \qquad (10.4)$$

where:

   g =   acceleration due to gravity (m/s^2)

   A =   cross-sectional area of trapezoidal channel (m^2)

   B =   base width of trapezoidal channel (m)

Equations 10.1, 10.2 and 10.3 will all slightly underestimate the upstream flow depth because they ignore friction loss. The degree of underestimation will depend on the roughness of the channel.

10.2 Causeway crossings

By definition, a 'causeway' is any elevated carriageway extending across a watercourse or tidal water. In effect, this means any 'culvert' crossing may also be termed a causeway. The discussion however, will focus on those causeways containing only a low-flow pipe of such minor capacity that most flood flows are required to pass over the embankment.

A causeway overtopped by minor flood flows will act as a broad-crested weir with the discharge principally being controlled by the tailwater conditions. During low flows, upstream water levels may be independent of downstream conditions. Submerged flow conditions typically occur when tailwater levels are high—during which flow passing over the causeway remains subcritical. Book 7 of ARR (1998) and Department of Main Roads (2002) provide details of design methods for both tailwater situations.

A major consideration in the design of causeways is the safety of vehicles and pedestrians during periods of overtopping. In real terms, **no** flow conditions can be considered 'safe' for pedestrian while a causeway is subject to flood flows or recent storm runoff. For vehicles, a maximum depth*velocity product (d*V) of 0.3, a maximum flow depth of 200 mm should allow trafficable conditions across urban causeways (refer to tables 7.4.2 & 7.4.4). Guidelines are not provided here for traffic safety on rural causeways.

Warning signs should clearly indicate likely traffic hazards. These warning signs should indicate that safety risks exist whenever water is passing over the causeway.

Fish passage conditions are improved if the profile of the causeway follows the 'natural' cross-section of the streambed, thus providing variable flow depths over the causeway; however, such conditions are generally **not** recommended for reasons of traffic safety. Causeways are not considered desirable within fish passage streams.

10.3   Ford crossings

Fords are bed level crossings. Such crossings are generally only suitable for the crossing of alluvial streams (i.e. sand or gravel-based waterways). In some cases ford crossings can also be formed across ephemeral clay-based waterways; however, these crossings are usually only trafficable during periods of dry weather.

A primary feature of alluvial streams is the gradual downstream migration of bed material (as opposed to fixed-bed clay-based waterways). Consequently, fixed bed ford crossings (e.g. crossings stabilised with a concrete pad) should **not** be used to cross alluvial streams, especially if fish passage conditions are to be maintained along the waterway. Such fixed-bed structures can cause bed material to accumulated upstream of the crossing and a scour hole to form immediately downstream of the crossing.

On the other hand, ford crossings of ephemeral clay-based waterways generally need to be stabilised to minimise damage to the streambed during periods of wet weather. This bed stabilisation is best achieved through the use of rock recessed into the streambed.

10.4   Culvert crossings

Designers are referred to the detailed culvert design procedures presented within Department of Main Roads, 2002 (also refer to 2013 edition).

10.4.1   Choice of design storm

Table 7.3.1 provides recommendations for the selection of design storms for road culverts.

Some local governments may require flood free access to new residential development during the major design storm to provide safe passage for emergency vehicles. As a result, some culverts will be designed to carry the major design storm. In such circumstances, consideration should be given to the impact of flows greater than the major design storm as discussed in section 10.4.2 below. The potential impacts of full or partial debris blockage of the culvert must also be considered as discussed in section 10.4.10.

If a local government specifies a design storm less than the 1% AEP, it would be considered reasonable for the local government to also require that the effects of a 1% AEP design storm shall not unreasonably:

- increase the flooding of critical areas defined by the local government, such as habitable floor levels;
- adversely affect the actual or potential 'use' or 'value' of adjacent land
- cause unacceptable property damage.

10.4.2 Consideration of flows in excess of design storms

The likely effects of channel flows resulting from storm event in excess of the major design storm should be considered and the consequences discussed with the local government (also refer to sections 7.2.4, 7.3.3 and 9.3.2 of this Manual). This process may require the development of a 'Severe Storm Impact Statement' (refer to section 7.2.5).

Considerations of the potential impact of severe storms should to include the following:

- whether these impacts can be adequately predicted or modelled
- the likelihood of significant debris blockage of the culvert, roadway fences and crash barriers
- the relative elevation of property floor levels (residential or commercial) upstream and adjacent to the culvert
- the path of overflows (e.g. overflows may pass through downstream properties before entering the downstream channel) Figure 10.4.
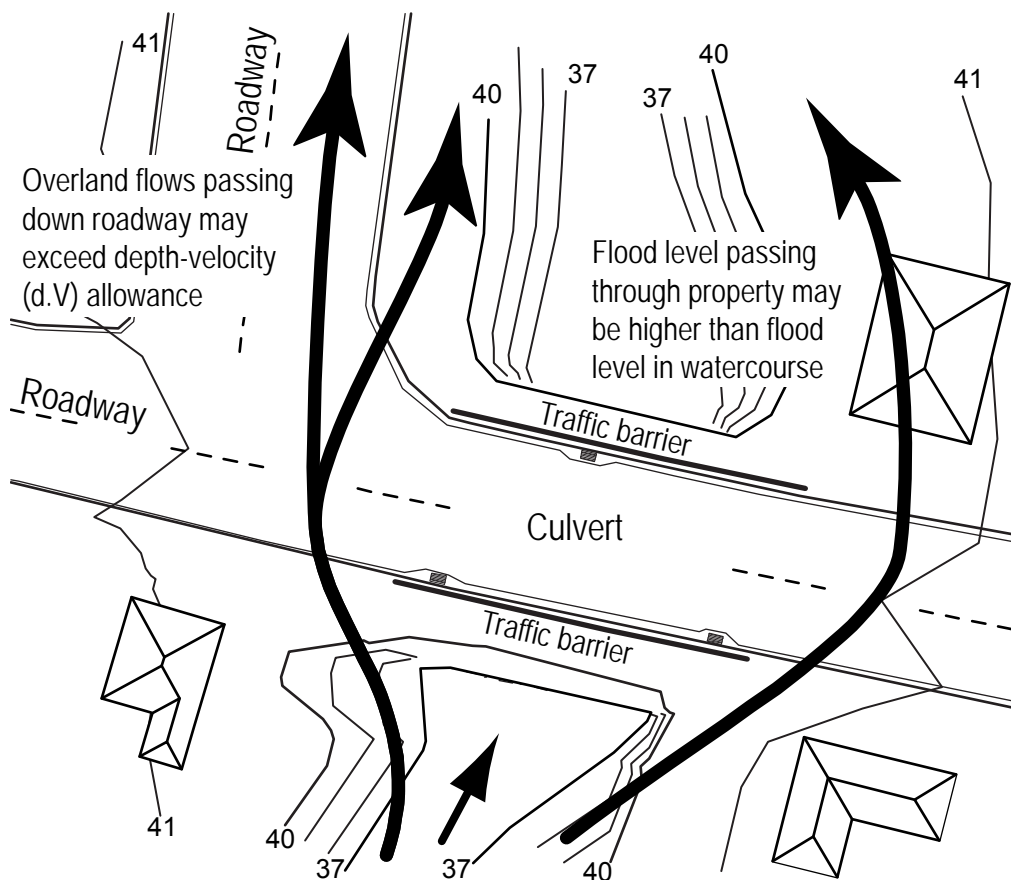
Figure 10.4 – Example of overtopping flows at an urban culvert crossing

10.4.3 Location and alignment of culverts

The location of a roadway crossing is usually governed by the location of an existing road reserve, but when circumstances allow, waterway crossings should ideally be located:

- on a straight section of the waterway
- well downstream of sharp channel bends
- on a stable channel section
- upstream of a channel riffle (i.e. locating the culvert within a 'pool' if a pool-riffle system exists within the stream).

Wherever practical, culverts should be aligned with the stream channel; however this can significantly increase the length and cost of the culvert (e.g. skewed culverts) compared to a culvert aligned perpendicular to the roadway. It is generally not considered acceptable to realign an existing waterway channel simply to reduce the length of a culvert. The advantages and disadvantages of each option should be considered on a case-by-case basis.

If it is not practical to align a culvert barrel with both the upstream and downstream channels, then priority should be given to aligning the culvert outlet with the direction of the downstream channel.

10.4.4   Allowable afflux

In choosing the allowable afflux caused by the culvert, designers shall consider the following:

- the afflux must not cause unacceptable damage to adjacent properties, or adversely affect the use of the land
- adequate freeboard (minimum desirable 100 mm) should exist between the design flood water surface and the lowest part of the road cross-section at the crossing.

10.4.5   Culvert sizing considerations

When sizing a culvert, the following recommendations/issues should be considered:

- The general minimum size of all cross drainage culverts should be 375 mm diameter for pipes and 375 mm height for box culverts.
- The larger the culvert cells, the lower the risk of debris blockage.
- To minimise the effects of debris blockage, and to minimise the drowning risk to a person swept through the culvert, all reasonable and practical measures should be taken to maximise the height of the culvert, even if this results in the culvert's hydraulic capacity exceeding the design standard.
- The Department of Main Roads (2000b) provides guidelines on minimum culvert sizes for terrestrial fauna passage.
- If fish passage through the culvert is considered necessary, then the minimum flow area may be controlled by fish passage requirements as discussed in section 10.4.14.
- In multi-cell culverts it may be desirable to include one or more larger-cells. These cells are usually recessed into the channel bed and are designed to allow sedimentation to occur within the culvert to simulate natural streambed conditions to aid fish passage.

10.4.6   Preliminary sizing of culverts

A first 'estimate' of the culvert size may be obtained using equation 10.5 (culverts flowing full only).

$$\Delta H = C \cdot (V^2/2g) \tag{10.5}$$

where:

ΔH = approximate head loss through culvert flowing full (i.e. outlet control) (m)
C = constant equal to 1.5 for large culverts, or 1.7 for small, high-friction culverts
V = average flow velocity within culvert = Q/A (m/s)
Q = total flow rate passing through culvert (m³/s)
A = total flow area of culvert (m²)
g = acceleration due to gravity (m/s²)

Equation 10.5 may be rearranged and presented as equation 10.6.

$$A = Q/(3.6 \, (\Delta H^{0.5})) \tag{10.6}$$

Following preliminary design, a more refined culvert size may be obtained from equation 10.7 (culverts flowing full only).

$$\Delta H = \left(K_e + \left(\frac{2\,g\,L\,n^2}{R^{4/3}} \right) + K_{exit} \right) \cdot \left(\frac{V^2}{2\,g} \right) \tag{10.7}$$

where:

K_e = entrance loss coefficient (assume 0.5 if unknown)

L = length of culvert (m)

n = average Manning's roughness of culvert

R = hydraulic radius of culvert flowing full (m)

K_{exit} = exit loss coefficient (assume 0.8 if unknown) otherwise the exit loss component equals the change in velocity head from within the culvert ($V^2/2g$) to a downstream location where the flow has expanded to approximately full channel width ($V_{d/s}^2/2g$), thus:

$$K_{exit}\ (V^2/2g) = (V^2/2g) - (V_{d/s}^2/2g) \tag{10.8}$$

10.4.7   Hydraulic analysis of culverts

Final hydraulic analysis of a culvert should be carried out either using hand calculations (minor culverts only), or using numerical modelling. Culverts should **not** be sized using *Inlet Control* charts if the outlet of the culvert is likely to be drowned (i.e. when *Outlet Control* conditions exists). It is noted that *Inlet Control* conditions require free surface flow conditions to exist at the culvert outlet.

10.4.8   Culvert elevation and gradient

In general, the culvert's invert should follow the stream's natural gradient; however, in medium to steep waterways this design aim can result in fish passage problems.

The invert of fish-friendly culverts should be set at an elevation that allows at least 0.2 to 0.5 m flow depth during periods of extended low flows (i.e. base flow conditions in perennial streams, or flow conditions following prolonged wet weather in ephemeral streams).

If rock or natural bed material is allowed to settle along the base of the culvert, then an allowance must also be made for the expected depth of this bed material (Figure 10.5). This may require the culvert to be set significantly more than 0.2 to 0.5 m below the existing bed level.
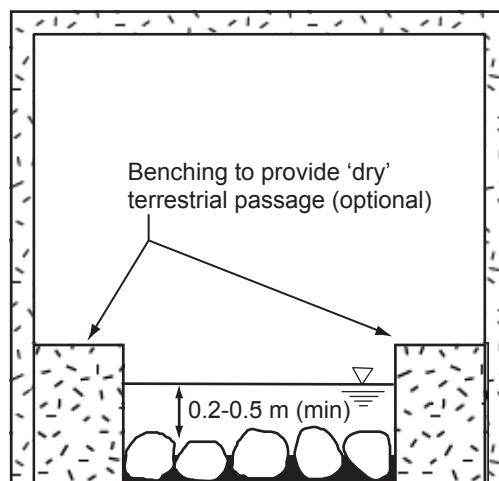
Figure 10.5 – Minimum desirable flow depth to achieve fish passage

In multi-cell culverts, at least one cell should be recessed into the bed to form a 'wet' cell. The remaining cells can be elevated as 'dry' cells suitable for terrestrial passage (Figure 10.6). An appropriate adjustment to the flow area and roughness needs to be made in the hydraulic analysis. Table 9.3.3 provides information on Manning's roughness values for rock-lined culvert cells.
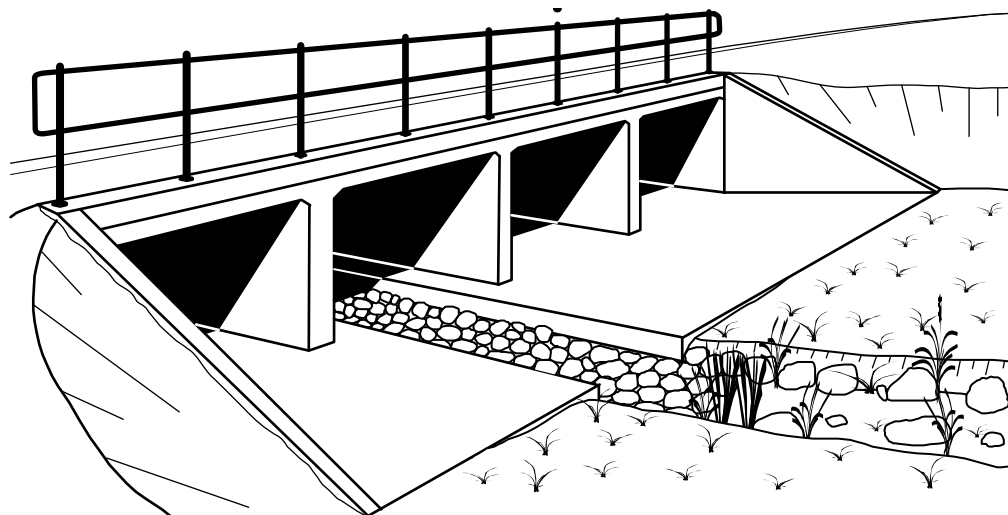
Figure 10.6 – Multi-cell culvert with 'wet' and 'dry' cells

Drop inlets shall not be used on culverts that are required to be fish friendly, unless suitable fish passage conditions are provided. Fish friendly drop inlets usually consist of a pool-riffle system, or a rock chute with maximum 1:20 to 1:30 (V:H) gradient.

10.4.9　Minimum cover

Depending on the concrete/loading class, the generally accepted minimum allowable fill is 300 mm over Concrete Pipes, 100 mm over Reinforced Concrete and Slab Link Box Culverts (RCBC and SLBC respectively) and Reinforced Concrete Slab Deck Culvert (RCSDC) and 600 mm over Corrugated Metal Pipes.

Designers should refer to the latest recommendations from the Concrete Pipe Association of Australasia to confirm desirable minimum cover requirements.

10.4.10　Blockage considerations and debris deflector walls

Waterway culverts can experience debris problems in a number of circumstances, especially where:

- the upstream waterway is heavily vegetated and is currently undergoing channel expansion due to changing catchment hydrology
- the waterway has a large catchment area
- the culvert has insufficient clear waterway area to allow the free passage of debris
- the culvert is downstream of potential slip areas that could result in significant debris flow
- the culvert has a history of debris problems.

Hydraulic analysis of a culvert should take reasonable consideration of likely debris blockage. Engineers Australia (2012) provides a detailed risk assessment process for determining the likelihood of debris blockage at cross drainage structures. In the absence of a site-specific risk assessment, Table 10.4.1 provides suggested blockage factors for culverts.

The severe storm blockage conditions presented in Table 10.4.1 are suggested blockage conditions that should be considered reasonably possible during severe storms.

Table 10.4.1 – Suggested blockage factors for culverts [1]

Culvert conditions	Blockage factor	
	Design value	Severe storm
Inlet height < 3 m, or width < 5 m:		
Inlet	20%	100% [3]
Chamber (barrel)	[2]	
Inlet height > 3 m and width > 5 m*:		
Inlet	10%	25%
Chamber (barrel)	[2]	[2]
Culvert inlets with effective debris control features for culverts with inlet height > 3 m and width > 5 m	As above	As above
Screened culvert inlets	50%	100%

Notes (Table 10.4.1):

[1] Developed from Engineers Australia (2012).

[2] Adopt 25% bottom-up sediment blockage unless such blockage is unlikely to occur.

[3] Degree of blockage depends on availability of suitable bridging [4] matter. If a wide range of bridging matter is available within the catchment, such as large branches and fallen trees, then 100% blockage is possible for such culverts.

[4] Bridging matter refers to the material of sufficient strength to bridge across the opening of a structure, and on which other blockage matter can collect, thus potentially resulting in the full or partial blockage of a structure.

One means of maintaining the hydraulic capacity of culverts in high debris streams is to construct debris deflector walls as shown in Figure 10.7. The purpose of these walls is to allow the debris raft to rise with the flood, thus maintaining a relatively clear flow path under the debris.
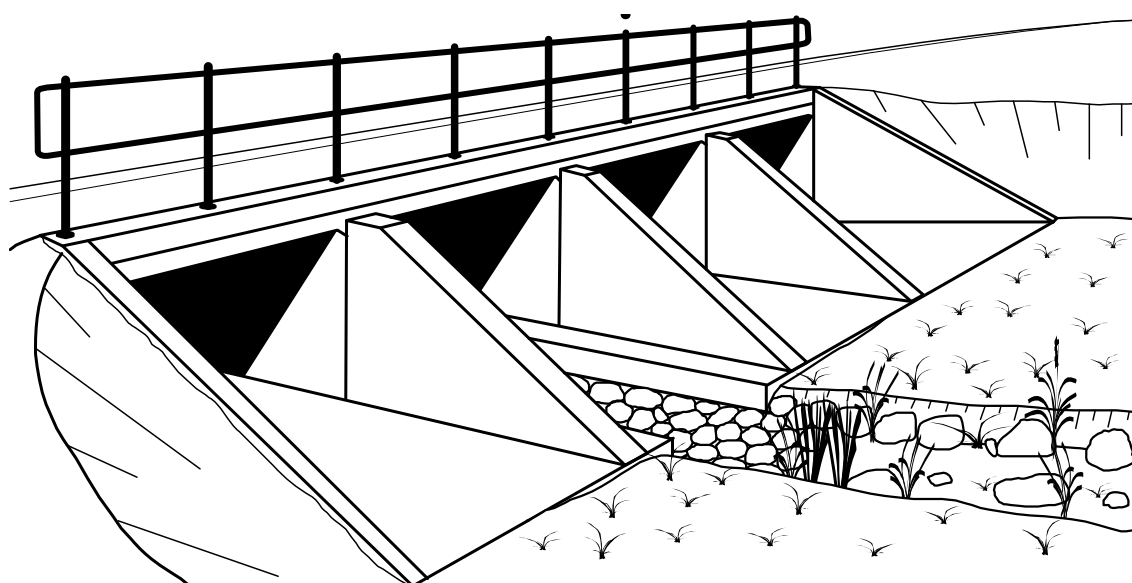
Figure 10.7 – Culvert inlet with debris deflector walls

10.4.11 Sediment control issues

Multi-cell culverts typically experience sedimentation problems within the outer cells of the culvert. This is primarily caused by the stream channel trying to reform the natural channel cross-section that existed prior to construction of the culvert as shown in Figure 10.8.
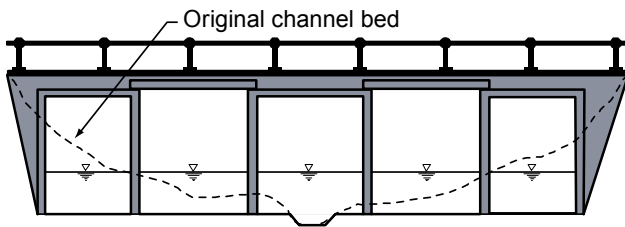
Figure 10.8 (a) – Multi-cell culvert showing original channel cross-section
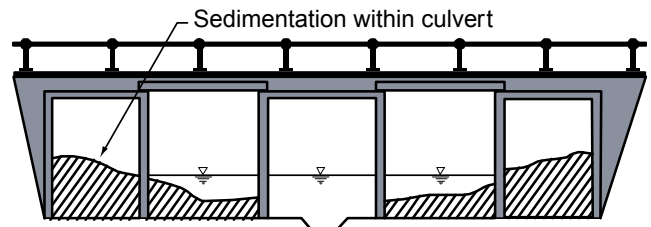
Figure 10.8 (b) – Typical long-term sedimentation within alluvial waterways

Sedimentation of culverts can be managed using one or more of the following activities:

- formation of an in-stream sedimentation pond upstream of the culvert
- formation of a multi-cell culvert with variable invert levels such that the profile of the base slab simulates the natural cross-section of the channel
- installation of *sediment training walls* on the culvert inlet.

Sediment training walls reduce the risk of sedimentation of the outer cells by restricting minor flows to just one or two cells as shown in figures 10.9 and 10.10.
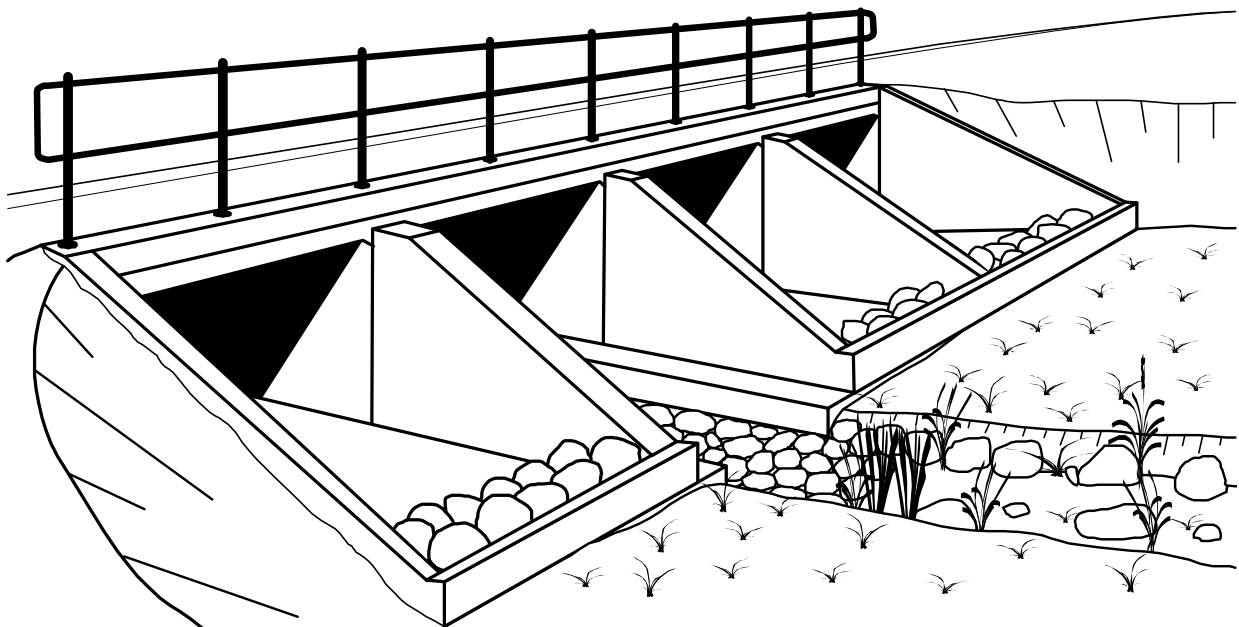
Figure 10.9 – Sediment training walls incorporated with debris deflector walls
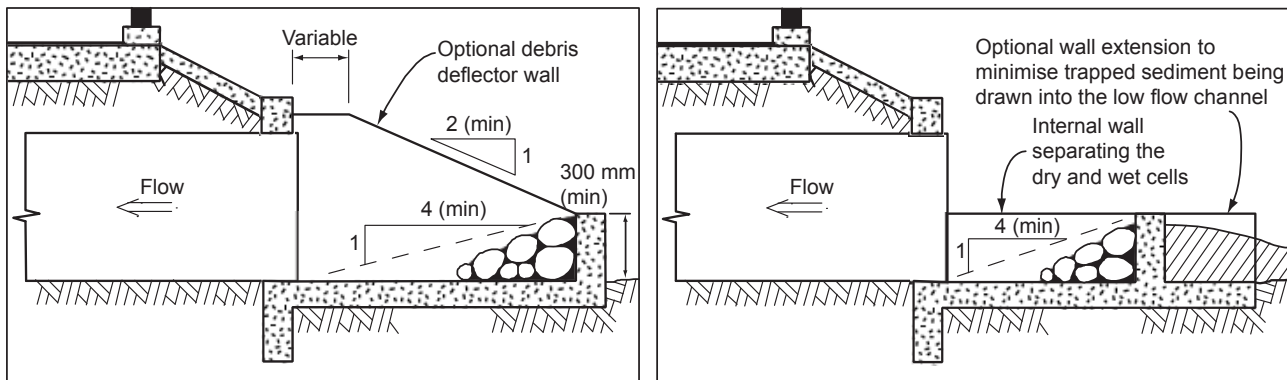
Figure 10.10 – Various arrangements of sediment training walls with (left) and without (right) a debris deflector wall

If the culvert is located within a terrestrial passage corridor, it may be necessary for grouted rock ramps (Figure 10.10) to be formed on the downstream face of the training walls to assist in the passage of terrestrial wildlife such as tortoises. As with all aspects of sediment training walls, the application of this feature should be assessed on a case-by-case basis.

As with debris deflector walls, the use of sediment training walls should be restricted to those culverts where the benefits gained by their use outweigh the additional costs. In most cases, their use will be restricted to clay-based creek systems.

10.4.12 Roadway barriers

Prior to the installation of any traffic safety barriers, consideration must be given to their impact on flood levels and terrestrial passage. During overtopping flows, raised median strips can raise upstream flood levels as well as restrict traffic movement to one side of the road. In critical flood control areas, it may be necessary to use a painted median.

10.4.13 Terrestrial passage requirements

Terrestrial passage is normally required to be incorporated into the design of a culvert when the road crosses a fauna corridor and traffic conditions on the road are such that unacceptable road kills are likely occur.

Dry passage should extend through the culvert along one or both sides of waterway channel as required. These dry paths should extend along the wing walls until they intersect with the waterway bank. Guidelines on the integration of terrestrial passage into waterway crossings may be obtained from Department of Main Roads (2000b).

10.4.14 Fish passage requirements

Fish passage considerations are normally required in the following circumstances:

- as directed by Queensland Fisheries (DAFF) or the local government (refer to Queensland Fisheries waterway mapping)
- when identified within a Wildlife Corridor Map
- streams containing permanent water (pooled or flowing)
- streams containing aquatic life that requires passage.

Fish passage may also be required through dry-bed culverts located within a floodplain adjacent to a bridge crossing (Figure 10.11). Such conditions would normally exist within river systems containing fish species that primarily migrate along floodplains during high flows.
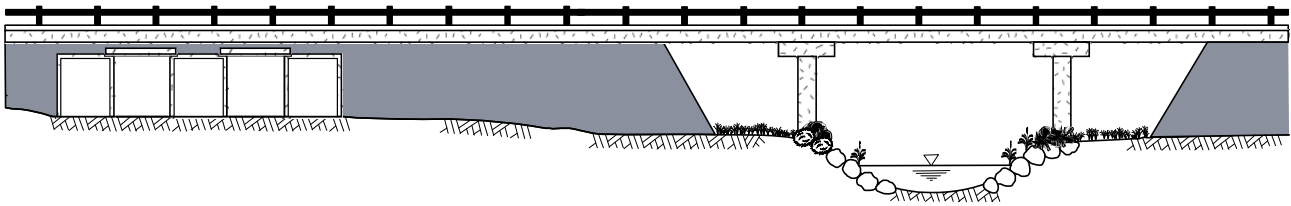
Figure 10.11 – Floodplain culvert adjacent a bridge crossing

Desirable hydraulic conditions for fish passage may exist for a wide range of flow rates. Witheridge (2002) describes three design conditions: 'High', 'Medium' and 'Low-flow' designs. The *self-assessable codes* developed by Queensland Fisheries (DAFF) assign minimum design standards for various waterway classifications defined within their waterway mapping system (i.e. 'green', 'amber', 'red' 'purple' zones).

10.4.15 Outlet scour control

Discussion on the attributes of various energy dissipaters is provided in section 8.7 of this Manual. In most cases, safety concerns will prevent the use of most plunge pool and impact energy dissipaters, thus limiting downstream scour control to the use of outlet rock pads.

The required depth of apron cut-off walls is dependent on a number of factors including flow rate, outlet velocity, and type of bed material. A minimum depth of cut-off wall penetration of 0.6 m is recommended unless otherwise directed by the local authority. In critical situations, designers should consult Chiu & Rahmann (1980) and Peterka (1984) for procedures concerning the determination of required cut-off wall depths.

Figure 10.12 (after Witheridge, 2012b) provides guidance on the selection of mean rock size and length of scour protection downstream of multi-cell stormwater outlet and culverts.
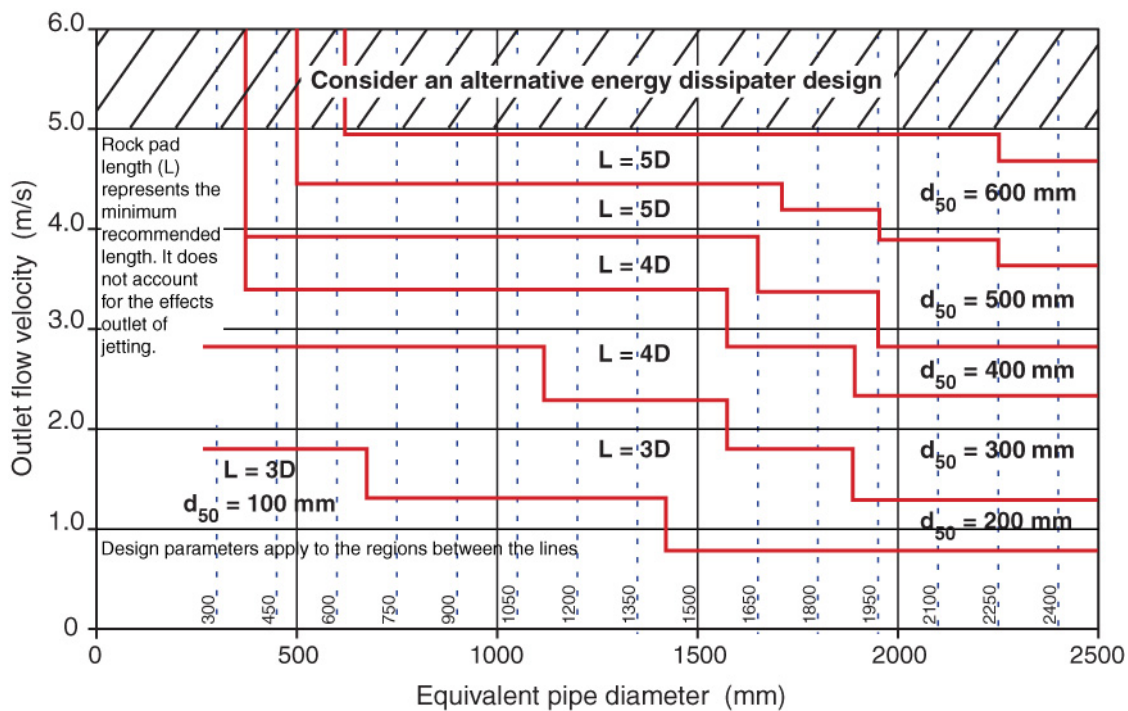
Figure 10.12 – Sizing of rock pad outlet structures for multi-pipe and box culvert outlets

11.   Environmental considerations

11.1   Introduction

The national framework for the management of water quality, including stormwater management, is the National Water Quality Management Strategy (NWQMS).

A legal cornerstone for the environmentally responsible management of stormwater within Queensland is the general environmental duty as presented within the *Environmental Protection Act 1994*, that being:

> 'A person must not carry out any activity that causes, or is likely to cause, environmental harm unless the person takes all reasonable and practicable measures to prevent or minimise the harm.'

It is recognised that there is still a significant degree of uncertainty associated with many of the environmental aspects of stormwater management, including:

- pollution loading generated from different land uses in different regions

- the response of various receiving waters to pollutant loadings

- the geomorphological response of waterways to changes in catchment hydrology

- the effectiveness of different stormwater treatment measures within different geographical regions, flow regimes and catchment conditions

- the relationship between the predictions of water quality models and real world outcomes.

In addition to the above, there are the ongoing hydrologic uncertainties associated with:

- rainfall prediction

- long-term hydrologic changes (e.g. climate change)

- errors associated with hydrologic modelling

- errors associated with the numerical modelling of complex waterway hydraulics

- data collection and monitoring errors.

Despite these uncertainties, stormwater managers must take all reasonable and practicable measures to prevent, or at least minimise potential environmental harm caused by stormwater runoff and the construction and operation of stormwater management systems. Specifically, consideration must be given to potential adverse impacts resulting from changes to the natural water cycle, water quality and the volume, rate, velocity, duration and frequency of stormwater runoff.

In circumstances where these uncertainties are significant, or where significant questions arise regarding the suitability of a proposed stormwater management system, consideration should be given to the following:

- A lack of understanding, or the degree of uncertainty associated with an action or response, should not be used in **isolation** as an excuse to avoid the incorporation of current best management practice within stormwater design.

- It is the responsibility of the designer to be aware of current best management practice within the stormwater industry.

- The preferred design outcomes should be those that retains sufficient space and capabilities (to the best estimate of the designer) within a stormwater catchment for the future upgrading of

the stormwater treatment system once a better understanding of the treatment system and/or the catchment's needs has been achieved. It is noted that retro-fitting drainage layouts and treatment systems is very difficult when 'space' becomes the major site constraint.

- Wherever practical, stormwater treatment systems within a catchment or sub-catchment should not rely on a single treatment system or product, but should incorporate diversity i.e. the treatment train approach.

11.2 Waterway management

11.2.1 General

The following discussion focuses on the potential impacts of stormwater management systems on the physical aspects of urban waterways. For the purpose of this discussion, reference is made only to vegetated waterways including creeks, rivers, estuaries and constructed channels having a natural appearance.

11.2.2 Waterway integrity

Significant changes can occur to the structural integrity of urban waterways following a change in the catchment hydrology. The degree of change primarily depends on the type and degree of changes to the catchment's runoff characteristics. These changes may result from the full or partial urbanisation or de-forestation of the catchment.

Land clearing, even if replaced by vegetative surfaces such as grass or crops, can significantly alter the runoff characteristics of a catchment, and as a result cause long-term changes to downstream waterways. Specifically, de-forestation has the potential to:

- reduce initial rainfall losses
- significantly increase the total annual runoff volume
- increase the frequency of minor stream flows
- reduce the effective 'time of concentration' (refer to Chapter 4) of stream flows
- initiate gully erosion
- alter the morphology of downstream waterways.

Even though erosion is a natural aspect of all waterways, the basic aim is to avoid an un-natural acceleration or deceleration of this erosion. Stream flows at or near the bankful flow rate are generally considered to have the greatest influence on channel erosion; however, once significant vegetation loss has occurred within a waterway, regular bed and bank erosion can be initiated by much smaller flows.

The impacts of land clearing and urbanisation are more likely to affect minor waterways such as creeks. There are generally four types of creek systems: clay-based, sand-based, gravel-based and spilling (rock-based) creeks. Each of these creek systems will respond differently to changes in catchment hydrology.

It is not possible to 'accurately' predict the response of a natural waterway to changes in catchment hydrology. Past history has shown that in the absence of major flow control systems (i.e. dams and large retention basins) urbanised creeks typically expand from around a 1 to 2 year bankful capacity to around a 1 in 5 to 1 in 10 year bankful capacity. There are of course many exceptions to this typical outcome.

Changes to a waterway cross-section can result in many adverse effects, including:

- Channel expansion causes significant amounts of coarse sediment to be released into the waterway. This sediment smothers aquatic habitats, harms the ecological benefits of riffle systems, in-fills pools, smothers essential bed vegetation, increases the potential for weed growth within the channel, and initiates bank erosion caused by the excessive growth of reeds within the settled bed sediments.

- Loss of useable land from properties adjacent to the waterway.

- Damage to both private and public assets located immediately adjacent an expanding urban waterway, and the public and private expense of stabilising these waterways to prevent further damage. Often these stabilisation works involve the use of hard-engineering measures, which can further harm aquatic and riparian ecosystems.

- Lateral movement of the waterway channel within the floodplain. It is noted that the radius of a channel bend is usually dependent on the top width of the channel. Thus an expansion of the channel often results in a change in the streams meander pattern.

- Conversion of some sections of 'closed-canopy creeks' into 'open-canopy creeks'. Such changes can significantly change bed and bank vegetation, increase low-flow water temperatures, and alter the ecological balance within the affected reach of the waterway.

- Potential changes to the cultural and spiritual values of the waterway. For example, excess sedimentation may alter the traditional use of water holes. All in-stream works (construction and maintenance) need to comply with the *Aboriginal Cultural Heritage Act 2003*. The main purpose of the act is to provide effective recognition, protection and conservation of Aboriginal cultural heritage.

The types of waterways that are most susceptible to physical change caused by changes in catchment hydrology include:

- natural creek systems
- constructed, vegetated channels of natural appearance.

Large waterways, such as river systems, are rarely physically altered as a result of the hydrological changes resulting from urbanisation. Large waterways are more susceptible to hydrologic changes resulting from the introduction of major storage reservoirs, or changes in land use over a significant proportion of the catchment such as farming and de-forestation.

11.2.3 Effects of changes in tidal exchange

Flood mitigation works often involve increasing the channel capacity of waterways. If these works occur within a tidal reach of the waterway, then there is the potential for these works to increase the volume of tidal exchange. Designers need to investigate and address potential problems including the following issues:

- scour problems resulting from changes in channel velocity and the redistribution of flows within the waterway
- potential flooding concerns associated with increased mangrove growth within upstream waterways and drainage channels
- ecological problems resulting from a change in tidal flow, including changes in water quality, water salinity, and the extent of tidal flow
- changes to the tidal exchange within tidal wetlands, or the introduction of tidal exchange within freshwater wetlands
- encroachment of saline water into fresh water environments can also impact on concrete structures as well as vegetation.

11.2.4　Cause and effect of changes in catchment hydrology

Table 11.2.1 summarises the possible causes of changes in waterway characteristics.

Table 11.2.2 summarises likely impacts of land use change on catchment hydrology and waterway characteristics.

Table 11.2.3 summarises likely impacts of various stormwater management practices on catchment hydrology and waterway characteristics.

Table 11.2.4 summarises likely benefits of various stormwater management practices on catchment hydrology and waterway characteristics.

Table 11.2.1 – Possible causes of changes in waterway characteristics

Change in waterway	Possible causes of changes in waterway characteristics
Creek erosion	• Increase in the duration or frequency of near-bankful flows, typically the 1 in 1 year to 1 in 10 year AEP events. • Increase in channel flow velocity (possibly caused by an increase in channel grade, straightening of the channel, decrease in channel roughness, or a lowering of downstream water levels).
Stress to aquatic habitats and ecosystems	• Increase in the duration, velocity or frequency of low flows (i.e. the runoff from regular minor storms less than the 1 in 1 year event). • Deterioration in the water quality of low flows within creeks and minor waterways. The critical flows are the dry weather base flows and those extended low flows that occur for days or weeks after wet weather. • Deterioration in the water quality of major flows within lakes, wetlands, rivers and other major waterways. • Inflow of coarse sediment (during any storm event). • An increase in the area of impervious surfaces directly connected to an impervious drainage system.
Deterioration of water quality	• Urbanisation of the catchment and the consequent higher pollutant loading. • Inadequate erosion and sediment control measures applied on building and construction activities. • Long-term damage to grassed surfaces (e.g. parks and road verges) causing ongoing soil erosion.
Weed infestation of banks and riparian zones	• Urbanisation of the catchment and the consequent higher nutrient loading. • Removal of canopy cover from urban waterways. • Direct connection of drainage systems to the waterway. • Inappropriate selection of street trees. • Plant and seed infestation originating from private property.
Weed/reed infestation of channel bed	• Removal of canopy cover from urban waterways. • Direct connection of drainage systems to the waterways. • Inflow of coarse sediment (during any storm event). • Accelerated creek erosion resulting in an increased channel top width, loss of canopy cover, and increased bed load sediment.

Table 11.2.2 – Likely impacts of land use change on catchment hydrology and waterway characteristics

Land use change	Likely changes
Changes in the fire management of bushland, including management of fuel load	• Increase or decrease in volume of runoff. • Reduced initial loss rates. • Increased frequency of minor flows. • Possible increase in frequency and duration of bankful flows. • Increased peak discharge rates. • Increased erosion and sediment flow within alluvial streams (e.g. sand-based and gravel-based creeks). • Channel expansion within well-vegetated, clay-based creeks.
De-forestation for the development of grasslands, including farming and rural-residential development	• Significant increase in runoff volume. • Reduced initial loss rates. • Increased frequency and duration of channel flows. • Increased frequency and duration of over-bank flows. • Increased peak discharge rates for all but extreme flood events. • Significant increase in erosion and sediment flow within alluvial streams with a resulting increase in channel depth and/or width. • Channel expansion within well-vegetated, clay-based creeks, and possibly a significant increase in sediment flow. • Gully erosion extending laterally from existing creeks.
Urbanisation of farmland or grassland	• Significant increase in runoff volume. • Reduced initial loss rates. • Significant increase in frequency and duration of in-bank flows. • Increased peak discharge rates for all but extreme flood events. • Possible increase in average recurrence interval (ARI) of bankful flows. • Significant increase in erosion and sediment flow within minor waterways (i.e. creeks) with a resulting increase in channel depth and/or width.
Urbanisation of bushland	• Significant increase in runoff volume. • Significant reduction in initial loss rates. • Significant increase in the frequency and duration of in-bank flows. • Significant increases in peak discharge rates. • Significant increase in the average recurrence interval (ARI) of bankful flows. • Significant increase in erosion and sediment flow within minor waterways (i.e. creeks) with a resulting increase in channel depth and/or width. • Development of open-canopy, weed-infested creek systems.

Table 11.2.3 – Likely impacts of various stormwater management practices on catchment hydrology and waterway characteristics

Stormwater practice	Likely changes in catchment hydrology and waterway characteristics
Adoption of rainwater tanks and/or stormwater harvesting systems	• Slight decrease in annual volume of runoff. • Potential increase in the duration of the 'critical storm'. • Slight decrease in the volume, rate, frequency and duration of minor flows. • Increase or decrease in dry weather base flows depending on whether the stormwater is used for garden watering. • Altered flow conditions possibly affecting aquatic biota.
Establishment of a piped drainage system throughout the catchment	• Significant decrease in the duration of the 'critical storm'. • Significant increase in peak flows. • Increase in the frequency of in-bank flows. • Decrease in dry weather base flows. • Significant erosion and expansion of natural creeks.
Channelisation of minor creeks and overland flow paths	• Significant decrease in the duration of the 'critical storm'. • Significant increase in peak flows. • Increase in the frequency of in-bank flows. • Significant erosion and expansion of downstream creeks. • Decline in biodiversity and ecosystem values.
Adoption of on-site detention (OSD) in association with a piped drainage system and/or channelisation of overland flow paths and creeks	• Increase in flood flows can still occur downstream of the piped drainage and channelised flow paths. • Decrease in peak flows from minor storms. • Potential increase in the duration of channel flows. • Significant erosion and expansion of medium to large creeks, but possibly little change in minor creeks (i.e. the benefits of OSD decrease with increasing catchment area).
Use of regional detention and retention basins sized for flood control only	• Possible increase or decrease in 'critical storm duration' compared to the undeveloped catchment. • Significant increase in the duration of in-bank and bankful flows. • Significant erosion and expansion of creek channels.
Use of extended detention basins	• Possible increase or decrease in 'critical storm duration' compared to the undeveloped catchment. • Significant increase in the duration of post-storm flows. • Possible minor erosion of creek channels.
Adoption of Water Sensitive Urban Design	• Possible increase in annual volume of runoff, although less than for traditional urban developments. • An increase in stress to aquatic biota may still occur. • Possible minor erosion of creek channels.

Table 11.2.4 – Likely benefits of various stormwater management practices on catchment hydrology and waterway characteristics

Stormwater practice	Likely benefits compared to traditional stormwater management systems
Water Sensitive Urban Design	• Reduced changes to the volume, rate, frequency and duration of runoff as a result of urbanisation. • Reduced changes to pollutant runoff. • Improved low-flow water quality. • Reduced impact of development on in-stream ecological values and biodiversity. • Reduced likelihood of waterway erosion/expansion.
Stormwater designs based on minimal changes in runoff volume	• Reduced changes to the frequency, rate and duration of runoff. • Reduced changes to pollutant runoff. • Reduced likelihood of waterway erosion/expansion.
Use of extended detention basins	• Reduced changes to the rate and duration of high-flows, but an increase in the duration of low-flows. • Improved water quality of in-stream pools through the provision of prolonged post-storm low-flows. • Reduced likelihood of waterway erosion/expansion.
Use of detention basins sized for low-flow discharge	• Reduced changes to the rate and duration of bankful flows, but increase in the duration of low-flows. • Reduced likelihood of waterway erosion/expansion.
Replacement of traditional piped drainage with swales, vegetated drainage channels and the preservation of natural waterways	• Increase in effective 'time of concentration' relative to a traditional piped catchment. • Reduced changes in rate of runoff. • Reduced changes to pollutant runoff. • Improved low-flow water quality. • Preservation of in-stream ecological values and biodiversity. • Reduced likelihood of waterway erosion/expansion.
Use of Natural Channel Design for constructed drainage channels	• Increase in effective 'time of concentration' relative to a piped or channelised catchment. • Slightly reduced changes in rate of runoff. • Improved low-flow water temperature and habitat value.

11.2.5 Fauna issues

Fauna issues need to be considered in the design of many waterway structures as summarised in Table 11.2.5.

Table 11.2.5 – Incorporation of fauna issues into waterway structures

Structure Type	Fauna Issues
Bridges, and culverts	• Detailed discussion and design guidelines on fauna passage are provided in sections 9.7.3, 9.7.4, 10.4.13 and 10.4.14.
Channel stabilisation works	• Detailed discussion on habitat impacts of various bed and bank stabilisation methods is provided in Brisbane City Council (1997).
Constructed wetlands and treatment ponds	• The provision of suitable fish passage conditions both into and out of a wetland can significantly improve mosquito control. • Constructed wetlands can provide excellent bird habitat; however, the impact of bird life on water-borne pathogen levels must be considered.
Gross pollutant traps and trash racks	• If it is necessary to place a trash rack within an aquatic habitat, consider the use of overlapping, partial width screens that allow unrestricted aquatic passage between the two screens. • GPTs placed downstream of urban lakes should incorporate coarse vertical bar screens suitable for fish passage.
Lakes	• Constructed in-stream lakes typically provide significant restrictions to fish passage resulting from upstream and downstream water level controls. • Urban lakes can cause a significant discontinuity in terrestrial movement corridors. Where possible, a riparian zone should be established along at least one side of the lake.
Snag management within urban waterways	• The retention of snags in urban waterways may cause adverse water quality conditions due to the high nutrient loadings expected within urban runoff (refer to discussion in Table 9.6.1). • Though essential within natural catchments for aquatic biota habitat, the retention of snags within urban waterways should be assessed on a case-by-case basis.
Vegetated drainage channels	• Detailed discussion and design guidelines on fauna passage are provided in Chapter 9 and in Brisbane City Council (2000a).
Waterway corridors	• Urban waterway corridors often act as the primary terrestrial wildlife corridor. • The development of a council-wide Wildlife Corridor Plan that identifies linkages between terrestrial and riparian corridors can provide a valuable planning tool for urban development.
Weirs and grade control structures (e.g. drop structures, riffles, chutes, rock weirs)	• Potential restriction to aquatic passage. • Limit fall height to 0.5 m wherever practical within aquatic habitats, otherwise the structure should incorporate an appropriate fishway. • Fish 'ladders' should not be used on bed control structures.

11.3   Stormwater quality management

11.3.1   Planning issues

The long-term success of water sensitive urban developments depends largely on the appropriate planning of the development layout. The following guidelines may assist in the successful planning of water sensitive residential and commercial developments.

Step 1: Consider soil properties

- The selection of the preferred stormwater treatment and conveyance measures should reflect the soil infiltration capacity.

- Stormwater infiltration measures should be given priority when working in soil with a high infiltration capacity (e.g. sandy soils).

- In clayey soil areas, many stormwater treatment measures will require the establishment of a subsoil drainage system. It is essential for this subsoil drainage system to be allowed to drain freely into either a stormwater pipe or open channel.

- Soil properties can have a significant bearing on the long-term outcomes of urban lakes. The existence of dispersive soils may result in urban lakes having a permanent brown colour. The expected social acceptance of a lake's colour should be given appropriate consideration during project planning.

Step 2: Consider opportunities for stormwater infiltration

- The promotion of stormwater infiltration is desirable in most cases, even on clayey soils; however, potential problems must be considered.

- Promoting stormwater infiltration can result in permanent seepage problems along boulder/retaining walls in terraced estates unless adequate subsoil drainage provisions are included. Infiltration systems need to be sustainable without causing disputes between neighbouring properties.

- The promotion of stormwater infiltration may also cause or aggravate salinity problems further down the slope or catchment. Such problems can occur well away from the property being developed.

- The promotion of stormwater infiltration does not necessarily mean the use of grass swales. It can also be achieved through the use of bio-filtration systems and rubble pits.

- Appropriate landscaping is critical for the long-term success of most infiltration systems.

Step 3: Look for natural opportunities within the catchment

- Look for opportunities to use the natural features of the catchment to optimise the cost-effectiveness and efficiency of the stormwater management system.

- Identify those areas of land with topographic features best suited to specific stormwater treatment systems (e.g. natural detention areas for wetland placement and highly porous soils for infiltration systems).

Step 4: Consider the maintenance capabilities of the land owner

- Avoid using stormwater treatment techniques that require maintenance funding or equipment that is beyond the capabilities of the asset manager.

- Stormwater treatment systems that require the access of personnel into confined spaces (e.g. some OSD systems) should not be incorporated into residential or commercial properties

unless supported by a risk assessment study. A detailed maintenance manual that clearly identifies maintenance risks, issues and procedures must be prepared to the satisfaction of the local government.

Step 5: Review conditions for the retention of natural waterways and the adoption of Natural Channel Design procedures

- Guidelines for the retention of natural waterways are provided in section 9.2 (b) of this Manual.

- Guidelines for the adoption of Natural Channel Design are provided in section 9.6 of this Manual.

Step 6: Look at the needs of receiving waters

- Certain stormwater treatment systems are preferred adjacent to certain receiving waters (refer to Table 11.5.6).

- As a general guide, large water bodies such as lakes and rivers, are adversely affected more by fine sediments generally less than 100 µm (i.e. turbidity) than coarse sediments, and thus requires good management of clayey soils. Conversely, minor water bodies, such as creeks and wetlands, experience greater physical change as a result of the inflow of coarse sediments. Note; this does **not** imply that large water bodies do not experience problems resulting from coarse sediment, or that small water bodies do not experience problems resulting from turbidity.

- The groundwater can be a 'receiving water' for significant quantities of stormwater. It is important that the environmental values of groundwater are identified. If groundwater quality issues are critical, then designers may need to modify the detail design of WSUD features to prevent polluted runoff from entering the groundwater system.

11.3.2   Water Sensitive Urban Design

Water Sensitive Urban Design (WSUD) involves the integration of urban stormwater, water supply, and wastewater issues during the planning and design of urban developments in a manner that uses water in a resource-sensitive and ecologically sustainable manner.

Water Sensitive Urban Design seeks to:

- Preserve the existing topography and features of the natural drainage system including waterways and water bodies.

- Integrate public open space with stormwater drainage corridors to maximise public access, passive recreation activities and visual amenity, while preserving essential waterway habitats and wildlife movement corridors.

- Preserve the natural water cycle including minimising changes to the natural frequency, duration, volume, velocity, and peak discharge of urban stormwater runoff.

- Utilise surface water and groundwater as a valued resource.

- Protect surface water and groundwater quality.

- Minimise the capital and maintenance costs of stormwater infrastructure.

It is recommended that the principles of WSUD are applied wherever practical to greenfield urban developments as well as infill developments and urban redevelopment programs.

11.3.3   Water Sensitive Road Design

Water Sensitive Road Design (WSRD) focuses on water-sensitive stormwater management within car parks and road reserves. The principles can be applied to both urban and rural roads.

Basic design tools incorporated into WSRD are:

- minimising the extent of impervious surfaces
- stormwater detention/retention
- stormwater treatment systems
- pollution containment systems
- the concepts of indirectly connected impervious surface area
- appropriate street landscaping.

Design features of WSRD include the following:

(a)   Minimising the extent of impervious surfaces

- Use of narrow, single-crossfall residential roads incorporating road drainage along only one side of the roadway.
- Provision of adequate formal on-street and/or off-street parking to prevent vehicular damage to roadway verge and other grassed areas. It is noted that vehicle damage often leads to soil compaction, loss of grass cover and ongoing soil erosion (water quality) problems.
- Use of permeable pavements, wherever practical, for car parks and pedestrian areas (e.g. CBD and community areas).
- Incorporation of a footpath along only one side of the road reserve.

(b)   Stormwater detention and retention

- Incorporation of stormwater detention and water quality treatment into roundabouts (primarily used along sub-arterial and arterial roads).
- Incorporation of stormwater detention and water quality treatment into the median of dual-carriageways. This can include dual-carriageway sub-arterial roads entering large residential estates.
- Incorporation of low-velocity drainage swales along sub-arterial and arterial roads.

(c)   Stormwater treatment systems

- Priority given to the treatment of road runoff from areas where there is a high concentration of vehicle braking and turning (i.e. roundabouts, intersections and off-ramps).
- Incorporation of grassed swales (where appropriate) to reduce total pollutant loadings to receiving waters.
- Incorporation of water treatment systems into roadway features such as bio-retention filters into traffic calming devices.
- Incorporation of litter collection systems into the car parks and surrounding roadways of shopping centres, takeaway food centres, community areas, entertainment facilities and sporting fields.
- Incorporation of public education messages onto the face of stormwater inlet lintels (e.g. PROTECT OUR WATERWAYS – FLOWS TO CREEK).

- In rural areas, the retention of sediment basins—established during the construction of the road—as permanent pollution containment systems (see (d) below) or stormwater treatment devices.

(d)   Pollution containment systems

Pollution containment systems are different from traditional stormwater treatment devices in that they are primarily designed to capture pollutant runoff from isolated incidents such as traffic accidents. The pollution is collected after the incident and removed from the site for treatment and disposal.

Design issues include:

- Use of pollution containment systems at critical locations including: freeway off-ramps; roadways where there is a high risk of traffic accidents particularly involving industrial transport vehicles; roadways immediately up-slope of critical waterway habitats; high risk industrial estates.

- Roadside detention/retention basin outlet structures modified to allow Emergency Services (e.g. fire service, councils, EHP) to temporarily shut-off the basin's outlet system to allow the containment and later removal of pollutant spills. This typically involves the use of a gate or stop board system.

- The incorporation of oil skimmers into the outlet structures of roadside retardation basins and constructed wetlands.

- The incorporation of oil skimmers, or other appropriate hydrocarbon treatment systems, into long-term or high volume car parks (e.g. large shopping centres, airports, bus interchanges and railway stations).

(e)   Indirectly connected impervious surface areas

- Minimising the direct drainage of road and car park surfaces to impervious drainage systems.

- Allowing stormwater runoff from roads and car parks to discharge as sheet flow across adjacent grassed surfaces prior to entering the formal drainage system.

(f)   Appropriate street landscaping

- Appropriate selection of street trees to reduce leaf fall and the resulting stormwater passage of organic matter into receiving waters.

- Appropriate selection of street trees, especially in cyclone prone areas, to reduce the discharge of organic matter into receiving waters.

11.4   Stormwater treatment techniques

11.4.1   General

The National Water Quality Management Strategy (ARMCANZ & ANZECC, 2000) has established the following hierarchy for the management of stormwater quality:

1. Retain, restore, or rehabilitate valuable ecosystems
2. Source control through non-structural measures
3. Source control through structural measures
4. Regional in-stream treatment measures

This hierarchy places a priority on the establishment of non-structural source controls over the adoption of structural source controls and regional in-stream treatment measures. However, this does **not** mean that structural or regional controls should not be adopted until after all non-structural source controls have been implemented.

For most urban land uses it is unlikely that non-structural source controls alone will achieve the required water quality objectives, thus for the time being, structural stormwater treatment measures will remain an integral part of the urban landscape.

11.4.2   Non-structural source control

Non-structural source controls principally rely on pollution prevention through the use of community education and appropriate work place management practices.

(a)   State government

State government activities that support stormwater pollution prevention include:

- Active enforcement of the *Environmental Protection Act 1994* and its associated Environmental Protection Policies.

- Provision of a leadership role in the adoption of best management practice stormwater management on State works.

- Provision of a leadership role in the adoption of best management practice *Erosion and Sediment Control* on State construction projects.

- Cooperation with local governments, industry groups and professional bodies in the development of Best Management Practice guidelines.

- Encouragement and support for best management practice technology transfer between local and interstate authorities, industry groups and professional bodies.

- Training of emergency services in the operation of shut-down systems on stormwater treatment devices and pollution containment systems.

- Promotion of litter and nutrient reduction campaigns.

(b)   Local government

Detailed discussion on municipal activities is provided in the Urban Stormwater Quality Planning Guidelines (DERM, 2010).

Municipal activities that support stormwater pollution prevention include:

- Active enforcement of the *Environmental Protection Act, 1994* and its associated Environmental Protection Policies.

- Adoption of best management practice stormwater management and treatment measures on council works.

- Adoption of best management practice erosion and sediment control on council construction projects.

- Development of Urban Stormwater Quality Management Plans and Healthy Waters Plans in accordance with the requirements of the *Environmental Protection Act, 1994*.

- Establishment of local Water Quality Objectives (WQOs) and waterway Environmental Values.

- Development and adoption of local planning policies and development regulations that support best management practice stormwater management, including WSUD.

- Development and implementation of Asset Maintenance Plans (section 2.9.5) for existing stormwater treatment systems.

- Development and promotion of public education activities.

- Investigation and control of illegal dumping, including the disposal of garden waste within parks and along waterways corridors.

- Integration of the planning and management of sewer overflows into catchment management planning.

- Establishment of best management practice plant and equipment maintenance and wash-down facilities within council depots, including covered parking and chemical storage, stormwater runoff isolation areas in association with oil and grit traps.

- Promotion of an environmentally sensitive septic tank replacement program.

- Establishment of local laws on the containment of dog faeces within public areas.

- Staff training and awareness programs.

- Town planning protection of natural waterways and the rehabilitation of hard-lined drainage channels.

Street cleaning:

- Adoption of only suction-type sweeper units.

- Focusing street cleaning on critical areas including: the central business district (night sweeping), commercial areas, public activity and sporting areas, areas of high building activity, and residential streets following heavy winds.

- Reviewing night time parking restriction in high risk areas to improve the efficiency of street sweeping activities, otherwise, conduct street sweeping during periods of daytime parking restriction.

- Adoption of wind-proof community litter bins.

Domestic waste collection:

- Establishing green waste collection facilities.

- Introducing community clean-up and waste collection prior to cyclone season.

Management of road shoulders:

Unsealed road shoulders can represent a significant source of coarse and fine sediments, and metals.

- Sealing or otherwise stabilising road shoulders wherever practical.

- Considering the use of grassed structural soils in areas where normal grassing or single coat bitumen seal is not practical.

(c)   Business unit operations

Business activities that support stormwater pollution prevention include:

- Active enforcement of the *Environmental Protection Act 1994* and its associated Environmental Protection Policies within business activities.

- Adopting alternative water sensitive practices relating to start-of-day and end-of-day wash-down and clean-up procedures, with preference given to portable sweeper/suction devices.

- Establishment of best management practice plant and equipment maintenance and wash-down facilities, including covered parking and chemical storage, stormwater runoff isolation areas in association with oil and grit traps.

- Development of industry-based stormwater management codes of practice for industries such as: fast food outlets, roof/house cleaning, carpet cleaning, mobile dog washing, building industry, construction industry, driveway/pavement stencilling and saltwater pool maintenance.

- Site and local areas litter and debris collection.

- Staff training and awareness programs.

(d)   Public activities and education Programs

Community attitudes and values greatly influence the selection and ranking of environmental values. Activities such as the *Clean-Up Australia* campaign have greatly influenced community's attitudes to gross pollutants, even though these attitudes may not necessarily be ecologically based. Stormwater managers have a responsibility to both identify and understand community values, and to assist in the education and guidance of the community in a manner that will assist in the protection of both existing and anticipated future environmental values.

The benefits of community participation are outlined in ARMCANZ & ANZECC (2000).

Community education programs can incorporate the following features:

- Development of fact sheets, brochures, booklets and videos.

- Stormwater guidelines for the community (e.g. Environmental Protection Authority SA, 1997).

- Development of community-based guidelines on: the management of green waste, operation of fresh and salt water swimming pools, use of garden fertilisers and pesticides, car washing, building site erosion and sediment control.

- Promotion of environmentally sensitive septic tank maintenance and adoption of replacement systems.

- Community awareness campaigns such as the Adopt A Lake, Adopt A Waterway schemes, Clean-Up Australia campaign, and stormwater lintel messages (e.g. PROTECT OUR WATERWAYS – FLOWS TO CREEK).

Community education programs should reinforce key issues such as:

- Potential impacts of increased impervious surface area within residential homes on the quantity and quality of stormwater runoff and downstream ecosystems.

- Impact of waste organic matter (e.g. garden waste and grass cuttings) on stream water quality.

- The importance of a complete vegetative cover over all earth surfaces, including the footpath regions of road reserves.

- The ecological importance of maintaining high quality stormwater runoff.

- The financial cost to ratepayers for litter collection within residential areas (e.g. street sweeping), parks and waterways (e.g. construction and maintenance of various end-of-pipe stormwater treatment systems).

11.4.3   Structural controls

Treatment levels for structural controls can be graded into primary, secondary and tertiary (polishing) treatment in a manner aligned with the classifications adopted for wastewater treatment. The various treatment levels are outlined in tables 11.4.1 to 11.4.3.

Table 11.4.1 – Primary treatment classifications

Mechanics	Description	Target pollutants
Screening	Physical separation of solids from a liquid passing through a screen. Promoted by fine screen opening.	Solids, litter, debris
Isolation	Physical entrapment of substances. Promoted by storage volume and flow control barrier.	Hydrocarbons, chemicals, toxicants
Separation	Physical isolation of two collective substances by an impervious barrier. Promoted by low turbulence and depth of surface skimmer.	Hydrocarbons, floating litter and debris
Settling (sedimentation and oil separation)	The separation or layering of substances according to their relative mass. Promoted by low turbulence.	Solids, BOD, pathogens, particulates, COD, nutrients (particulates), hydrocarbons (if skimmer is used)

Table 11.4.2 – Secondary treatment classifications

Mechanics	Description	Target pollutants
Adsorption	The attachment of a substance to the surface of a solid by virtue of forces arising from molecular attraction. Promoted by high soil Al, Fe; high soil organics; circumneutral pH.	Dissolved P, nutrients (N, P), metals, synthetic organics
Filtration	Physical retention of particles on surface of the filter or within the filter medium. Promoted by fine, dense herbaceous plants; or fine, homogeneous porous medium (e.g. sand with uniform grain size)	Solids, BOD, pathogens, particulates, COD, nutrients (particulates)
Flocculation	The process by which suspended colloidal or very fine particles coalesce and agglomerate into well-defined hydrated floccules of sufficient size to settle rapidly. Promoted by flocculating agent and low turbulence.	Turbidity, fine sediments, metals, nutrients (particulates)
Infiltration	The movement of water into the soil. Promoted by highly porous soils.	As for filtration

Table 11.4.3 – Tertiary treatment classifications

Mechanics	Description	Target pollutants
Aeration	The combining of oxygen from the atmosphere with the water body.	Oxygen demanding substances, process resulting in low DO water
Biological decomposition	To separate or resolve into constituent parts or elements through biological activity. Promoted by high plant surface area and soil organics.	BOD, COD, organic matter, petroleum hydrocarbons, synthetic organics
Biological uptake	A process by which materials are absorbed and incorporated into organic matter. Promoted by high plant activity and surface area; soil pH (variable depending on substance).	Nutrients (P, N) and metals
Disinfection	Destruction of pathogens (e.g. bacteria) by ultra-violet light. Promoted by high light, shallow water depth, low turbidity.	Pathogens
Fixation	Fixation of atmospheric nitrogen to ammonia by microbial organisms and chemical fixation.	Nitrogen
Nitrification and denitrification	Microbial conversion of ammonia to nitrite, then to nitrate; and the reduction of nitrate or nitrite to nitrogen gas, in the absence of oxygen. Promoted by variable oxygen levels, circumneutral pH, low toxicants, water temperature > 15°C.	Nitrogen
Oxidation	The combination of oxygen with a substance. Promoted by aerobic conditions.	COD, nutrients (N, P), petroleum hydrocarbons, synthetic organics
Solar treatment (volatilisation and disinfection)	Destruction of pathogens (e.g. bacteria) and the breakdown of hydrocarbons by ultra-violet light. Promoted by high light, shallow water depth, low turbidity.	Pathogens, hydrocarbons
Volatilisation	The conversion of a chemical substance from a liquid or solid to a gaseous or vapour state. Promoted by high temperature and air movement.	Mercury, volatile petroleum hydrocarbons and synthetic organics

Unless otherwise specified, stormwater treatment systems should be designed for a discharge of 0.5 times the peak 63% AEP (1 in 1 year) discharge throughout Queensland. This discharge is often reported as the 1 in 3-month design discharge; however, the relationship between the peak 1 in 1 year discharge and the 1 in 3-month design discharge varies throughout Queensland, which can result in confusion as to an appropriate design storm. To avoid such confusion, design standards should focus solely on specifying design discharges relative to the peak 63% AEP (1 in 1 year) discharge and avoid reference to the 1 in 3-month discharge.

11.5   Selection of treatment techniques

Various design procedures may be followed depending on the existence of local or regional Stormwater Management Plans (SMPs) or Water Quality Objectives (WQOs). Six design procedures are presented in Table 11.5.1 representing the typical range of procedures adopted around Australia.

Within Queensland, stormwater quality investigations are usually based on Methodology 2. Design Methodologies 5 and 6 should only be used where it is not considered practical to establish a numerical water quality model of the site, such as minor council road works.

There are numerous combinations of treatment measures that can theoretically (i.e. through computer modelling) satisfy the required WQOs. The treatment train should not be selected with the sole aim of achieving these WQOs, but should show due consideration towards the following factors:

- an understanding of the pollutant runoff characteristics of different land uses
- an understanding of the benefits of different treatment techniques
- an understanding of the needs of different downstream waters and ecosystems.

To provide the best and most robust stormwater treatment system, all waterway catchments should ideally incorporate an array of primary, secondary and tertiary treatment measures. The selection of treatment techniques should give appropriate consideration to numerous factors including the following:

- aims of a relevant Stormwater Management Plan
- site and catchment conditions and target pollutants
- cost-effectiveness of each treatment method or device, including life cycle costs
- capability of the asset manager to operate and maintain the treatment measure
- opportunities provided by the particular land use, land area and soil type
- the potential for stormwater management infrastructure to enhance urban amenity.

Wherever practical, the design of stormwater treatment measures must consider the following issues on a site-by-site basis:

- topography – land area and slope
- soil type – erosivity, porosity, depth to bedrock
- groundwater issues – watertable level, risk of contamination, rising salinity problems
- ecology issues – habitats, vegetation, waterways etc.
- land ownership
- cultural heritage considerations
- provision of services (power and water)
- flooding issues
- public safety
- maintenance equipment and access
- proximity of residents
- potential odour problems
- visual impacts
- possible long-term site contamination
- health problems relating to mosquitoes and vermin.

*Note: The information presented in tables 11.5.2 to 11.5.8 has been provided as a **general guide only**. The suitability of a treatment system to a particular catchment location is governed by numerous factors that can significantly alter its benefit, function, efficiency, suitability and ranking. The information presented within these tables should not supersede site specific investigation, modelling or design.*

Table 11.5.1 – Various stormwater quality design procedures

Design methodology	Design steps
Methodology 1. Design to achieve a given water quality	• Identify environmental values and obtain from relevant regulating authority the required WQOs • Model post-development stormwater runoff conditions
Methodology 2. Design to achieve a % reduction in pollutant runoff	• Obtain local or regional pollution reduction standard from relevant regulating authority • Model post-development stormwater runoff conditions
Methodology 3. Design to achieve pre-development catchment discharge quality	• Obtain existing water quality conditions (if available) • Model stormwater runoff conditions for pre and post development
Methodology 4. Design to protect a given downstream environment	• Consult the Queensland Government and local authority for terms of reference for study • Identify environmental values • Conduct scientific research/study (if study has not already occurred) • Develop water quality model of research area, including downstream waters • Establish WQOs to protect, restore or secure (as appropriate) the environmental values as defined by the National Water Quality Management Strategy • Where appropriate, develop an Urban Stormwater Quality Management Plan (section 2.6) • Adopt Methodology 1 or 2 (above) depending on the type of WQOs determined from scientific study
Methodology 5. Selecting BMPs based on a given critical receiving water	• Use tables 11.05.2 and 11.05.3 to select a short list of techniques most appropriate for the given catchment area and soil porosity—these tables should not be viewed as prescriptive, but as a general guide • Use tables 11.05.4 to 11.05.6 to further refine this short list of treatment measures based on the receiving waters
Methodology 6. Selecting BMPs based on land use activities	• Use tables 11.05.2 and 11.05.3 to select a short list of techniques most appropriate for the given catchment area and soil porosity—these tables should not be viewed as prescriptive, but as a general guide • Use tables 11.05.7 and 11.05.8 to further refine this short list of treatment measures based on the land use category

Table 11.5.2 – Typical optimum catchment area for treatment techniques

	Catchment area (hectares)								
	1	**2**	**5**	**10**	**20**	**50**	**100**	**500**	**>500**
Primary Treatment									
Grate inlet screens	Yes								
Side entry pit traps	Yes								
Litter baskets	Yes	Yes	Yes	Yes	Yes	Yes			
Outlet litter cages	Yes	Yes	Yes	Yes	Yes	Yes			
Release nets	Yes	Yes	Yes	Yes	Yes	Yes			
Enclosed GPTs	Yes	Yes	Yes	Yes	Yes	Yes			
Oil and grit separators	Yes								
Open GPTs				Yes	Yes	Yes	Yes		
Trash racks			Yes	Yes	Yes	Yes	Yes	Yes	Yes
Floating booms							Yes	Yes	Yes
Floating GPT							Yes	Yes	Yes
Sedimentation basins				Yes	Yes	Yes	Yes	Yes	
Roadside pollution containment systems	Yes								
Secondary Treatment									
Porous pavements	Yes								
Filter strips	Yes								
Grass swales	Yes	Yes	?						
Bio-retention cells	Yes	Yes							
Infiltration trenches	Yes	?	?						
Infiltration basins		Yes	Yes	?	?				
Exfiltration systems	Yes	?	?						
Extended detention			Yes	Yes	Yes	Yes	Yes		
Sand filters	Yes	?							
Filter basins		?	Yes	Yes	Yes	Yes	Yes		
Mini wetlands	?	Yes	Yes	Yes	?				
Tertiary Treatment									
Ponds			?	Yes	Yes	Yes	Yes	Yes	Yes
Constructed wetlands				?	Yes	Yes	Yes	Yes	Yes

Notes (Table 11.5.2):

'Yes' means that the technique is likely to be suitable for catchment area

? means the suitability of the technique to the given catchment area is questionable.

Table 11.5.3 – Optimum soil permeability for various treatment systems [1]

	Sand 210 mm/hr	Loam sand 54 mm/hr	Sandy loam 26 mm/hr	Loam 13.2 mm/hr	Silt loam 6.9 mm/hr	Clay loam 2.3 mm/hr	Clay 0.5 mm/hr
Secondary Treatment							
Porous pavements	Yes	Yes	Yes	Yes	Use subsoil drainage		
Filter strips	Yes	Yes	Yes	Yes	Yes	[2]	
Grass swales	Yes	Yes	Yes	Yes	Use subsoil drainage		
Bio-retention cells	Use swale or infiltration			Yes	Yes	Yes	Yes
Infiltration trenches	Yes	Yes	Yes	Yes	■	■	■
Infiltration basins	Yes	Yes	Yes	Yes	■	■	■
Exfiltration systems	Yes	Yes	Yes	Yes	■	■	■
Extended detention	Yes	Yes	Yes	Yes	Yes	[3]	
Sand filters	Use infiltration systems			Yes	Yes	Yes	Yes
Filter basins	Use infiltration systems			Yes	Yes	Yes	Yes
Mini wetlands	■	[4]		Yes	Yes	Yes	Yes
Tertiary Treatment							
Ponds	■	[4]		Yes	Yes	Yes	Yes
Wetlands	■	[4]		Yes	Yes	Yes	Yes

Notes (Table 11.5.3):

[1]   Consideration should be given to the likely long-term soil permeability (i.e. during normal operating conditions) taking appropriate consideration of long-term maintenance and possibly ongoing replacement of the filtration system.

[2]   Water quality benefits decrease with decreasing soil porosity. Likely maintenance mowing problems can exist during wet season due to soil saturation.

[3]   Runoff detention benefits still achieved, but water quality benefits are reduced due to limited infiltration.

[4]   Possible plant sustainability problems due to low soil water levels. Consider design of sub-surface flow wetland or melaleuca wetland.

Table 11.5.4 – Typical pollutant removal efficiencies of treatment systems

Benefit ranking: L = Low benefit M = Medium benefit H = High benefit	Litter and debris	Coarse sediment	Fine sediment	Nutrients Dissolved	Particulate	Metals	Hydrocarbons	Oxygen demanding substances	Pathogens
Primary Treatment									
Grate inlet screens	L								
Side entry pit traps	L-M							L	
Litter baskets	L-M							L	
Outlet litter cages	H							M-H	
Release nets	H							L-M	
Enclosed GPTs	H	H	L		L	L	L	L-M	
Oil and grit separators	L	H	L		L	L	M		
Open GPTs	M-H	M-H	L		L	L		L-M	
Trash racks [1]	M-H	L						L-M	
Floating booms	L						M		
Floating GPT	L						L		
Sedimentation basins	L	M-H	L-M		L	L	L	L	L
Roadside pollution containment system [2]							H		
Street sweeping	H-M	M						L	
Secondary Treatment									
Porous pavements		L-M	L-M	L	M	M	M	L	M
Filter strips	L	M	L-M	L	L-M	L-M	L	L	M
Grass swales	L	M-H	L-M	L	L-M	L-M	L	[3]	L-M
Bio-retention cells	L	M-H	M	L	M	M	L-M	L-M	L-M
Infiltration trenches	L	M-H	M	L-M	M	M	M	M	M
Infiltration basins	L-M	H	M-H	L-M	M-H	M-H	M-H	M-H	M-H
Exfiltration systems	H	H	M-H	L	M-H	M-H	M-H	M-H	M
Extended detention	M	H	L-M	L	M	M	L	L	M
Sand filters	L-M	H	M	L	M-H	M-H	M-H	M-H	M
Filter basins	L-M	H	M-H	L	M-H	M-H	M-H	M-H	M-H
Mini wetlands	M	H	L	M	M-H	M	L	L	L
Tertiary Treatment									
Ponds	M-H	H	M	M	M-H	M	L	L	[4]
Constructed wetlands	M-H	H	M	H	H	M-H	M	L	[4]

Notes (Table 11.5.4):

[1]   Benefits depend on maintenance frequency and whether trapped organics remain wet or dry between storms.

[2]   Target pollutant is usually hydrocarbons and liquid chemicals released from spills and traffic accidents.

[3]   Grass swales can generate large volumes of cut grass that, if not collected, can be washed into receiving waters.

[4]   Pathogen level may be increased due to resident bird life.

Table 11.5.5 – Potential ecological impact of pollutants on waterways [1]

		Ephemeral creeks	Perennial creeks	Freshwater rivers	Lakes	Natural wetlands	Canals	Saline rivers and estuaries	Bays	Ocean
Gross Pollutants	**Litter [2]**	L	L	L	M	M	M	M-H	H	H
	Organic Debris	H	H	M	M-H	L	H	L-M	L	L
	Coarse Sediment	H	H	M	L-M	H	L-M	L	L	L
Fine Sediment		M-H	H	H	H	M	M	M	M	L
Nutrients		H	H	H	H	M	H	H	H	L
Metals		H	H	H	H	M	H	H	H	H
Hydrocarbons [3]		H	H	H	H	H	M-H	M	M	M
Oxygen Demanding Substances		H	H	M	H	M	M-H	L-M	L-M	L
Pathogens [4]		M	M	H	H	L	M-H	M	M	M

Notes (Table 11.5.5):

[1]   Potential impacts are highly variable and site specific. Values provided are only a guide to typical ecological impacts. Consideration has not been given to safety, social or economic impacts.

[2]   Litter impact on coastal water is high due to potential digestion of litter (plastic bags, etc.) by large marine life.

[3]   Reference is made to minor quantities of hydrocarbons, not to major oil or fuel spills.

[4]   Reference is made to potential impact on human and aquatic health.

Table 11.5.6 – Typical benefits of treatment systems on waterways

Benefit Ranking: L = Low benefit M = Medium benefit H = High benefit	Ephemeral creeks	Perennial creeks	Freshwater rivers	Lakes	Natural wetlands	Canals	Saline rivers and estuaries	Bays	Ocean
Primary Treatment									
Grate inlet screens	L	L	L	L	L	L	L	M	M
Side entry pit traps	L	L	L	L	L	M	M	M	M
Litter baskets	L	L	L	L	L	M	M	M	M
Outlet litter cages	L	L	L	L	L	M	M	M	M
Release nets	L	L	L	L	L	M	M	M	M
Enclosed GPTs	M-H	M-H	M	M-H	M	M-H	M	M	M
Oil and grit separators	M-H	M-H	M	M	M	L-M	L	L	L
Open GPTs	M-H	M-H	M	M-H	L-M	M-H	M	M	M
Trash racks	M	M-H	L	M	L	M	M	M	M
Floating booms	■	■	L	M	■	■	M	M	M
Floating GPT	■	■	L	L	■	■	M	M	M
Sedimentation basins	M-H	M-H	L-M	L	L-M	L	L-M	L	L
Roadside pollution containment systems	H	H	H	H	H	H	H	H	H
Street sweeping	M	M	L	L	L-M	M	H	H	H
Secondary Treatment									
Porous pavements	M-H	M-H	M	M	L	M	M	L	L
Filter strips	M	M	L-M	L	L	M	M	M	M
Grass swales	M-H	M-H	M	M	L	M	M	M	L-M
Bio-retention cells	H	H	M-H	H	M	M-H	M-H	M-H	M-H
Infiltration trenches	H	H	H	H	H	H	H	H	H
Infiltration basins	H	H	H	H	H	H	H	H	H
Exfiltration systems	H	H	H	H	H	H	H	H	H
Extended detention	M-H	M-H	L-M	L-M	L-M	L-M	L-M	L	L
Sand filters	H	H	M-H	H	H	M-H	M-H	M	M
Filter basins	H	H	H	H	H	H	H	M	M
Mini wetlands	H	H	M	M-H	L-M	M	M	M	M
Tertiary Treatment									
Ponds	H	H	H	H	H	H	H	M-H	M
Constructed wetlands	H	H	H	H	H	H	H	M-H	M

Table 11.5.7 – Suitability of treatment systems to various land uses [1]

	Car parks	Shopping centres	Sporting and public areas	Rural roads	Residential roads	Arterial roads	Coastal roads	Road intersections
Primary Treatment								
Grate inlet screens	L	L	L			L	L	
Side entry pit traps	M	H	M		L	L	M	L
Litter baskets	M	H	M			L	M	L
Outlet litter cages		M	M			L	H	
Release nets		M	M			L	H	
Enclosed GPTs	M	H	H		L	M	H	
Roadside pollution containment systems						H	M	M
Secondary Treatment								
Porous pavements	H	H	H					
Filter strips	M	M	M	M	L	L	L	L
Grass swales	M	M	M	M	M	M	M	M
Bio-retention cells	H	H	H		H	H	M	H
Infiltration trenches	H	H	H			H	M	M
Infiltration basins		H						
Exfiltration systems	H	H						
Extended detention	L	M						
Sand filters	H	H	H					
Filter basins		H						
Mini wetlands	M	M	M	M	M	M	M	M

Note (Table 11.5.7):

[1]   This table is provided as a general guide only. The suitability of a treatment system to a particular land use is governed by numerous factors which can significantly alter its function, efficiency and overall suitability. The indicated suitability of a treatment system to a given land use has been based on the likely integration of the system into the land use, and the ability of the system to capture and/or treat the type of pollutants most commonly associated with the land use activity. The information presented in these tables should **not** supersede site specific investigation, modelling or design.

Table 11.5.8 – Suitability of treatment systems to various land uses [1]

	Parks and Open Space	Rural Residential	Low Density Residential	Medium Density Residential	High Density Residential	Commercial Areas	Industrial Areas	Central Business Districts
Primary Treatment								
Grate inlet screens	■	■	■	■	■	L	L	L
Side entry pit traps	■	■	■	■	L	L	M	M
Litter baskets	■	■	■	L	L	M	M	M
Outlet litter cages	■	■	■	M	M	M	M	M
Release nets	■	■	■	M	M	M	M	M
Enclosed GPTs	■	■	■	M	M	M	M	M
Open GPTs	■	■	■	M	M	M	M	M
Roadside pollution containment systems	■	■	■	L	L	■	M	■
Secondary Treatment								
Porous pavements	L	L	M	L	L	L	L	■
Filter strips	L	M	M	L	L	L	L	■
Grass swales	M	M	M	M	M	M	M	M
Bio-retention cells	M	M	H	H	H	H	H	H
Infiltration trenches	■	■	■	H	H	H	H	H
Infiltration basins	■	■	■	H	■	H	H	■
Exfiltration systems	■	■	■	■	H	H	H	H
Extended detention	■	■	■	M	M	M	M	■
Sand filters	■	■	■	■	H	H	H	■
Filter basins	■	■	■	■	H	H	H	■
Mini wetlands	M	M	M	M	M	M	M	M

Note (Table 11.5.8):

[1] This table is provided as a general guide only. The suitability of a treatment system to a particular land use is governed by numerous factors which can significantly alter its function, efficiency and overall suitability. The indicated suitability of a treatment system to a given land use has been based on the likely integration of the system into the land use, and the ability of the system to capture and/or treat the type of pollutants most commonly associated with the land use activity. The information presented in these tables should **not** supersede site specific investigation, modelling or design.

11.6   Stormwater management plans

11.6.1   General

Stormwater Management Plans set out the proposed management of activities within a catchment which are likely to:

- alter stormwater runoff volume, rate, duration and frequency
- adversely affect the environmental values of receiving waters, including groundwater, downstream water and coastal water.

Stormwater Management Plans should identify the proposed protection, treatment and management of the identified waterways and water bodies within the catchment with respect to:

- the impact of stormwater on these features and their ecosystems
- the impact of these features on stormwater.

There are two forms of Stormwater Management Plans:

- Urban Stormwater Quality Management Plans
- Site-based Stormwater Management Plans

A brief description of catchment-based Urban Stormwater Quality Management Plans is presented in section 2.6 of this Manual.

11.6.2   Site-based stormwater management plans

Site-based Stormwater Management Plans (SMPs) are normally developed by an applicant as part of a development application for approval under the *Sustainable Planning Act 2009*.

These plans are prepared for urban developments to provide a set of guidelines to control soil erosion and pollutant transport during the construction phase, post-construction maintenance phase, and ongoing operational phases of the development.

Site-based SMPs must be consistent with any current catchment-based Urban Stormwater Quality Management Plan or Catchment Management Plan.

Three separate SMPs may need to be produced dealing with the Construction Phase, Post-construction Maintenance Phase, and the Operational Phase. The Operational Phase SMP is prepared as a guide for the long-term management and maintenance of the various stormwater management systems installed within the development.

(a)   Erosion and sediment control

Erosion and sediment control requirements during the construction phase of a development may be incorporated into, or otherwise linked to, the site-based SMP.

Sediment, in all its forms—sand, silt, clay, earth and mud—constitutes a pollutant if it exceeds an undesirable or environmentally damaging concentration or deposition quantity. Thus the removal and transportation of sediment by rainfall and stormwater runoff must be appropriately managed.

Erosion and Sediment Control Plans (ESCPs) should be developed for the building/construction phase of all significant land disturbances, whether or not the land disturbance is regulated by an external authority. A significant land disturbance would include any soil disturbances that occur over a period exceeding 24 hours during which time rainfall is possible, or any soil disturbance

exceeding 100 square metres that will remain unprotected during a period in which rainfall is possible.

The degree of complexity and detail provided in the ESCP shall depend on the extent and complexity of the works, and the potential for environmental harm resulting from the works. Guidelines on the preparation of ESCPs can be found in the latest version of the Engineers Australia ESC guidelines, or other regional or local government ESC guideline.

(b)   Site-based Stormwater Management Plans – Construction phase

Issues to be addressed within the SMP (Construction) include:

- site constraints
- water quality objectives and indicators
- statement of who is responsible for each task
- erosion and sediment control (submission of plan, review and monitoring of plan, amendment and re-submission of plan)
- management of trapped fauna
- management/protection of permanent stormwater treatment systems during the construction phase
- management of changing weather and site conditions
- treatment of acid sulfate and dispersive soils
- on-site chemical and fuel storage
- waste and litter receptors
- maintenance procedures (ESC and waste)
- clean-up of pollution spills/deposition (on-site and off-site)
- clean-up after storm events
- water quality monitoring requirements (location and testing)
- site inspection and monitoring
- procedures for recording and addressing external complaints
- incident reporting
- reporting procedures.

The management of wastes and chemicals on building and construction sites should incorporate appropriate storage, handling and disposal of any material or pollutant that may be incorporated into, or transported by, stormwater, whether or not such material was initially displaced by rain, flowing water, wind or mechanically by construction practices. These management practices should ensure:

- chemicals and fuels stored, handled and disposed of in a manner which ensures that no pollutants are discharged to stormwater
- wastewater from construction and building activities (e.g. equipment clean-up and site wash-down water, and cooling water from material cutting) to be contained on-site
- litter and building waste to be adequately stored and disposed.

(c)   Site-based Stormwater Management Plans – Post-construction maintenance phase

Where necessary, a separate SMP may be required for the post-construction maintenance phase that addresses the following issues:

- water quality objectives and indicators
- statement of who is responsible for each task
- management and maintenance of permanent stormwater treatment systems
- maintenance of retained ESC measures
- water quality monitoring requirements (location and testing)
- site inspection and monitoring
- incident reporting
- reporting procedures.

Specifically, instructions should be provided on the necessary site conditions that should exist prior to commissioning each stormwater treatment system.

(d)   Site-based Stormwater Management Plans – Operational phase

Where necessary, a separate SMP may be required for the operational phase that addresses the following issues:

- water quality objectives and indicators
- short-term (e.g. weekly, monthly, annual, biannual) maintenance requirements for various stormwater treatment systems
- long-term (e.g. 5, 10, 20-year plan) maintenance requirements and procedures for various stormwater treatment systems
- water quality monitoring requirements (location and testing).

12. Safety aspects

12.1 General

Urban waterways and stormwater drainage systems can represent a significant safety risk to pedestrians during storms and times of flood. These risks may be associated with a person deliberately entering a drain or waterway, or as a result of an accidental slip or fall. The following chapter discusses these safety risks. For information on safety risks to vehicles, refer to section 7.4.

Undesirable interaction with stormwater structures can cause various physical and psychological injuries, including:

- cuts and bruises
- psychological trauma
- permanent bodily injuries
- fatal injuries.

These injuries may result from:

- physical harm caused by a person being swept into, or against, a solid object, including a safety screen
- short or long-term psychological trauma caused by a stormwater-related event
- drowning.

The hazard experienced by people wading through water can be influenced by such factors as:

- lighting conditions
- depth and velocity
- the rate of rise in water level
- evenness and firmness of the ground surface
- presence of large, hidden or sharp debris
- presence of depressions, potholes, fences, open drains, and pits with displaced lids
- water quality, especially if contaminated with hazardous or toxic chemicals
- the person's footwear and clothing (e.g. the drag caused by loose or heavy clothing).

Brown, turbid floodwaters can hide unexpected dangers, such as open stormwater pits, even if the overtopping water depth is only a few centimetres.

The risks associated with stormwater structures can be managed satisfactorily through the use of appropriate management techniques, including:

(i) Designing stormwater flow conditions so that the waters do not represent a risk of injury or death (first priority).

(ii) Designing or shaping the land and environments in and around a stormwater structure in a manner that minimises the risk of a person accidentally falling or slipping into a stormwater system (high priority).

(iii) Designing or shaping the land and environments in and around a stormwater structure in a manner that discourages a person/child from wishing to enter or play in the water during a storm or flood event (high priority).

(iv)   Designing stormwater systems to allow a person to readily exit a drain or waterway before being swept into a pipe inlet, culvert or other unsafe waters.

(v)   Designing long culverts such to maximise occurrence of those flow conditions where floodwaters pass as free surface flow through the culverts (i.e. adequate headroom exists to reduce the risk of drowning).

(vi)   Erecting warning signs to alert people of potential danger.

(vii)   Erecting external barriers (e.g. fencing) to limit the entry of people into stormwater structures.

(viii)   Erecting inlet screens to prevent entry of people into a pipe or culvert.

(ix)   Public education programs.

Point (v) above refers only to 'long culverts'. Unless otherwise agreed, a long culvert should be taken as a culvert with a flow travel time in excess of 10 seconds. There is no recognised design criteria for the provision of adequate head room; however, 300 mm freeboard between the water surface and the culvert obvert may be considered a minimum. Whether this freeboard should exist during the design minor storm, major storm, or an intermediate flow rate, will depend on the assessed safety risk. It should be noted that such design criteria would not render a culvert safe, but would at best only reduce the potential safety risk.

Safety risks to persons wading through water have been documented by Engineers Australia (2010) and various floodplain management guidelines. The risks typically relate to the depth*velocity product (d.V) of the water and the height*mass product (H.M) of the person.

Table 12.1.1 outlines the safety risks categories recommended by Engineers Australia (2010).

Table 12.1.1 – Flow hazard regimes for infants, children and adults

d.V (m^2/s)	Infants, small children (H.M < 25 m.kg) and frail persons	Children (H.M = 25 to 50 m.kg)	Adults (H.M > 50 m.kg)
0	Safe	Safe	Safe
0 to 0.4	Extreme hazard	Low hazard [1]	Low hazard [1]
0.4 to 0.6		Significant hazard [3]	
0.6 to 0.8		Extreme hazard	Moderate hazard [2]
0.8 to 1.2			Significant hazard [3]
> 1.2			Extreme hazard

Notes (Table 12.1.1):

[1]   Stability uncompromised for persons within ideal laboratory conditions for a maximum flow depth of 0.5 m for children and 1.2 m for adults, and a maximum velocity of 3.0 m/s for very shallow flow.

[2]   Flow conditions dangerous to some. Considered the working limit for trained safety workers and experienced well-equipped adults.

[3]   Flow conditions dangerous to most. Considered to be the upper limit of stability observed during most investigations.

12.2 Risk assessment

In circumstances where a local government or stormwater designer considers it necessary to develop a risk assessment profile or develop a risk management strategy, reference should be made to Australian Standard AS 4360. An example of how AS 4360 could be applied to stormwater systems is presented below.

Local governments can use a 'Risk Assessment Matrix' (refer to Table 12.2.3) to identify those areas of a stormwater network that warrant highest priority for a safety review. The likelihood of contact has traditionally been based on a Contact Class system (e.g. U.S. Bureau of Reclamation, 1987). A modified alternative to the USBR system is presented in Table 12.3.1. Local governments may choose to adopt an alternative Contact Classification based on local conditions.

The main elements of the risk management process presented in AS 4360 may be summarised as:

(a)   Communicate and consult

- Identify the current asset manager.

- Identify those sections/departments of the community and local government that need to be consulted as part of the risk assessment and risk management process.

- Communicate with the local community about the risks and the adopted management procedures. This communication may include the use of warning signs adjacent high-risk stormwater systems.

(b)   Establish the context

- Define the parameters within which risks must be managed. Where possible, relate the risk management objectives to the overall stormwater and organisational objectives. Also define the criteria against which these risks are to be evaluated.

(c)   Identify risks

- Identify where, when, why and how stormwater related risks may occur.

- Define what risks are under the control of the organisation and/or asset manager.

- Identify existing processes, devices and practices used to manage the risks and assess their strengths and weaknesses.

- Identify current best management practice options.

- Identify the consequences and likelihood of an event (refer to Glossary, Chapter 13) for definition of terms).

Tables 12.2.1 and 12.2.2 provide examples of possible stormwater related likelihood and consequence scales.

(d)   Analyse risks

Identify and evaluate existing controls. Determine consequences and likelihood. Develop an appropriate risk assessment table, such as the example shown in Table 12.2.3

Table 12.2.1 – Example of likelihood scale

Level	Descriptor	Description
A	Almost certain	The event will occur on an annual basis
B	Likely	The event has occurred several times in recorded history
C	Possible	The event is likely to occur once in 50 years
D	Unlikely	The event has occurred once before
E	Rare	The event has not occurred locally, but has occurred elsewhere
F	Very rare	Never known to have occurred
G	Almost incredible	Theoretically possible, but not expected to occur

Table 12.2.2 – Example of consequence scale

Level	Descriptor	Description
V	Major	Fatal injuries
IV	Significant	Permanent injury or psychological trauma
III	Moderate	Broken bone or open flesh wound
II	Minor	Cuts and bruises
I	Very minor	Wet clothes or mild scare or mild trauma

Table 12.2.3 – Example of a risk assessment matrix

Likelihood scale	Consequence scale				
	I	II	III	IV	V
A	Medium	High	High	Very high	Very high
B	Medium	Medium	High	High	Very high
C	Low	Medium	High	High	High
D	Low	Low	Medium	Medium	High
E	Low	Low	Medium	Medium	High

(e)   Evaluate risks

Compare the level of risk with the established assessment criteria and identify risks that require a change in management/operational procedures.

(f)   Treat risks

Identify options for managing the risks. Consider the potential benefits and adverse outcomes of proposed risk management options.

Develop an 'Action Plan'. Identify the appropriate asset manager and how the action plan should integrate into the operational procedures of the asset manager.

The adopted risk management measures should be determined from an assessment of all relevant issues including, the likelihood of an incident, the consequences of an incident, cost, aesthetics,

legal risk to the utility owner, and community expectations. Consideration must also be given to potential risks to maintenance and rescue/emergency personnel.

It is acknowledged that safety risks are unlikely to be eliminated from all stormwater systems; however, all reasonable and practicable measures should be taken to minimise identified risks.

(g)   Monitor and review

Monitor the implementation of the action plan. Develop incident reporting procedures. Where appropriate, photograph the stormwater system during runoff/flood conditions. Review the risks, adopted action plan, and the management procedures as necessary.

12.3   Example safety risk ranking system

The following safety risk ranking systems were initially developed for Brisbane City Council.

Table 12.3.1 – Contact classification

Contact class	Location description
Class A1	Within or immediately adjacent to a school or childcare centre, including the adjoining road reserve, where access to the inlet or upstream channel is unrestricted and/or there is reasonable risk that a person could be swept towards the inlet during a storm or flood.
Class A2	As above except access to potentially dangerous waters or inlets is unlikely and/or severely restricted. If full exclusion fencing exists, then Class D applies.
Class B1	Within 100 metres of an existing or future urban residential area or public gathering area such as a park, shopping centre, entertainment or sporting facility. Access to the inlet or upstream channel is unrestricted and/or there is reasonable risk that a person could be swept towards the inlet during a storm or flood.
Class B2	As above except access to potentially dangerous waters or inlets is unlikely and/or severely restricted. If full exclusion fencing exists, then Class D applies.
Class C1	More than 100 metres from a school, park, childcare centre, or existing or future urban residential area. Access to the inlet or upstream channel is unrestricted and/or there is reasonable risk that a person could be swept towards the inlet during a storm or flood.
Class C2	As above except access to potentially dangerous waters or inlets is unlikely and/or severely restricted. If full exclusion fencing exists, then Class D applies.
Class D	Within an area surrounded by heavily trafficked arterial roads, childproof fencing or is otherwise considered inaccessible (legally or illegally) to the general public.

Table 12.3.2 – Potential safety risks associated with a conduit flowing full [1]

Ranking	Hydraulic capacity of pipe/culvert
Low (score 1)	*Intent: Site conditions mean that the conduit is **unlikely** to be flowing full or near-full during an incident, thus a trapped person is likely to be able to breath freely while passing through the conduit.* **Typical assessment condition:** Conduit capacity is greater than or equal to 2% AEP.
Medium (score 3)	*Intent: Site conditions mean that there is a reasonable chance that the conduit will not be flowing full or near-full during an incident.* **Typical assessment condition:** Conduit capacity is greater than 39% AEP (2 year ARI) and less than 2% AEP.
High (score 6)	*Intent: Site conditions mean that the conduit is **likely** to be flowing full or near-full during an incident resulting in a trapped person being submerged while passing through the conduit.* **Typical assessment condition:** Conduit capacity is less than or equal to 1 in 2 years ARI.

Note (Table 12.3.2):

[1]   This table may be considered to represent likely consequences if it is accepted that the risk of drowning is directly related to the probability of the pipe or culvert flowing full.

Table 12.3.3 – Potential safety risks associated with the length of conduit

Ranking	Hydraulic capacity of pipe/culvert
Low (score 1)	*Intent: Site conditions mean that a trapped person is likely to take less than 10 seconds to pass through the conduit.* **Typical assessment condition:** Conduit length is less than or equal to 30 metres.
Medium (score 3)	*Intent: Site conditions mean that a trapped person is likely to take around 10 to 20 seconds to pass through the conduit.* **Typical assessment condition:** Conduit length is greater than 30 and less than 60 metres.
High (score 6)	*Intent: Site conditions mean that a trapped person is likely to take more than 20 seconds to pass through the conduit.* **Typical assessment condition:** Conduit length is greater than or equal to 60 metres.

Table 12.3.4 – Potential safety risks associated with flow conditions within a stormwater pipe or culvert [1]

Ranking	System	Example flow and conduit conditions [2]
Low (score 1)	Any system	• Conduit containing junction pits without a change-of-direction.
Medium (score 3)	Any system	• Low velocity (< 1 m/s) stormwater pipe or culvert containing an internal split-flow chamber, change-of-direction junction pit, or drop pit less than 1 m in fall. • Conduit discharging to a low-risk (non-impact) energy dissipater.
High (score 6)	Culvert	• High velocity (> 1 m/s) culvert containing an internal split-flow chamber. • Culvert discharging to a high-risk energy dissipater (e.g. impact structure).
	Stormwater pipe	• Conduit containing a covered deepwater chamber (e.g. enclosed GPT) accessible to a person being washed through the pipe. • High velocity (> 1 m/s) conduit containing a split-flow chamber, change-of-direction junction pit, or drop pit. • Conduit discharging to a high-risk energy dissipater (e.g. impact structure).

Notes (Table 12.3.4):

[1] The intent of this table is to assign the risks associated with a person experiencing serious head or otherwise fatal injuries while passing through a conduit or as a result of the person exiting through an associated energy dissipater.

[2] In this case it is assumed that an outlet energy dissipater is part of the internal flow conditions of the conduit.

Table 12.3.5 – Potential safety risks associated with flow conditions at the outlet of a stormwater pipe or culvert [1]

Ranking	Flow conditions
Low (score 1)	*Intent: Outlet conditions represent a low risk of drowning or head injuries and favourable conditions for rescue.* **Examples:** • Open water, low velocity channel with good egress.
Medium (score 3)	*Intent: Outlet conditions represent a low risk of drowning or head injuries, but egress may be difficult and/or questionable conditions exist for rescue.* **Examples:** • Deep and/or high-velocity water downstream of the inlet. • Outlet to turbid waters more than 1200 mm deep. • Outlet to waters not in clear sight of a potential rescuer.
High (score 6)	*Intent: Outlet conditions represent a high risk of drowning or head injuries, and/or egress may be difficult, and/or adverse conditions exist for rescue.* **Examples:** • Outlet to waters with no egress. • Outlet to waters not accessible to a potential rescuer.

Note (Table 12.3.5):

[1]   The intent of this table is to assign the risks associated with a person (after passing through a conduit) entering dangerous water where the person may drown or be placed in a position where rescue will be difficult.

Table 12.3.6 – Risk ranking matrix

Contact class	Cumulative consequence score [1]				
	1 – 4	**5 – 9**	**10 – 14**	**15 – 19**	**20 – 24**
	Risk ranking [1]				
A1	13	7	4	2	1
A2, B1	16	12	8	5	3
B2, C1	18	14	11	9	6
C2, D	20	19	17	15	10

Notes (Table 12.3.6):

[1]   The cumulative consequence score is determined by summing the values from Tables 12.3.2 to 12.3.5.

[2]   A ranking of 1 represents the highest risk. A ranking of 20 represents the lowest risk. Within each ranking allotment a secondary ranking may be achieved based on the cumulative consequence score (i.e. a ranking of 8 with a Cumulative Consequence Score of 14 would rank higher than a ranking of 8 with a score of 10).

Table 12.3.7 provides a guide to the selection of site mitigation works of various safety risks at culverts and stormwater inlets.

Table 12.3.7 – A guide to mitigation options for various safety risks [1]

Class	Cumulative consequence score				
	1 – 4	**5 – 9**	**10 – 14**	**15 – 19**	**20 – 24**
A1	Case 2	Case 1			
A2, B1		Case 3			
B2, C1	Case 6	Case 5	Case 4		
C2, D	Case 9	Case 8	Case 7		

Notes (Table 12.3.7):

[1]   This table is presented as a guide only. Indicated outcomes do not represent mandatory practices. Each site must be assessed on a case-by-case basis to determine appropriate treatment measures.

Case 1   It would be difficult to justify not placing appropriate inlet screens on culverts or stormwater inlets with openings exceeding a diameter of 300 mm or a slot width of 125 mm.

Case 2   Typically stormwater inlets would be treated as per Cases 1/3. Culverts would unlikely require inlet screens, however, the need for fencing of the upstream channel should be assessed on a case-by-case basis based on the risk of drowning or injuries caused by accidental falls.

Case 3   Appropriate inlet screens should ideally be placed on stormwater inlets with openings exceeding a diameter of 300 mm or a slot width of 125 mm wherever practical. However, the appropriate management of debris blockage issues and associated flooding risks could prevent

the use of inlet screen in some cases. In such cases, consider the use of fencing (i.e. exclusion fencing or barrier fencing) to restrict access to the upstream channel.

The placement of inlet screen on culverts and large stormwater pipes should generally only be considered as a last option if access to the upstream channel cannot be severely restricted and/or the safety risk associated with the culvert/pipe cannot be otherwise mitigated.

Warning signs should be used liberally.

Case 4   Treatment is likely to be similar to Case 3. The focus should generally be on restricting access to dangerous waters instead of the using inlet screens that could potentially cause flooding problems.

Case 5   The focus should generally be on restricting access to dangerous waters and the use of warning signs.

Case 6   Placement of warning signs where appropriate.

Case 7   In Class C2 areas, treatment is likely to be similar to Case 4. In Class D areas, exclusion fencing should generally be used in preference to barrier fencing.

Case 8   In Class C2 areas, treatment is likely to be similar to Case 5. In Class D areas the choice of fencing should be appropriate for the assessed safety risk.

Case 9   In Class C2 areas, treatment is likely to be similar to Case 6. In Class D areas the choice of fencing should be appropriate for the assessed safety risk.

Note: In all case, the use of inlet or outlet screens on hydraulic structures such as stormwater pipe and culverts should always be assessed on a case by case basis. It is important to acknowledge and assess both the beneficial and adverse consequences of such screens, including the potential impacts on wildlife migration. Table 12.5.1 lists some of the potential beneficial and adverse consequences of inlet and outlet screens.

12.4   Safety fencing

Safety fencing may be divided into the following three categories:

- *Childproof fencing*: used to prevent access by children that are not old enough to properly assess the safety risks.

- *Exclusion fencing*: used to exclude people (children and adults) from an area.

- *Barrier fencing*: primarily a warning system used to minimise the risk of a person accidentally falling onto a hard surface or into dangerous waters.

It is not considered to be in the general interest of the community to design urban drainage channels in a manner that will require the need for safety fencing. The first preference should always be to design stormwater channels with gentle grassed slopes or heavily vegetated banks that will minimise the risk of a person accidentally falling into dangerous waters, and allow a child or injured person easy egress.

As a general rule, urban waterways, wetlands and lakes are not fenced unless there is an edging treatment (e.g. high, steep, slippery bank) that is reasonably expected to result in a person accidentally falling into dangerous waters, or falling more than 1500 mm onto a hard surface.

(a)   Childproof fencing

The need for childproof fencing must be assessed on a case-by-case basis based on a risk assessment analysis.

Examples of stormwater systems likely to represent reasonably foreseeable danger are presented in Table 12.4.1

Table 12.4.1 – Stormwater systems likely to represent a reasonably foreseeable danger

Systems likely to represent a reasonably foreseeable danger	Systems not likely to represent a reasonably foreseeable danger
• Stormwater systems not clearly visible • Overland flow paths • Temporary sedimentation basin associated with a construction activity • Small stormwater treatment systems, wetlands or ponds not directly associated with a watercourse and as such their existence may not be obvious to a visitor to the area • Deep-chamber gross pollutant traps installed within a stormwater pipe system • Stormwater pipes or culverts discharging to a high-risk energy dissipater	• Vegetated drainage channels or waterways • Large wetlands or lakes likely to be well known to residents (i.e. clearly visible) • Stormwater systems located within the road reserve of a busy arterial road that would itself represent an obvious safety risk to children • Most stormwater detention/retention basins

(b)   Exclusion fencing

Even if the risks are considered reasonably foreseeable by the general public, if a stormwater drain, inlet, culvert or other structure is considered to represent a significant safety risk, then all reasonable and practicable measures should be taken to minimise or otherwise manage these risks. In cases where the safety risks exist for both adults and children, then the use of exclusion fencing may be required.

Unfortunately, most exclusion fencing can be scaled, crossed or damaged by a determined person; therefore, the type and height of fencing used should be appropriate to the expected risks and the desired functions of the fence.

(c)   Barrier fencing

Barrier fencing is not primarily designed to exclude access by a person. Rather its focus is on providing a visual warning of danger and preventing accidental falls.

If the edge treatment of a stormwater device represents a risk of a person falling more than 1000 mm, then appropriate barrier fencing may be required. Such fencing should be designed to sustain the imposed actions specified in AS 1170.1.

12.5   Inlet and outlet screens

12.5.1   General

The use of inlet or outlet screens on hydraulic structures such as stormwater pipe and culverts should not become an automatic function, but should always be assessed on a case by case basis. It is important to acknowledge and assess both the beneficial and adverse consequences of such screens. Ultimately their use should be based on an appropriate risk assessment process. Table 12.5.1 lists some of the potential beneficial and adverse consequences of inlet and outlet screens.

Table 12.5.1 – Potential beneficial and adverse consequences of inlet and outlet screens

Potential beneficial consequences	Potential adverse consequences
• Persons prevented from being swept into or through the structure (inlet screen) • Persons prevented from entering (exploring) stormwater structures that contain potential dangers, such as deep chambers or a GPT • Benefits to upstream land owners as a result of persons being prevented from entering private residence or commercial areas through the stormwater system (outlet screen)	• Interference with terrestrial movement • Interference with aquatic movement • Safety risks to persons trapped on the screen by hydraulic pressure (inlet screen) • Safety risks to persons swept into the stormwater pipe and unable to exit the pipe due to the screen (outlet screen) • Local flooding resulting from debris blockages and safety risks associated with flow bypassing

12.5.2   Use of outlet screens

Outlet screens should not be used in circumstances where a person could enter, or be swept into, the upstream pipe network during a period of pipe flow.

In appropriate circumstances, consideration should be given to the placement of outlet screens on stormwater pipes of 600 mm diameter or greater that contain an accessible, enclosed, deepwater chamber (e.g. gross pollutant trap) in Contact Class zones A, B and C (refer to Table 12.3.1).

Grates should only be installed on the stormwater outlets provided:

• Possible debris loadings from upstream catchment are adequately assessed.

• The consequences of system failure (e.g. property damages and safety hazards) resulting from debris blockage of the screen have been investigated and addressed to the satisfaction of the local government.

• All upstream inlets and access chambers are secured against unauthorised entry.

12.5.3   Site conditions where barrier fencing or inlet/outlet screens may not be appropriate

Site conditions where the installation of barrier fencing or inlet/outlet screens may not be appropriate include the following examples:

• Where the over-all risk to human life as a result of the installation is judged to be greater than if the device was not installed.

• Where there is an unacceptable risk of trapping wildlife (aquatic or terrestrial) or causing disruption to an essential wildlife corridor.

- Where debris blockage of the fence or screen will cause or increase floor level flooding. In such case the degree of blockage must be commensurate with the existing and likely future catchment conditions (refer to Engineers Australia, 2012 for guidance on assessing debris availability).

Each site **must** be assessed on a case-by-case basis.

12.5.4 Inlet screen arrangement

Figures 12.1 to 12.6 provide examples of possible inlet screen arrangements. These examples have been provided for the purpose of assisting designers in developing design concepts. Design of the screen should not necessarily be limited to the examples provided e.g. various tower inlets chambers (not shown) have been used within detention basins.
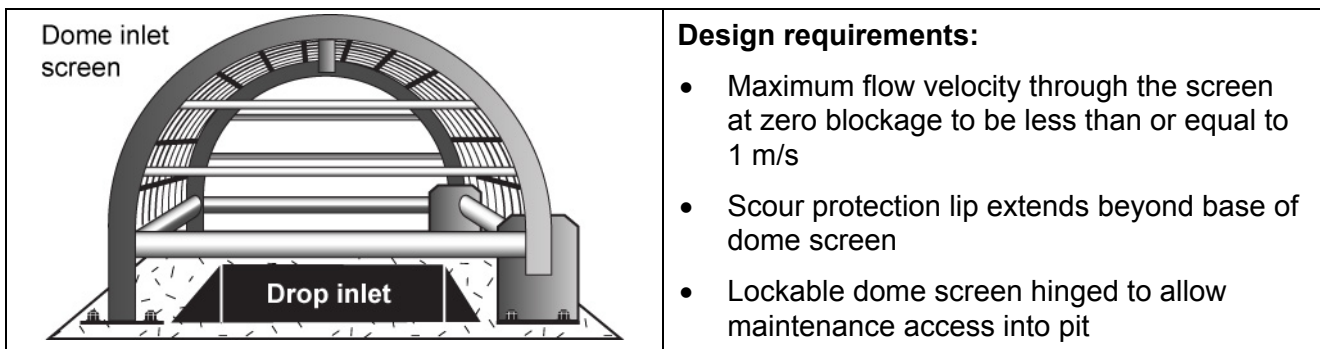
	Design requirements: • Maximum flow velocity through the screen at zero blockage to be less than or equal to 1 m/s • Scour protection lip extends beyond base of dome screen • Lockable dome screen hinged to allow maintenance access into pit
Figure 12.1 – Dome inlet screen	
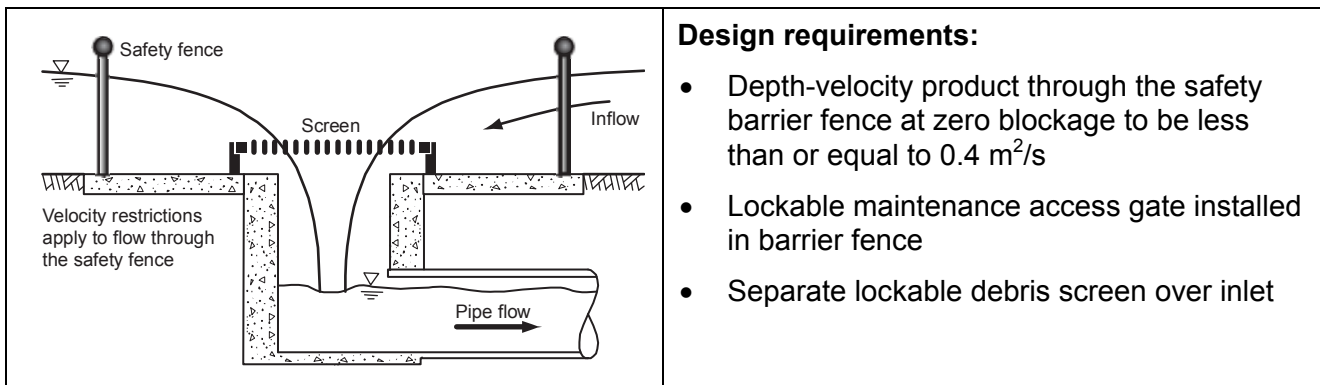
	Design requirements: • Depth-velocity product through the safety barrier fence at zero blockage to be less than or equal to 0.4 m^2/s • Lockable maintenance access gate installed in barrier fence • Separate lockable debris screen over inlet
Figure 12.2 – Major inlet structure	
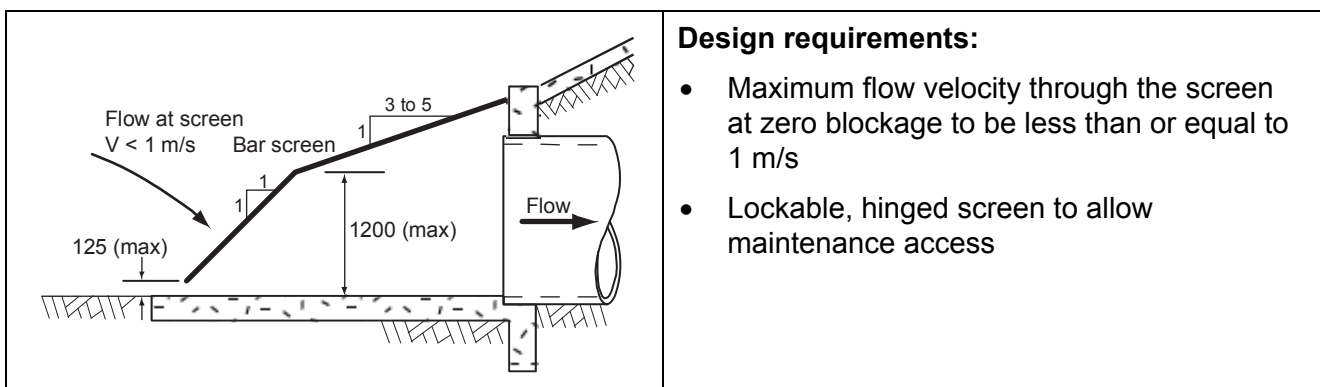
	Design requirements: • Maximum flow velocity through the screen at zero blockage to be less than or equal to 1 m/s • Lockable, hinged screen to allow maintenance access
Figure 12.3 – Hinged inlet bar screen	
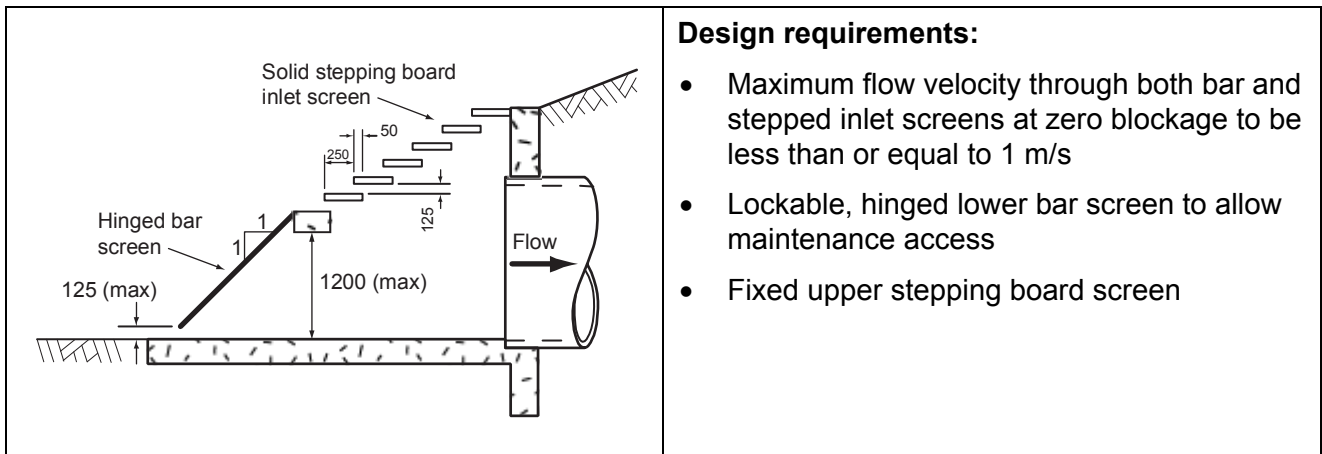
Design requirements:

- Maximum flow velocity through both bar and stepped inlet screens at zero blockage to be less than or equal to 1 m/s

- Lockable, hinged lower bar screen to allow maintenance access

- Fixed upper stepping board screen

Figure 12.4 – Bar screen with upper stepping board inlet screen
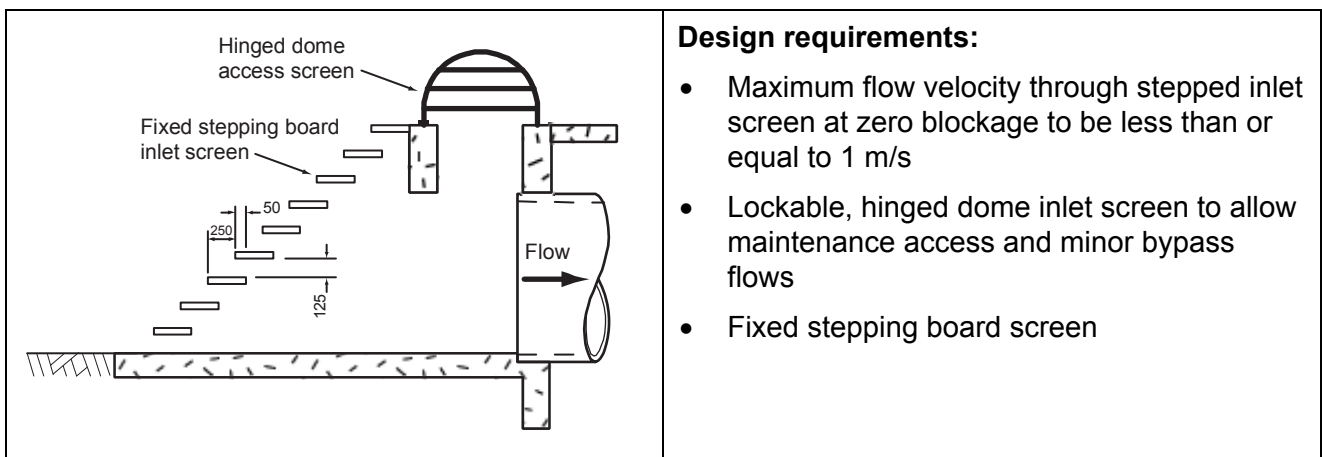
Design requirements:

- Maximum flow velocity through stepped inlet screen at zero blockage to be less than or equal to 1 m/s

- Lockable, hinged dome inlet screen to allow maintenance access and minor bypass flows

- Fixed stepping board screen

Figure 12.5 – Fixed stepping board inlet screen
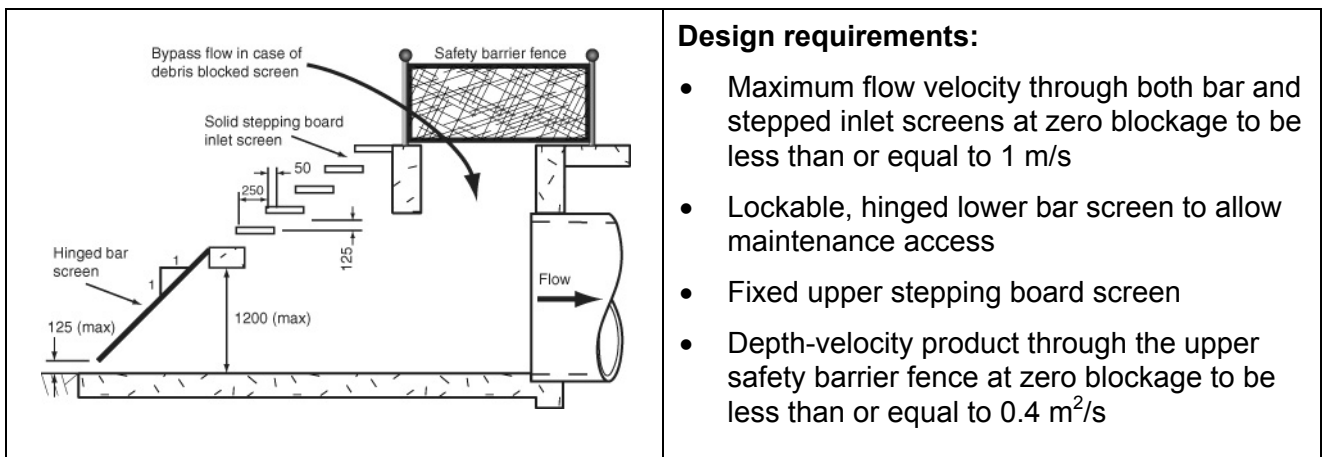
Design requirements:

- Maximum flow velocity through both bar and stepped inlet screens at zero blockage to be less than or equal to 1 m/s

- Lockable, hinged lower bar screen to allow maintenance access

- Fixed upper stepping board screen

- Depth-velocity product through the upper safety barrier fence at zero blockage to be less than or equal to 0.4 m²/s

Figure 12.6 – Alternative major inlet structure

12.5.5 Design guidelines for inlet and outlet screens

The following guidelines apply to the design of inlet and outlet screens.

- Any culvert or pipeline provided with outlet protection shall be provided with inlet protection.

- Maximum 'clear' spacing of vertical bars is provided in Table 12.5.2.

Table 12.5.2 – Maximum clear spacing of vertical bars

Location	Maximum clear spacing of vertical bars
Child-proof barrier fencing	100 mm
Inlet safety screens	125 mm
Outlet screens	150 mm

- Maximum clear spacing of the screen above ground level is 125 mm and 150 mm for inlet and outlet screens respectively.

- Maximum inclined spacing of horizontal support bars is 600 mm. This maximum bar spacing aims to allow a trapped person to climb up the screen to safety. Note; a 'minimum' spacing of 1 m applies to safety fencing (not inlet/outlet screens) to prevent a child climbing over the fence.

- In waterways containing permanent water, either still or flowing, aquatic passage requirements must be considered.

- The 'net' open surface area of the inlet rack should be at least three times the cross-sectional area of the pipe/culvert inlet.

- Recommended slope of 'inlet' safety screens is provided in Table 12.5.3. A variable slope inlet screen may be developed as shown in Figure 12.7.

Table 12.5.3 – Recommended slope of inlet safety screens

Height of screen	Maximum recommended slope to the horizontal
Less than or equal to 375 mm with an approach velocity no greater than 1 m/s	Vertical
Greater than 375 but less than 1200 mm with an approach velocity no greater than 1 m/s	Slope of 1:1 (45°)
Less than or equal to 600 mm with an approach velocity greater than 1 m/s	Slope of 1:1 (45°)
Greater than 600 but less than 1200 mm with an approach velocity greater than 1 m/s	Slope of 1:3 to 1:5 (V:H)
Greater than 1200 mm	Slope of 1:3 to 1:5 (V:H)
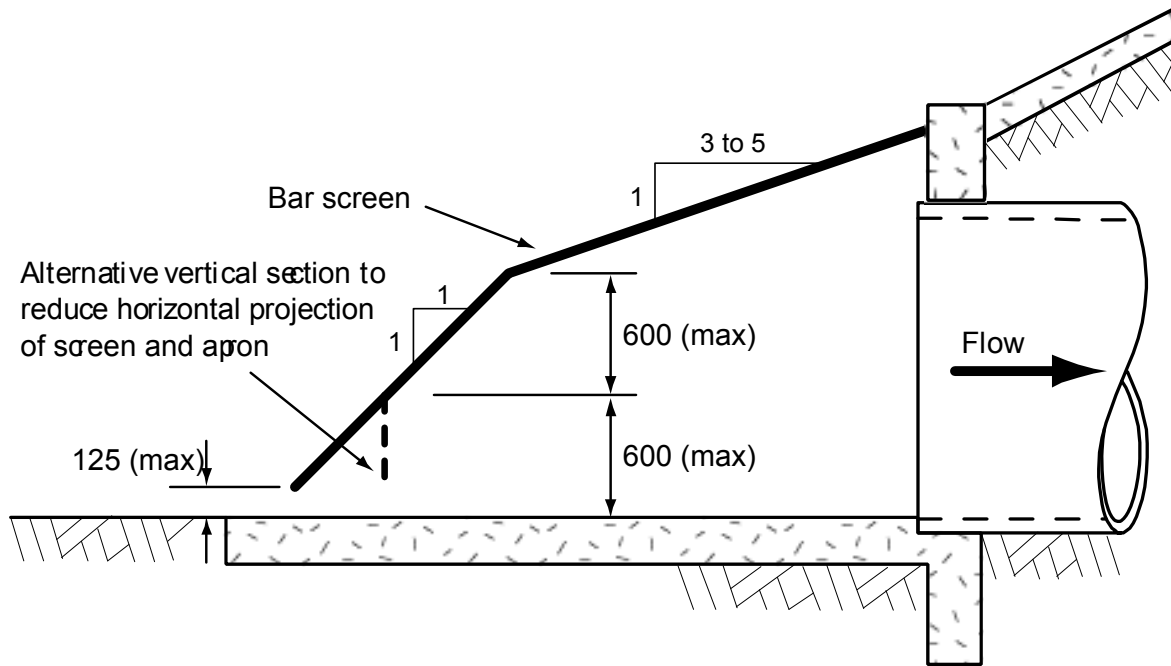
Figure 12.7 – Design requirements for inlet screens

- Where practical, inlet screens should be located and designed such that flow velocities through the 'clean' screen will be low enough (typically equal to or less than 1 m/s) to allow a person to egress from the structure.

- If the inlet consists of a transition that significantly contracts stormwater flow into the pipe or culvert, then where practical, the screen should be located upstream of the resulting drop in water surface profile.

- Where practical, the vertical downward component of water velocity at an inlet grate should be minimised.

- Appropriate access must be provided to the screen for dry weather maintenance including the removal of debris.

- All screens should be appropriately designed to allow cleaning even when fully blocked.

- Inlet screens/racks should have a removable feature to permit access for cleaning inside the pipe/culvert.

- Outlet screens shall not be used in circumstances where a person could either enter, or be swept into, the upstream pipe network.

- Outlet screens on pipe/box units up to 1800 mm in width should be designed such that the full width of the outfall pipe/box can be accessed for periodic maintenance.

- All screens should be secured with tamper-proof bolts or a locking device.

- Outlet screens should be structurally designed to break away under the conditions of 50% blockage (or lower if needed to prevent undesirable backwater flooding) during the pipe's design storm event.

- Local governments may consider allowing the use of top hinged outlet screens installed at an angle of say, 10 degrees to the vertical, to restrict unauthorised entry, but allow the passage of water during significant debris blockage.

12.5.6 Hydraulics of inlet screens

Hydraulic analysis of screened inlets should consider the following:

- Upstream flood levels should ideally be based on 100% blockage of the screen during the designated major storm, but only if 100% blockage is considered likely.

- The designated minor storm should be analysed assuming at least 50% debris blockage of any inlet screen, unless such blockage is considered unlikely to occur.

Case A:

Head loss through a clean or partially blocked screen that is located well upstream of the pipe/culvert inlet (i.e. not bolted directly to the face of the headwall, or inside the pipe) may be assessed based on equation 12.1. In such cases, pipe entry losses need to be considered separately.

$$\Delta H = K_t^* \, (V_n^2/2g) \tag{12.1}$$

where:

$$K_t^* = 2.45 \, A_r - A_r^2 \tag{12.2}$$

and:

ΔH = Head (energy) loss (m)

K_t^* = head loss coefficient based on velocity through screen

A_r = Area ratio = $A_b/A = 1 - A_n/A$

A_b = Blockage surface area of the screen bars (including debris blockage where applicable) (m^2)

A_n = Net flow area through screen (i.e. excluding bars and debris)

A = Gross flow area at the screen, $A = A_b + A_n$ (m^2)

V_n = flow velocity through the partially blocked screen (m/s)

g = acceleration due to gravity (9.80 m/s^2)

Technical note 12.5.1

Equation 12.2 has been developed from the original recommendations of US Bureau of Reclamation (1987). The coefficients are generally higher than those recommended by Miller (1990), but are considered to be more realistic for heavily blocked screens. The coefficients provided by equation 12.2 for a clean screen (say $A_r < 0.2$) are comparable with those recommended by Miller.

Case B:

If the screen is bolted directly to the face of the inlet headwall, or where flow immediately downstream of the screen is confined within a conduit with a cross-sectional area approximately equal to the gross area of the screen, then the head loss for the screen may be determined from equation 12.3. In such cases, the pipe entry loss cannot be considered separately, and thus the head loss of the screened pipe inlet must be taken as the greater of the screen loss or the pipe entry loss.

$$\Delta H = K_t \, (V_o^2/2g) \tag{12.3}$$

where:

$$K_t = (2.45A_r - A_r^2)/(1 - A_r)^2 \tag{12.4}$$

and:

K_t = head loss coefficient based on downstream flow velocity

V_o = flow velocity downstream of screen (m/s)

Case A:

Screen upstream of pipe/culvert inlet (Figure 12.8)

Energy loss (ΔH) consists of screen loss plus pipe entry loss.
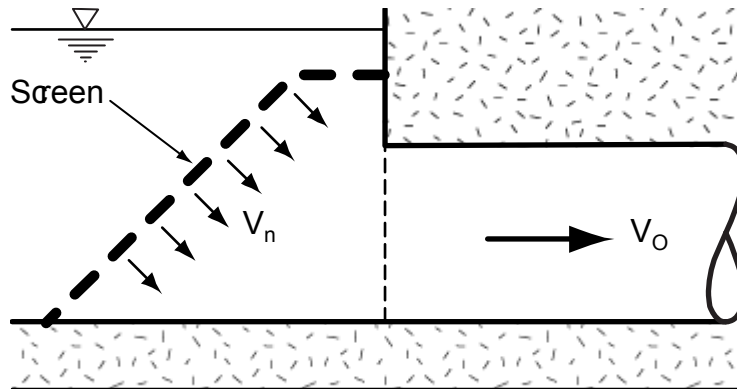
Figure 12.8 – Inlet screen mounted away from the inlet

$$\Delta H = K_t^* .(V_n^2/2g) + K_e .(V_o^2/2g) \qquad (12.5)$$

Where K_e is typically 0.5 for square edged inlet, and K_t^* is determined from equation 12.2.

Case B:

Screen located at pipe/culvert inlet (Figure 12.9)

Energy loss (ΔH) is the greater of the screen loss and the pipe entry loss.
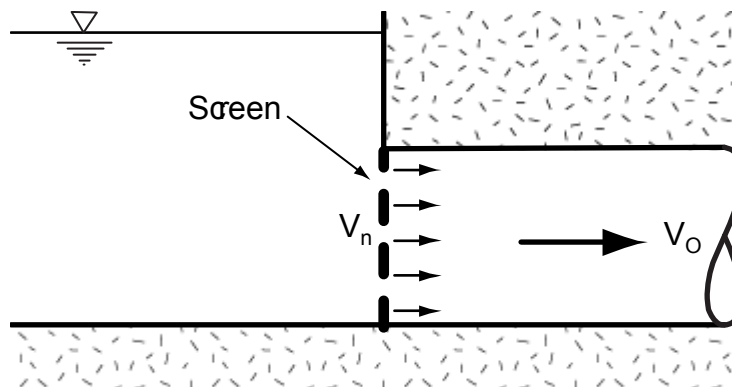
Figure 12.9 – Inlet screen mounted close to the inlet

ΔH = the greater of:

(i) $K_t .(V_o^2/2g)$ or, $\qquad (12.6)$

(ii) $K_e .(V_o^2/2g)$ $\qquad (12.7)$

Where K_e is typically 0.5 for square edged inlet, and K_t is determined from equation 12.4. It is noted that is this case $K_t .(V_o^2/2g) = K_t^*.(V_n^2/2g)$.

12.5.7   Hydraulics of outlet screens

Head loss through a fixed outlet screen that is located downstream of a pipe/culvert outlet headwall may be estimated using the procedures presented for Cases C to E below.

If partial debris blockage of the screen is considered possible, then an appropriate adjustment should be made to the assumed net area through the screen.

Case C:

Clean screen, i.e. ($A_r < 0.2$) located at pipe/culvert outlet (Figure 12.10)

Energy loss (ΔH) consists of screen loss (based on drag force equation) plus normal exit loss.
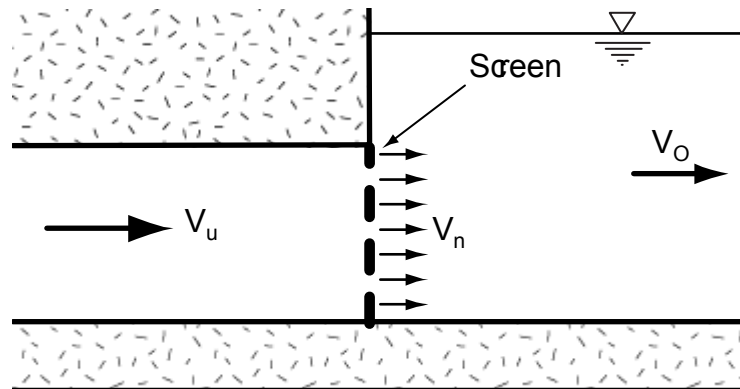
Figure 12.10 – Outlet screen with minimal blockage

$$\Delta H = C_d.A_r.(V_u^2/2g) + K_{exit} [(V_u^2/2g) - (V_o^2/2g)] \tag{12.8}$$

Where the drag coefficient C_d is typically 1.5 for round bars and 1.9 for rectangular bars.

Exit loss coefficient, K_{exit} may be determined from section 5.16.9 based on side wall conditions at the exit, typically $K_{exit} = 0.7$ in such cases due to the effects of a flush channel bed in expansion of the outlet jet.

Case D:

Partially blocked screen, i.e. ($A_r > 0.2$) located at pipe/culvert outlet (Figure 12.11)

Energy loss (ΔH) consists of screen loss (heavily blocked screen) plus an exit loss component.
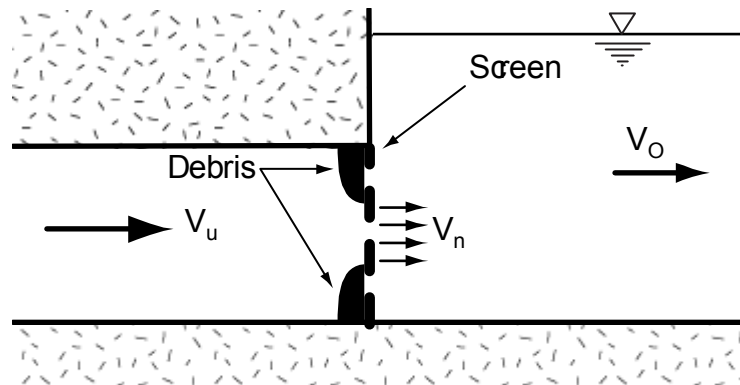
Figure 12.11 – Partially blocked outlet screen

$$\Delta H = K_t^*.(V_n^2/2g) + [(V_u^2/2g) - (V_o^2/2g)] \tag{12.9}$$

Case E:

Screen located well downstream of pipe/culvert outlet (Figure 12.12)

Energy loss (ΔH) consists of pipe/culvert exit loss plus screen loss.
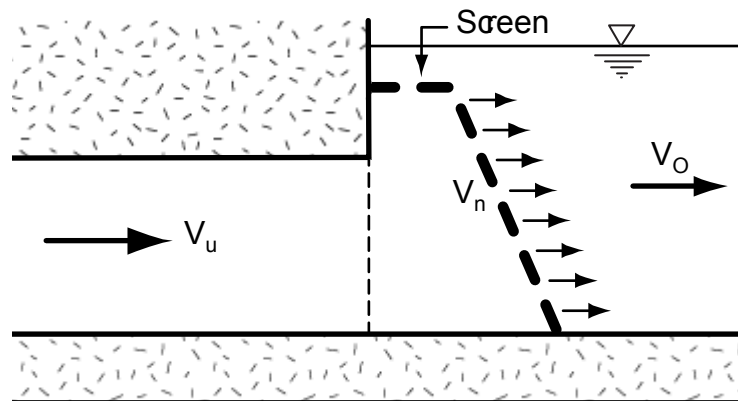
Figure 12.12 – Outlet screen mounted away from the outlet

$$\Delta H = K_t^* . (V_n^2/2g) + K_{exit} [(V_u^2/2g) - (V_o^2/2g)] \qquad (12.10)$$

Exit loss coefficient, K_{exit} may be determined from section 5.16.9 based on side wall conditions at the exit, typically $K_{exit} = 0.7$ in such cases due to the effects of a flush channel bed in expansion of the outlet jet.

where:

V_u = Average flow velocity upstream of outlet (m/s)
C_d = Drag coefficient
K_e = Entry loss coefficient
K_{entry} = Exit loss coefficient

12.5.8   Dome inlet screens

Dome inlet screens can be used in a variety of circumstances. As with all screens, the potential safety risks to pedestrians and vehicle users need to be considered in all cases. In the case of dome inlet screens, the primary concern is for those circumstances where a child could be swept up against the screen **and** the water level could rise above the child's head. This section deals with the design of dome inlet screens in circumstances where:

• A person wading through the water could approach the screen without appreciating the rapid change in flow velocity and/or pressure adjacent the screen, and the person would not be able to climb onto the screen to escape danger or otherwise remove themselves from the water, and where the flow depth adjacent to the partially blocked screen could exceed 300 mm.

• A person (with special reference to a child) could be swept off their feet (i.e. d*V > 0.4) and carried towards the screen either unconscious or in such a physical condition that prevented them from climbing onto the screen to escape danger or otherwise remove themselves from the water, and where the flow depth adjacent to the partially blocked screen could exceed 300 mm.

• Any other circumstances where a person could be held against the screen by water pressure potentially causing injury or death to the person.

Domed inlet safety screens constructed over a horizontal field inlet have two critical dimensions, those being:

- maximum clear bar spacing of 125 mm (Figure 12.13)
- minimum screen width and/or internal lip width (W) to achieve a screen through-velocity of 1 m/s.
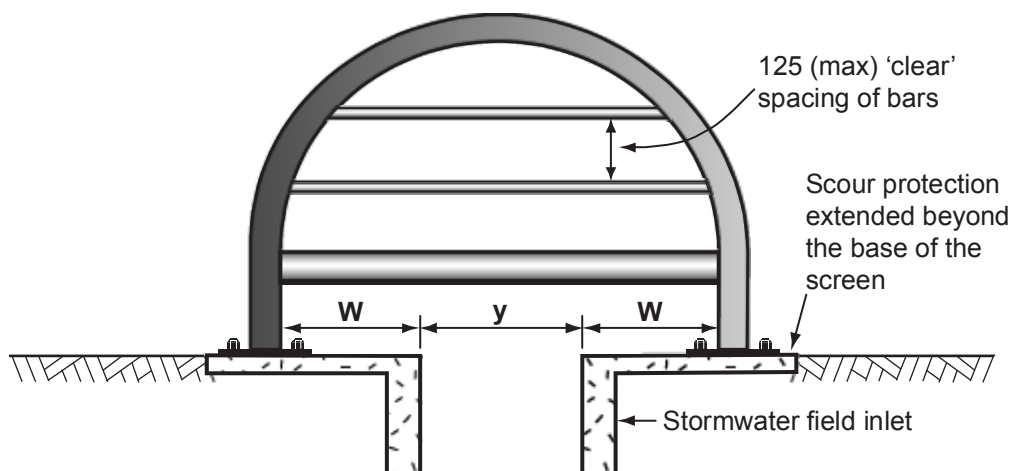
Figure 12.13 – Minimum internal lip width requirements of dome safety inlet screen

The minimum internal lip width required to achieve a through-velocity of 1 m/s (as defined in Figure 12.13) may be determined from Table 12.4.4. Table 12.4.4 and equation 12.11 have been determined through an analysis of 3-dimensional flow conditions where the critical flow velocity is assumed to occur a distance of $2.5y_c$ upstream of the edge of the lip during non-orifice flow conditions.

Table 12.5.4 – Standard dimensions of dome inlet safety screen

Total angle of approaching flow (Figure 12.14)	Minimum dimension of (W)	
	All inlets	**Square inlets operating under orifice flow**
90°	2.5 H^*	0.98 y
180° to 360°	2 H^*	0.78 y

where:

    W = width of screen extending beyond the edge of the field inlet (m)
    y = inside dimension of inlet (only relevant for square inlets) (m)
    H^* = the minimum of the following:

- maximum expected upstream water depth relative to the inlet crest
- maximum upstream head (H^*) prior to orifice flow conditions as presented in equation 12.11.

$$H^* = 1.56 (A_e/L) \tag{12.11}$$

where:

    A_e = effective clear area of the field inlet opening ($A_e = y^2$ for square inlets) (m²)
    L = total weir length of field inlet opening (L = 4y for square inlets)

| (a) Field inlet with 90° angle of approach flow | (b) Field inlet with 180° angle of approach flow | (c) Field inlet with 360° angle of approach flow |
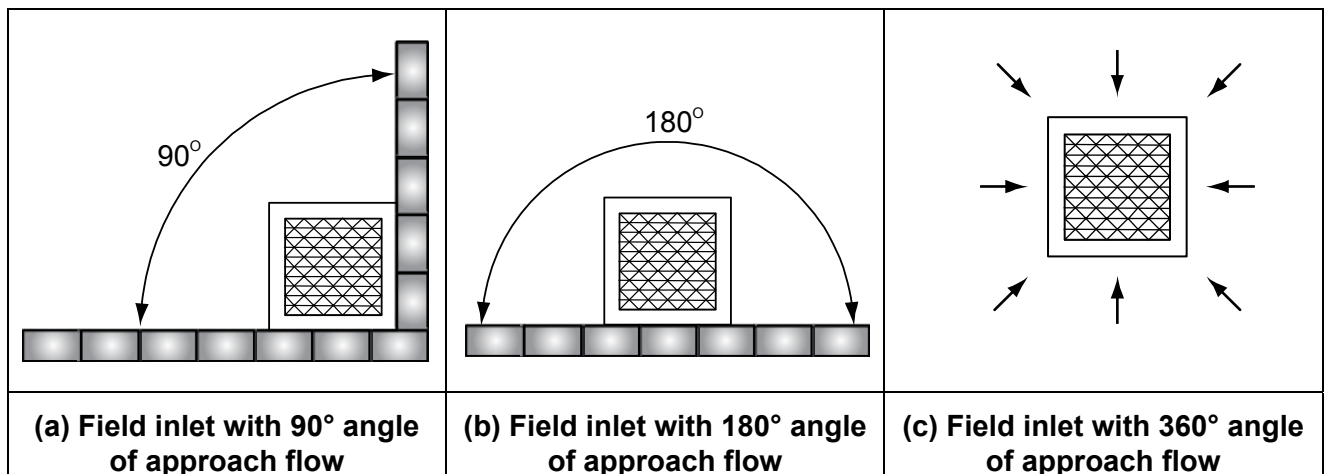
Figure 12.14 – Diagrammatic representation of approach flow angle (plan view)

12.5.9   Example culvert inlet screen

Figure 12.15 shows an example culvert inlet screen with dimensions (X and Y) provided in Table 12.4.5.

The dimensions presented in Table 12.4.5 are based on the following requirements and assumptions:

- The net open surface area of the inlet screen is at least three times the cross-sectional area of the pipe/culvert inlet.

- Flow velocity through an unblocked screen will be one-third the flow velocity through the culvert when the culvert is flowing full. Thus if the culvert velocity is less than 3 m/s then the flow velocity through the screen will be less than 1 m/s when the culvert is flowing full.

- The total width of the screen bars is assumed to cause a 14% reduction in the effective flow area at the screen.

- Wing walls are straight and parallel with the flow. If angled wing walls are used, then the design is conservative because the effective flow area at the screen is increased.

- No allowance has been made for debris blockage.

- Flow velocity approaching the inlet screen is less than 1 m/s.

Where appropriate, a more efficient and thus cost effective design may be achieved through the development of a site-specific design based on a detailed hydraulic analysis. If the maximum average flow velocity within the pipe or culvert is significantly less than 3 m/s, then a site-specific design should significantly reduce the size of the screen.
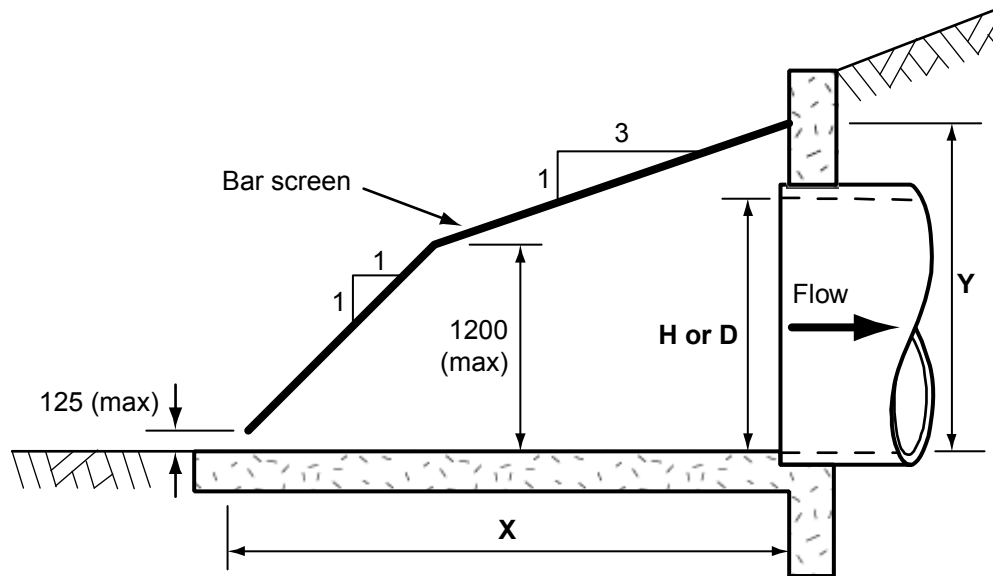
Figure 12.15 – Standard culvert inlet safety screen

Table 12.5.5 – Dimensions of example (Figure 12.15) culvert inlet screen

Box culvert			Pipe culvert		
Height (mm)	X (m)	Y (m)	Diameter (mm)	X (m)	Y (m)
600	1.77	1.39	375	1.01	0.89
750	2.26	1.55	450	1.21	1.03
900	2.75	1.72	525	1.41	1.17
1200	3.72	2.04	600	1.61	1.25
1500	4.69	2.36	750	2.01	1.38
1800	5.67	2.69	900	2.42	1.51
2100	6.64	3.01	1050	2.82	1.63
2400	7.61	3.34	1200	3.22	1.76
2700	8.59	3.66	1350	3.63	1.89
3000	9.56	3.99	1500	4.03	2.02
3300	10.53	4.31	1800	4.83	2.27
3600	11.51	4.64	2100	5.64	2.53

13. Miscellaneous matters

13.1 Relief drainage or upgrading works

13.1.1 General

Relief drainage or upgrading works are undertaken under the following circumstances:

- To augment an existing drainage system that may have been designed to a lower standard at some time in the past.

- To overcome specific drainage problems, which may include localised areas of property flooding or unacceptable road flows.

- To improve the performance of the drainage system in relation to safety, water quality, convenience, flow spread criteria or freeboard.

These works are often undertaken following receipt of complaints from the public, especially in relation to property flooding. Complaints may occur because in some cases the original scheme may have been designed to lower design criteria than those now acceptable. In some cases, changes in the intensity or type of land use may have exacerbated the runoff behaviour, whilst changes to the pattern of runoff by diversion or construction works may have directed additional flow to certain areas.

It should also be acknowledged that both hydrology and hydraulics are not exact sciences. It is possible for a drainage system to be designed in accordance with accepted design procedures, and for the final system to fail to deliver one or more of the desired outcomes. In such cases relief drainage may be required. In these circumstances, designers are encouraged to investigate the reasons for the unsatisfactory design outcomes and present their findings to the department to assist in future QUDM enhancement, and/or to a stormwater conference.

Designers should also be aware that undertaking relief drainage within a catchment may increase flows and flood levels downstream of the remediation works. Whether this relief drainage is done as a part of a new urban development, or as a local government drainage upgrade, it is important to investigate potential downstream impacts and to ensure, where practical, that the downstream drainage system does not fall below the accepted drainage standard.

13.1.2 Assessment procedures and remedial measures

A number of measures can often be taken to improve the capacity of an existing system without the need for major augmentation works. These should be examined first and include:

- Detailed field inspection to ensure that 'as constructed' records are correct, blockages or other operational faults are detected, and restrictions such as limitations in overland flow paths are identified.

- Assessment of the capacity of the existing drainage system for the purpose of identifying those areas which are deficient. This would involve a detailed hydrologic and hydraulic analysis of all components. Deficiencies frequently identified include inadequate gully inlet capacity, deficiencies in certain pipe sections, insufficiently high footpath profile or restrictions in overland flow paths. Many deficiencies can be easily remedied by relatively inexpensive augmentations or improvements.

- Partial augmentation of the underground piped drainage system. This might include upgrading of critical pipe reaches within the catchment combined with the acceptance of reduced standard of service for both major and minor storms. Examples of reduced standards of service

include reduced freeboard, a higher AEP (lower ARI) for the major storm, less stringent flow width/depth criteria, or higher AEP (lower ARI) for minor storm and non-compliance with the depth-velocity criterion. Thus, it may be possible to reduce the occurrence of property flooding through private property while allowing excessive overland flows to persist within road reserves. The local government may choose to accept on, economic grounds, a scheme to overcome property flooding problems whilst allowing a lesser standard of safety and convenience to persist in road reserves other than those outlined in Chapter 7. On this basis, a local government could target funding towards the mitigation of property flooding as its highest priority.

13.1.3   Design alternatives

In the investigation of an individual scheme, the full range of design options should be considered to determine the 'most economic' alternative to be chosen. Benefits and costs in both the short and long-term should be considered although least capital cost is commonly the method used to select the most economic alternative.

Design alternatives may include:

- doing nothing
- reduced standard of service and design criteria (section 13.1.5)
- above and below ground detention storage in parks, road reserves and private properties
- partial augmentation
- major augmentation
- purchase and removal of houses.

13.1.4   Priority ranking

It will usually be necessary not only to justify expenditure on relief drainage or upgrading works but also to have a system of ranking schemes in some form of priority order. In addition, where lesser standards of service are considered acceptable, the choice of the acceptable standard should be determined on a rational basis.

Ranking of schemes has generally been achieved using risk/consequence ranking and benefit/cost analyses. This may be as simple as considering the benefit to be equivalent to the number of properties which will no longer be flooded after implementation of the scheme and the capital cost of the scheme. Recurrent costs such as maintenance and repairs and economic loss to householders are often ignored. The difference between allotment flooding generally and property flooding including inundation of buildings is also often ignored.

Department of Natural Resources and Mines (2002) provides guidance on allotment flooding costs (i.e. backyard flooding with no flooding of habitable rooms) and property flooding costs which includes the flooding of habitable rooms.

Schemes are also commonly ranked on the basis of comparing the costs of the most obvious alternatives for each scheme rather than the most economic alternatives. This is because of the difficulties involved in comparing schemes with different standards of service between the existing and proposed drainage systems.

One such method which allows comparisons between projects with different design standards is average annual damage (AAD) analysis. AAD analysis determines the total flood damages cost (over the full range of flood probabilities from the initiating event where flood damage commences to the PMF) for a particular catchment. The difficulty in applying this method is the estimation of the flood damage resulting from the PMF flood event.

A simplified estimation of the PMF flood damage cost could be based on a pro-rata basis comparing the number of properties affected by a higher flood probability (e.g. 1 in 100 year ARI) to the potential number of properties affected by the PMF flood event and applying this ratio to the known flood damage cost for the higher flood probability. The Department of Natural Resources and Mines (2002) provides guidance in calculating flood damages and determining the AAD for a particular catchment.

A method allowing comparison between projects with different design standards of service is provided in section 13.4. The method is suitable for comparing alternatives within a given scheme and for comparing schemes on separate catchments.

It is suggested that these methods are inadequate to enable the absolute ranking of schemes but are probably sufficient to allow the determination of priority categories where all schemes within a particular category (say 5 to 10) are considered to have equal ranking. The use of Net Present Value or similar techniques is not considered warranted in the determination of priority categories. Such techniques however, may be justifiable in the case of schemes of significant cost in order to determine absolute ranking within priority categories.

13.1.5   Design criteria

Whilst the criteria set down in this Manual should be adhered to if possible for relief drainage and upgrading works, economic and physical limitations may require the adoption of less stringent criteria. These may include:

* limitation on pipe size because of easement restrictions
* reduced cover over pipes
* increased flow velocities
* increased pipe grades
* non-standard gully inlets, structures or pipe geometry
* reduced clearance to other pipes, services etc.
* departures from the flow width, d*V, freeboard and other criteria detailed in Chapter 7.

The adoption of these reduced criteria should take place only after consultation with the appropriate officer of the relevant local government. In some cases public consultation may be required.

13.2   Plan presentation

13.2.1   Design drawings

The main components that should be included in design drawing are the following:

* plan and general layout of the scheme
* structure data table including type, surface level and location
* pipe data table showing diameter, length and level information
* longitudinal sections of pipes including level and grade information, length of section, size and class of pipe, hydraulic grade line, services crossings etc.
* structure detail plan for special structures
* catchment plan and catchment table
* calculation sheets.

Note: It may be possible to omit the structure and pipe data tables if appropriate information is shown elsewhere on the drawings.

13.2.2   Standard plans

Most local governments have standard plans showing details of those drainage components that are repeated regularly within a project. Alternatively the plans may show standard requirements in respect of boundary clearances, allocations, etc. Where applicable, Department of Transport and Main Roads standard plans may also be suitable. In addition the Institute of Public Works Engineering Australia (Qld) have a number of standard drawings for road and drainage works.

Designers need to determine from the local government which standard plans are to be adopted for drainage works in its area.

13.2.3   As Constructed plans

Accurate 'As Constructed' plans shall be prepared to record any changes or departures from design that may have occurred during the construction phase if such plans are required by a local government.

These plans are usually required by local governments in order that a correct data base is available for record purposes, for asset management and maintenance. They are important as a reference source for other services authorities, future designers as well as police and emergency services.

As Constructed plans should record the following information as well as other details particular to the project, including items such as:

- pipe sizes and location
- invert levels and grades
- surface levels for structures
- the location and dimensions of structures
- structure types
- the location of subsoil drainage and clean-out points
- details of services that have been relocated.

As Constructed plans should be certified by a Registered Surveyor who was involved in the collection of the as-constructed information.

13.3   Subsoil drainage

It is desirable and, in some cases, essential practice to install subsoil or subsurface drains beneath road pavements and in conjunction with drainage pipes to drain the pavement or subgrade, or to collect seepage.

The construction of an underground stormwater drainage system with associated granular pipe bedding can result in the interception of seepage and the concentration of this intercepted water at drainage structures. The installation of subsoil drains in conjunction with the drainage pipes allows seepage water to be collected and conveyed into the drainage system.

Detailed recommendations in respect of design and installation of subsoil drains have been prepared by the Australian Road Research Board, (Gerke, 1987). In addition most Local Authorities have standard drawings and specifications detailing construction requirements. Standard Drawing MR-1116 shows typical details. Brisbane City Council (2003) provides details for subsoil drainage under grass swales.

Notwithstanding the above, it is recommended that a minimum of 3 metres of subsoil drain be installed on the upstream side of and discharging to every manhole, gully inlet or structure. The subsoil drains should be installed adjacent to every pipe leading to the manhole, gully inlet or structure.

Clean-out points should be provided in the subsoil drainage system to permit regular maintenance and the removal of accumulated silt etc.

13.4   Scheme ranking methods

13.4.1   Triple bottom line method

The Triple Bottom Line ranking method allows the incorporation of Financial, Ecological and Social issues within the assessment process. Guidelines on the application of the Triple Bottom Line analysis may be found in Taylor (2005) and Engineers Australia (2005).

13.4.2   Pseudo benefit cost analysis

The following method is a pseudo-benefits/costs method which may be used to rank schemes relative to each other in order to assist decision-makers to determine the optimum allocation of drainage funds. The method also provides a means of comparing schemes that are not of a like nature. For example, existing drainage works within competing relief drainage schemes may have been designed to different standards and the proposed augmentations may also be to different standards.

Using the following method, the cost of each scheme is apportioned as a cost per flooded allotment with the benefit simply being measured as the removal of flooding from the allotment. In order to account for flooding of a house (as distinct from flooding of an allotment only) a weighting is applied to those allotments where the house is also flooded. The unit cost is then determined by apportioning the total cost over the weighted number of allotments from which flooding can be removed by the scheme.

The unit cost is then adjusted by multiplying by a standard normalisation factor so that the unit cost comparison is made on the basis of a common standard. Schemes are then ranked in order of increasing unit cost with the lowest unit cost scheme being the preferred one, all other aspects being equal.

For scheme i:

Cost for Scheme	$= C_s^i$
Number of flooded allotments on which the house is flooded and which will be no longer flooded upon completion of the scheme	$= N_h^i$
Number of flooded allotments where house is not flooded and which will be no longer flooded upon completion of the scheme	$= N_a^i$
Weighted number of flooded allotments	$= N_w^i = W_i . N_h^i + N_a^i$
where: W_i	$=$ 5 to 100 (typically 20)

Note: The value of W_i is based upon the relative damage costs between the situation where a house is flooded and the situation where an allotment is flooded but the house on it is not. On an

allotment where the house on it is flooded, flooding of the allotment is likely to occur more frequently than on an allotment where the house is not flooded. This combination of house flooding and more frequent allotment flooding leads to a higher relative damage cost than for the less frequently flooded allotment with no house flooding.

Unit cost of Scheme $\quad = \quad C_u^i \quad = \quad (C_s^i)/(N_w^i)$

Normalised Unit Cost of Scheme $\quad = \quad C_n^i \quad = \quad S_i \cdot C_u^i$

where:

$$S_i = \frac{\left(\dfrac{1}{y_b} - \dfrac{1}{y_d}\right) \cdot \left(1 + x - \dfrac{x}{2}\left(\dfrac{1}{y_b} + \dfrac{1}{y_d}\right)\right)}{\left(\dfrac{1}{y_e} - \dfrac{1}{y_p}\right) \cdot \left(1 + x - \dfrac{x}{2}\left(\dfrac{1}{y_e} + \dfrac{1}{y_p}\right)\right)}$$

and:

y_b = ARI (years) for the base system (usually the lowest ARI of all the schemes being compared or a default value of 1).

y_d = ARI (years) for the design standard required.

y_e = ARI (years) for the existing drainage system ($y_e \geq y_b$).

y_p = ARI (years) for the proposed drainage system (see section 13.1)

x = Skewness factor (usually 0.5)

Rank schemes in order of increasing values of C_n^i with the scheme with the lowest value being the preferred.

***Note:** This procedure is indicative of priority only since it ignores the many recurrent costs to property owners as a result of flooding. In addition, with y_p equal to, say 20 years, some of the properties would be flooded more than once in those 20 years while some would only be flooded once.*

13.4.3 Hurrell and Lees procedure

An alternative procedure for scheme ranking is given in Hurrell and Lees (1992). In assessing the cost of flooding the procedure takes into account the regularity of flooding and ascribes a monetary value to the severity of flooding. It also includes a social impact factor for residential properties which are regularly flooded and a Frequent Flooding Factor for industrial and commercial properties. The benefits of mitigation proposals are determined by subtracting the post-mitigation costs of flooding in each case from the pre-mitigation costs. The certainty of construction cost estimates are factored depending upon the extent to which investigation, design and detailed estimates have been completed. A variety of ranking procedures are suggested.

13.5   Symbols and abbreviations

A Catchment area

A_e Equivalent area of pipe

A_g Clear opening area of gully inlet

A_i Impervious catchment area

A_L Area of lateral pipe at a junction

A_o Area of outflow pipe at a junction

A_p Pervious catchment area

AEP   Annual exceedence probability (per cent)

ACHA   *Aboriginal Cultural Heritage Act, 2003* (Qld)

ARI   Average recurrence interval (years)

ARR   Australian Rainfall & Runoff

b Channel base width

B Channel width, routing parameter RAFTS Model or junction pit width

C Coefficient of discharge

C_g Interpolation coefficient for intermediate Qg/Qo ratios in the Hare pressure change coefficient charts

C_i Coefficient of discharge (impervious area)

C_p Coefficient of discharge (pervious area)

C_u Total energy loss coefficient

C_v Volumetric runoff coefficient (either for a single storm or as an annual average)

C_w Weighted coefficient of runoff

C_y Coefficient of discharge for ARI of y years

C_{weir} Weir coefficient

$(C.A)$ Equivalent impervious area

CPM Act   *Coastal Protection and Management Act 1995* (Qld)

CHMP   Cultural Heritage Management Plan under the ACHA

d Channel flow depth or diameter of obstructing pipe

d_{av} Average flow distance in channel network

d_c Depth of flow at crown of road, or critical depth in closed conduit flow

d_g Depth of flow in gutter, or channel adjacent to a kerb

d_p Depth of flow at pavement edge (lip)

D Pipe diameter, or duration of rainfall excess

D_e Equivalent pipe diameter

D_f Pipe diameter – far lateral

D_L Pipe diameter – lateral

D_{LL} Pipe diameter – lateral left looking downstream

D_{LR} Pipe diameter – lateral right looking downstream

D_n Pipe diameter – near lateral

D_o Pipe diameter for outlet pipe

DOGIT   Deeds of Grant in Trust

DOT	Department of Transport, Queensland	h_s	Head loss or pressure change at a structure
D_u	Pipe diameter for upstream pipe	h_{sup}	Superelevation (difference in level) of the water surface across an open channel at a bend
d/s	Downstream		
EIS	Environmental Impact Statement	h_t	Head loss at a channel transition
EP Act	Environmental Protection Act 1994(Qld)	h_u	Head loss (pressure change) for the main pipe at a structure
f	Darcy-Weisbach friction factor	h_w	Change in water surface elevation
f_i	Fraction impervious	HGL	Hydraulic grade line
F	Kerb and channel flow correction factor in the Izzard equation, or factor of proportionality in the Bransby-Williams equation	HAT	Highest astronomical tide
		I	Average rainfall intensity (mm/hr)
F_y	Frequency factor	I_y	Average rainfall intensity for ARI of y years
FRC	Fibre-reinforced cement (pipes)	${}^t I_y$	Average rainfall intensity for duration of t hours and ARI of y years
g	Gravitational acceleration, (9.80m/s^2 in Queensland)		
		IDAS	Integrated Development Approval System under SPA
h	Depth of water		
ha	Hectares (area)	ILUA	Indigenous Land Use Agreement
h_a	Head loss (pressure change) at a surcharge manhole	k	Pipe boundary roughness (Colebrook-White)
h_b	Head loss (pressure change) at a channel bend	k_c	Empirical coefficient – RORB Model parameter
h_c	Height (distance) to the centreline of an obstructing pipe from the most distant pipe wall	k_r	Dimensionless ratio called the relative delay time – RORB Model
h_f	Pipe friction head loss (pressure change) or pressure change at far lateral	K	Conveyance = $(1/n)AR^{2/3}$, or head loss or pressure change coefficient
h_g	Gully head loss – grate inflow	K_a	Pressure change coefficient at a surcharge manhole
h_n	Head loss (pressure change) at near lateral	K_b	Bend loss coefficient
h_p	Head loss at penetration	K_e	entry loss coefficient

K_g	End gully pressure change coefficient or pressure change coefficient through a grate	L_{eff}	Effective length of drainage path
K_{HV}	Junction pit pressure change coefficient – higher velocity lateral, applied to downstream velocity head	Land Act	*Land Act 1994* (Qld)
		LAT	Lowest astronomical tide
		LG Act	*Local Government Act 2009* (Qld)
$\overline{K}_L$	Intermediate pressure change coefficient – lateral pipe	m	Exponent – RORB Model parameter
K_L	Junction pit pressure change coefficient – lateral pipe, applied to downstream velocity head	MHWN	Mean high water neap
		MHWS	Mean high water spring
K_{LL}	Junction pit pressure change coefficient – left lateral pipe (looking d/s), applied to downstream velocity head	MLWN	Mean low water neap
		MLWS	Mean low water spring
		MSL	Mean sea level
K_{LR}	Junction pit pressure change coefficient – right lateral pipe (looking d/s), applied to downstream velocity head	MWL	Mean water level
		M	Design ARI for gap flow (years)
		M_L	Pressure change coefficient multiplier – lateral pipe
K_{LV}	Junction pit pressure change coefficient – lower velocity lateral, applied to downstream velocity head	M_u	Pressure change coefficient multiplier – main pipe
K_p	Penetration loss coefficient	n	Manning's roughness coefficient, or Horton's roughness value
Ku	Junction pit pressure change coefficient – upstream pipe, applied to downstream velocity head	n_g	Manning's n – gutter or channel
		n_p	Manning's n – pavement
$\overline{K}_u$	Intermediate pressure change coefficient – main pipe	$n*$	Surface roughness/ retardance coefficient
K_w'	Water surface elevation increment coefficient applied to upstream velocity head	N	Design ARI for minor system (years)
K_w	Water surface elevation change coefficient applied to downstream velocity head	N_R	Reynold's Number = $V_o.D_o/\nu$
		NCA	*Nature Conservation Act 1992* (Qld)
L	Stream flow length, or overland flow path length, or pipe length, or gutter flow length, or weir length	NSB	Notes on the Science of Building (CSIRO)

P	Wetted perimeter, or depth of rainfall excess	$Q_{SURF()}$	Net surface flow at the given location
P^*	Effective wetted perimeter	$Q_{T()}$	Peak discharge from the total catchment
P&D Act	*Plumbing and Drainage Act 2002* (Qld)	Q_u	Upstream pipe flow
Q	Flow rate (m3/s or L/s)	Q_y	Peak discharge rate for ARI of y years
Q_f	Flow rate – full area, or inflow rate – far lateral	QDC	Queensland Development Code
Q_g	Surface inflow to gully inlet	RCBC	Reinforced concrete box culvert
Q_{gap}	Gap flow	RCP	Reinforced concrete pipe
$Q_{gs()}$	Required kerb inlet capacity of the inlets located in the section upstream of the critical location	r	Centreline radius of a pipe bend or reduction ratio
Q_i	Peak or design inflow rate	R	Hydraulic radius = A/P
Q_L	Lateral pipe flow to junction pit	R_c	Centreline radius of open channel bend
$Q_{LIM()}$	Permissible major storm street or overland flow at given location	R_i	Inner radius of open channel bend
Q_{LL}	Lateral pipe flow left looking downstream	R_o	Outer radius of open channel bend
Q_{LR}	Lateral pipe flow right looking downstream	RRJ	Rubber ring jointed
Q_m	Outflow rate from a surcharge manhole	S	Channel slope, storage or submergence depth at a structure
Q_n	Inflow rate – near lateral	S_e	Modified equal area slope (%)
Q_o	Peak or design outflow rate, or outlet pipe flow	S_f	Friction slope
Q_p	Discharge rate – part area	S & S	Spigot and socket
$Q_{P()}$	Required pipe discharge capacity at given location	SBR	Standard Building Regulation
Q_{peak}	Peak flow rate	T	Routing time step
$Q_{PU()}$	Sum of the pipe discharges at given location	t	Time
		t_c	Time of concentration or travel time from extremity of pervious area
		t_i	Travel time from extremity of impervious area or drop in pipe inverts at a drop manhole

U	Fraction of catchment urbanised
u/s	Upstream
UPVC	Unplasticised polyvinyl chloride
$V^2/2g$	Velocity head
V	Velocity (m/s)
V_{ave}	Average velocity of channel flow
V_e	Equivalent velocity of flow
V_g	Velocity through a grate
V_u	Velocity in junction pit inflow (upstream) pipe
$\boldsymbol{V}$	Runoff volume (m^3)
$\boldsymbol{V_i}$	Volume of inflow
V_o	Volume of outflow, or velocity in junction pit outflow pipe = Qo/Ao
V_s	Storage volume
w	Width of flow spread from kerb (m)

Water Act	*Water Act 2000* (Qld)
WSE	Water surface elevation
y	General ARI expression (years)
Z_g	Reciprocal of gutter or channel cross-slope
Z_p	Reciprocal of pavement cross-slope
α	Deflection angle, or velocity head coefficient
β	Triangular flow correction factor
Δ	Channel flow multiplier
θ	Upstream-downstream pipe deviation angle at junction pit
ν	Kinematic viscosity (water) = 1.14 x 10^{-6} m^2/s at 15°C
Δh	Water surface superelevation at a bend in an open channel

13.6   Glossary of terms

Allotment drainage	A system of field gullies, manhole chambers and underground pipes constructed within private property to convey flows through and from allotments.
Annual exceedence probability (AEP)	The probability of exceedence of a given discharge within a period of one year. AEP is generally expressed as a percentage.
Average recurrence interval (ARI)	The average or expected value of the period between exceedences of a given discharge. ARI is generally expressed as Y years or 1 in Y years. The terminology of ARI is generally used where the data and procedures are based on partial series analysis.
Bankful discharge	The channel flow rate that exists when the water is at the elevation of the channel bank above which water begins to spill out onto the floodplain. The identification of bankful elevation is described in ARR (1998) Book 4, Section 2.11.3.
Backwater curve analysis	A procedure for determining water surface levels in open channels under gradually varied flow conditions.
Bio-retention system	A well-vegetated, retention cell or pond designed to enhance water filtration through a specially prepared sub-surface sand filter. Bio-retention cells may be incorporated into grass or vegetated swales or may be a stand-alone treatment system. The system incorporates vegetation with medium-term stormwater retention and sub-surface filtration/infiltration. Also known as bio-filtration systems or biofilters.
Building	A habitable room; retail or commercial space; factory or warehouse; basement providing car parking space, building services or equipment; or enclosed car park or enclosed garage.
Bypass flow	That portion of the flow on a road or in a channel which is not collected by a gully inlet or field inlet, and which is redirected out of the system or to another inlet in the system.
Channel freeboard	Vertical distance between the design water surface elevation in an open channel and the level of the top of the channel bank.
Climate change	Changes in the earth's climatic conditions as a result of natural and human activities.
Coastal Management Area	The area of land covering:
	40 metres landward from MHWS where there is no approved revetment wall
	10 metres landward from the seaward edge of an approved revetment wall.
	It is noted that State Marine Parks generally extend to HAT.
Coefficient of runoff	A dimensionless coefficient, used in the Rational Method for the calculation of the peak rate of storm runoff.
Consequence	Outcome or impact of an event.

Constructed wetlands	A shallow pool of water, characterised by extensive areas of emergent aquatic plants/macrophytes, designed to support a diverse range of micro-organisms and plants associated with the breakdown of organic material and trapping of nutrients. Wetlands may be designed as permanent wet basins (perennial), or ephemeral systems.
Critical depth	The depth occurring in a channel or part full conduit at a condition of flow between subcritical and supercritical flow, such that the specific energy is a minimum for the particular flow per unit width.
Critical flow	The condition of flow in a section of a channel or part full conduit when the flow is at critical depth.
Critical velocity	The average velocity of flow in a section of a channel or part full conduit when the flow is at critical depth.
Cross drainage	A system of pipes or culverts which convey storm flows transversely across or under a roadway.
Defined flood event	The flood event adopted by a local government for the management of development in a particular locality. It defines the natural hazard management (flood) area. It does not define the extent of flood-prone land.
Detention basin	A large, open, free draining basin that temporarily detains collected stormwater runoff. These basins are normally maintained in a dry condition between storm events.
Development category	Refers to the land use within a catchment. A specific fraction impervious and drainage design standard is usually defined for a given development category.
Drainage catchment	The area of land contributing stormwater runoff to the point under consideration.
Drainage system	A system of gully inlets, pipes, overland flow paths, open channels, culverts and detention basins used to convey runoff to its receiving waters.
Enclosed GPTs	A fully enclosed trash rack and/or sediment collection sump usually located at or near the end of a stormwater pipe.
Exfiltration systems	Large underground stormwater detention tanks/pit from where stormwater is allowed to infiltrate into the surrounding soil. An infiltration trench is just one type of exfiltration system.
Extended detention	A stormwater detention basin or tank designed to drain over a period of days rather than hours to enhance its pollution retention and solar treatment while minimising the adverse effects of coincident flooding downstream of the basin.
Extreme flood	The rare flood event for which the performance of a detention basin or similar structure should be checked in order to assess the economic and social risk that could be associated with overtopping or failure of that structure.
Filter basin	Large excavated stormwater retention basin incorporating a sand filter bed. Filter systems primarily drain to surface waters or a piped drainage system, rather than rely on soil infiltration.

Filter strips	Grassed slopes with an even-gradient across the slope used to filter and infiltrate sheet flow. They must be absent of any drainage depressions that may concentrate flow. Also known as buffer zones. They differ significantly from the Grassed Filter Strips used in construction-site sediment control.
Floating boom	A floating boom with mesh skirt anchored across a permanently wet channel, creek or river. Originally designed as an oil slick retention device, the boom collects floating or partially submerged objects.
Floating GPT	A partial channel-width floating boom directing floating litter and debris into a floating pollutant retention cage.
Flood	The temporary inundation of land by expanses of water that overtop the natural or artificial banks of a watercourse, including a drainage channel, stream, creek, river, estuary, lake or dam, and any associated water holding structures.
Flood water	Those waters causing land to flood.
Floodplain	A floodplain is defined as the extent of land inundated by the Probable Maximum Flood.
Floodway	That part of the floodplain specifically designed to carry flood flows and ideally capable of containing the Defined Flood Event.
Fraction impervious	That part of a catchment which is impervious, expressed as a decimal or percentage.
Freeboard	The difference in height between the calculated water surface elevation and the top, obvert, crest of a structure or the floor level of a building, provided for the purpose of ensuring a safety margin above the calculated design water elevation. (See also Channel Freeboard).
Frequency factor	A factor which is multiplied by the coefficient of runoff for the 10% AEP (10 year ARI) to determine the coefficient of runoff for the design AEP, for the location being considered.
Friction slope	Sometimes referred to as the hydraulic gradient or pressure gradient and is the slope of the line representing the pressure head, or piezometric head in a pipeline.
Grass swale	Shallow, low-gradient, grass-lined overland flow path used primarily for stormwater treatment.
Grate inlet screen	Typically a coarse screen placed across the face of a roadside kerb inlet to filter gross pollutants from stormwater. Pollutants are retained on the screen for later collection usually by a street sweeper.
Half-bankful Discharge	The channel flow rate that exists when the water level is midway between the channel invert and the elevation of the channel bank above which water begins to spill out onto the floodplain.
Hazard	A source of potential harm.
Head loss coefficient	A dimensionless coefficient which, when multiplied by the velocity head in the outlet pipe, gives the difference in hydraulic grade level between inlet and outlet pipe. It may be positive (indicating that the HGL rises upstream) or negative (indicating that the HGL is less upstream).
High level basin outlet	The outlet of a detention or retention storage system provided for discharges that exceed the capacity of the low level outlet.

Hydraulic design	The component of drainage design that involves the determination of velocities, heads and water levels as storm runoff passes through the drainage system.
Hydraulic grade line	A line representing the pressure head along a pipeline, corresponding to the effective (free) water surface elevation in the piped portions of the stormwater drainage system.
Hydraulic gradient	The slope of the hydraulic grade line - see also Friction Slope.
Hydraulic radius	The ratio A/P, A being the cross-sectional area and P the wetted perimeter – that is, the length of the line of contact (on the cross section) between the water and the channel boundary.
Hydrologic design	The component of drainage design that involves determination of stormwater runoff, either discharge or volume.
Impervious surface (impervious area)	A surface or area within a drainage catchment where the majority of rainfall will become runoff i.e. no infiltration e.g. roadways, car parks, roofs etc.
Infiltration basin	Large, excavated basins designed to retain storm flows, allowing infiltration and evaporation.
Infiltration trench	An excavated pit filled with uniform gravel or rock into which runoff is directed for short to medium-term detention before finally infiltrating into the surrounding soil. The surface of the trench is usually vegetated.
Intensity-frequency-duration data (IFD)	Basic rainfall data used in the calculation of rainfall runoff rates.
Integrated Catchment Management (ICM)	Managing natural resources within a whole of system approach. In a stormwater context, this requires a whole of catchment approach incorporating the total water cycle. Consideration is given to all associated land and water processes and values.
Junction structure	A manhole, pit or chamber constructed at the junction of two or more pipes, or at a change of grade.
Land use (development category)	The particular use or uses (actual or allowable) of land within a catchment.
Large detention storage	A large detention or retention storage such as a lake, pond, basin or large car park designed or able to significantly reduce and attenuate the peak discharge from a catchment.
Lawful point of discharge	A point of discharge which is either under the control of a Local Authority or Statutory Authority, or at which discharge rights have been granted by registered easement in favour of the Local Authority or Statutory Authority, and at which discharge from a development will not create a worse situation for downstream property owners than that which existed prior to the development.
Likelihood	Probability or frequency of an event.
Litter basket	An in-pipe litter and debris collection basket installed within junction pit of a piped stormwater drainage system.

Local authority	Any local or regional external authority—whether government or non-government, including local governments and the State Government—that has a legal interest in the regulation or management of a given activity, or the land on which the activity is occurring, or is proposed to occur. Reference to the local authority shall also imply the plural.
Local government	The local city or shire council with jurisdiction over the land in which the activity in question is occurring, or is proposed to occur.
Low level basin outlet	The outlet of a detention/retention storage from which discharge will first occur (usually via a pipe).
Major design storm	The rainfall event for the AEP chosen for the design of the Major Drainage System.
Major drainage system	That part of the overall drainage system which conveys flows greater than those conveyed by the Minor Drainage System and up to and including flows from the Major Design Storm.
Major overland flow path	An overland flow path that drains water from more than one property, has no suitable flow bypass, and has a water depth in excess of 75mm during the major design storms; or is an overland flow path recognised as significant by the local government.
Major road	A road whose primary function is to serve through traffic. These roads include Collector Roads, Sub-Arterial and Arterial Roads. Refer to Department of Main Roads or AustRoads for further definition.
Manning's roughness coefficient	A measure of the surface roughness of a conduit or channel to be applied in the Manning's equation.
Mini wetland	Small, usually ephemeral wetlands, usually located adjacent stormwater outlets or in association with a landscaped area. Mini wetlands differ from bio-retention cells in that they may or may not incorporate stormwater retention (though it is preferred) and they do not rely on sub-surface filtration due to the typical long-term saturation of the clayey soil bed.
Minor design storm	The rainfall event for the AEP chosen for the design of the Minor Drainage System.
Minor drainage system	That part of the overall drainage system which controls flows from the Minor Design Storm e.g. kerbs and channels, inlets, underground drainage etc. for the purpose of providing pedestrian safety and convenience, and vehicle access.
Minor road	A road whose primary function is to provide access to abutting allotments. These roads include Residential Streets. Refer to Department of Main Roads (Access or Local Roads, max. 1000 vpd) or AustRoads for further definition.
Oil and grit separator	Generally a two or three chamber underground retention tank designed to remove hydrocarbons, floating pollutants, coarse sediment and grit. The first chamber is used for sedimentation and the collection of large debris. The second chamber is used for oil separation. The third chamber (if used) collects and disperses flow into the stormwater system.
On-site detention (OSD)	A relatively small open basin or enclosed stormwater tank fully contained within a single allotment or group-title allotment.

Open GPT	Combined sediment basin and trash rack usually located at the downstream end of a stormwater pipe or constructed drainage channel.
Outlet litter cage	Solid trash and litter collection cage attached to the outlet of a stormwater pipe which screens gross pollutants from stormwater holding the pollutants within the cage usually elevated above normal water level.
Overland flow path	Where a piped drainage system exists: it is the path where storm flows in excess of the capacity of the underground drainage system would flow.
	Where no piped drainage system or other form of defined watercourse exists: it is the path taken by surface runoff from higher parts of the catchment to a watercourse, channel or gully. It does not include a watercourse, channel or gully with well defined bed and banks.
Pervious surface (pervious area)	A surface or area within a drainage catchment where some of the rainfall will infiltrate thus resulting in a reduced volume and rate of runoff e.g. grassed playing fields, lawns etc.
Pollution containment system	Typically an open, non-draining pond designed to capture pollution spills from traffic accidents. The trapped pollution is usually pumped from the system and removed from the area in tankers for off-site treatment and disposal.
	Stormwater detention/retention systems may operate as pollution containment systems if the outlet system is suitably designed to allow quick shut down (usually through the use of a gate or stop boards) by emergency services or maintenance personnel.
Pond (stormwater treatment)	Large, open water treatment ponds often incorporating a heavily vegetated macrophyte (wetland) area.
Porous pavement	Formally constructed porous, light-traffic pavements that allow runoff to infiltrate into the underlying soil or a sub-surface drainage system.
Pressure change coefficient	Refer to 'Head loss coefficient'.
Probable maximum flood	The theoretically greatest runoff event from a particular drainage basin.
Probable maximum precipitation	The theoretically greatest depth of precipitation for a given duration that is physically possible over a particular drainage basin.
Regulating authority	Local Authority involved in the regulation of an industry or land use practice.
Release nets	A litter collection net attached to a stormwater pipe outlet used to filter gross pollutants (excluding sediment) from stormwater. A release system allows the net to break free of the pipe outlet in the case of excessive hydraulic pressure caused by extreme flows or debris blockage of the net. A tether is used to secure the net to the outlet so that the released net and its captured pollutants do not wash downstream.
Retardation system	Any detention, extended detention or retention system, including on-site detention systems and rainwater tanks.

Retention basin	A large, open, partially free draining basin designed to retain a portion of the storm runoff either for water quality treatment benefits, or to assist in reducing the effective volume of runoff. The free-draining portion of the basin may be designed to operate as a traditional detention or extended detention system.
Retention system	Any stormwater collection systems that retains stormwater runoff for water supply, replenishment of lake or wetland water, or as a long-term groundwater infiltration.
Risk	The chance of something happening that will have an impact on objectives. It is measured in terms of a combination of the consequences of an event and their likelihood.
Runoff	That part of rainfall which is not lost to infiltration, evaporation, transpiration or depression storage.
Sand filter	Excavated pit or structure filled with a filter sand medium through which stormwater is allowed to pass. The filtered runoff is then collected by a drainage system and discharged. Filter systems primarily drain to surface waters or a piped drainage system, rather than rely on soil infiltration.
Sedimentation basin	A permanent sediment collection basin as opposed to a temporary construction site sediment basin. A tank or basin designed for low-velocity, low-turbulent flows suitable for settling coarse sediment particles from stormwater runoff.
Side entry pit trap	Debris baskets placed within the collection pit of roadside gully inlets. The baskets are installed below the invert of the gutter.
Small detention storage	A small detention or retention storage such as a small car park or underground storage tank designed or able to reduce and attenuate the peak discharge from a catchment.
Specific energy	The energy per unit weight of water at any section of a channel or part full conduit measured with respect to the invert or bottom of the channel or conduit.
Structural soil	A surface soil profile which combines either synthetic or natural materials with in-situ soils to improve the strength or trafficability of the soil. Ongoing soil compaction is reduced which allows grassed surfaces to withstand light traffic.
Subcritical flow	Flow in a channel or conduit which has a depth greater than the critical depth and a velocity less than the critical velocity.
Supercritical flow	Flow in a channel or conduit which has a depth less than the critical depth and a velocity greater than the critical velocity.
Surcharge outflow or overflow	That portion of the flow which is forced out of a piped system at a kerb inlet, manhole or surcharge structure when the capacity of the downstream pipe system is exceeded.
Tidal definitions:	
(a) Highest astronomical tide (HAT)	Highest tide level which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions.

(b) Lowest astronomical tide (LAT)	Lowest tide level which can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions.
(c) Mean high water springs (MHWS)	The long term average of the heights of two successive high tides when the range of tide is greatest, at full moon and new moon.
(d) Mean low water springs (MLWS)	The long term average of the heights of two successive low tides when the range of tide is greatest, at full moon and new moon.
(e) Mean high water neaps (MHWN)	The long term average of the heights of two successive high tides when the range of tide is the least, at the time of the first and last quarter of the moon.
(f) Mean low water Neaps (MLWN)	The long term average of the heights of two successive low tides when the range of tide is the least, at the time of the first and last quarter of the moon.
(g) Mean sea level (MSL)	The average level of the sea over a long period.
(h) Storm surge	The increase in sea level occurring during a cyclone or severe storm resulting from the combined effect of reduced atmospheric pressure and the build up of water against the shore caused by onshore wind (wind stress).
(i) Wave setup	The raising of sea level inside the surf zone resulting from the momentum flux of broken waves.
Transition loss coefficient	Coefficient associated with head losses at open channel transitions.
Trash rack	A series of metal bars located across a stormwater channel or pipe to trap litter and debris. The bars may be vertical or horizontal depending on hydraulic and environmental requirements (e.g. fish passage issues), and may or may not be inclined to the horizontal.
Treatment train	A series of water quality treatment systems through which contaminated water flows and is treated where the treatment systems vary in both the type of treatment (i.e. settlement, filtration, infiltration, adsorption) and the standard of treatment (i.e. the equivalent of primary, secondary and tertiary wastewater treatment standard).
Velocity head	A measure of the kinetic energy of flow in a pipe or channel and equal to $(V^2/2g)$ where V is the average velocity of flow.
Volumetric runoff coefficient	The ratio of the volume of stormwater runoff to the volume of rainfall that produced the runoff. Different coefficients will be obtained when analysing single storm events compared to the assessment of the average annual runoff.
Water Sensitive Urban Design (WSUD)	A set of design elements and on-ground solutions that aim to minimise impacts on the water cycle from the built urban environment. It offers a simplified and integrated approach to land and water planning by dealing with the urban water cycle in a decentralised manner consistent with natural hydrological and ecological processes.

Water surface elevation (WSE)	The elevation of the water surface reached in a gully inlet, manhole or junction structure.
Water surface superelevation	The phenomenon where flow around a horizontal curve in an open channel is at a higher level at the outer edge than at the inner edge of the curve.

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15.   Index

Appendix 1 Pipe flow design charts

A1.1   Introduction

The hydraulic capacity of stormwater drainage pipes flowing full, but not under pressure, may be calculated using Manning's equation (equation A1.1).

$$V = (1/n).R^{2/3}.S_f^{1/2} \qquad\qquad (A1.1)$$

where:

V = Velocity (m/s)
A = Area of flow (m^2)
P = wetted perimeter (m)
R = Hydraulic radius = A/P (m)
S_f = Friction slope (m/m)
n = Manning's roughness coefficient

When selecting an appropriate Manning's roughness for drainage pipes, some allowance must be made for the induced energy loss resulting from the normal installation and use of stormwater pipes. Factors that may influence (i.e. increase) the effective Manning's roughness include:

- the junction of pipe sections (i.e. the joining of pipe segments between junction pipes)
- the potential misalignment of these pipe segments
- the transportation sediment within the fluid
- the influence of sedimentation along the base of the pipeline.

In effect, this means that even though an individual, new, pipe segment may have a Manning's roughness close to, say 0.011; the recommended 'design' roughness value is likely to be around 0.013.

Table A1.1.1 provides recommended 'design' Manning's roughness coefficients for various pipes encountered in urban drainage systems.

Table A1.1.1 – Recommended values for surface roughness (average pipe condition)

Type of pipe	Manning's *n*
Reinforced concrete (RCP and RCBC)	0.013
Fibre reinforced cement (FRC)	0.013
UPVC	0.011
GRP	0.011

Chart A1-1 and A1-2 provide solutions of Manning's equation for a Manning's roughness of 0.013. The nomographs are based on the internal diameters of exactly 150, 300, 375, 450 mm, etc. as identified on the figures. Designers should check **actual** internal diameters for the type and class of pipe being designed and make the necessary correction where this is significant.

Chart A1-3 provides design parameters for pipes flowing partially full.

Pipe flow capacity chart (Manning's equation)

For circular pipes flowing full but not under pressure.   Manning' equation: $Q = (1/n) A R^{2/3} S^{1/2}$

Manning's n = 0.013

VELOCITY (m/s)

INTERNAL DIAMETER

Discharge in litres per second

Discharge in cubic metres per second

Pipeline grade (S)
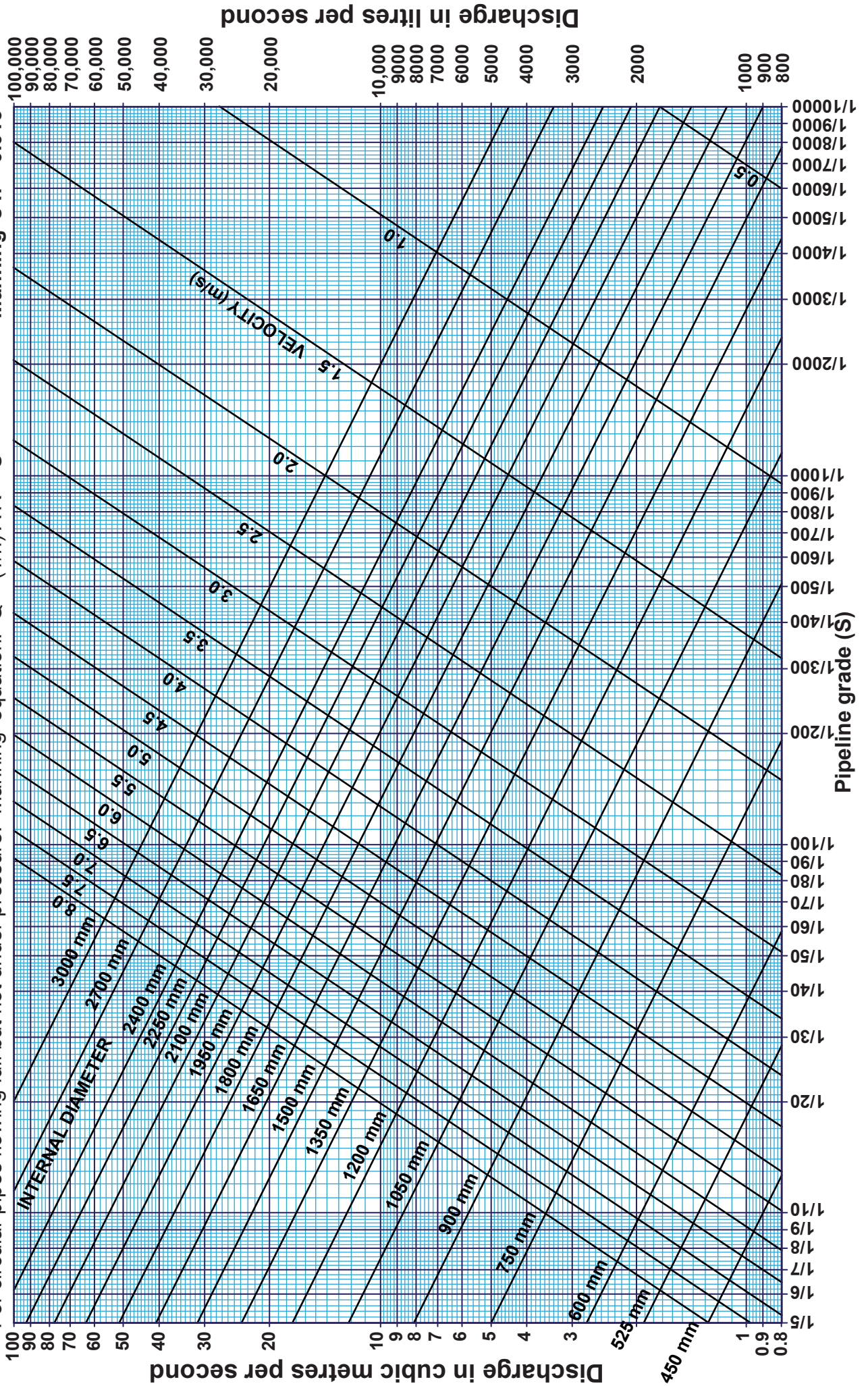
Chart No. A1-1

Chart No. A1-2
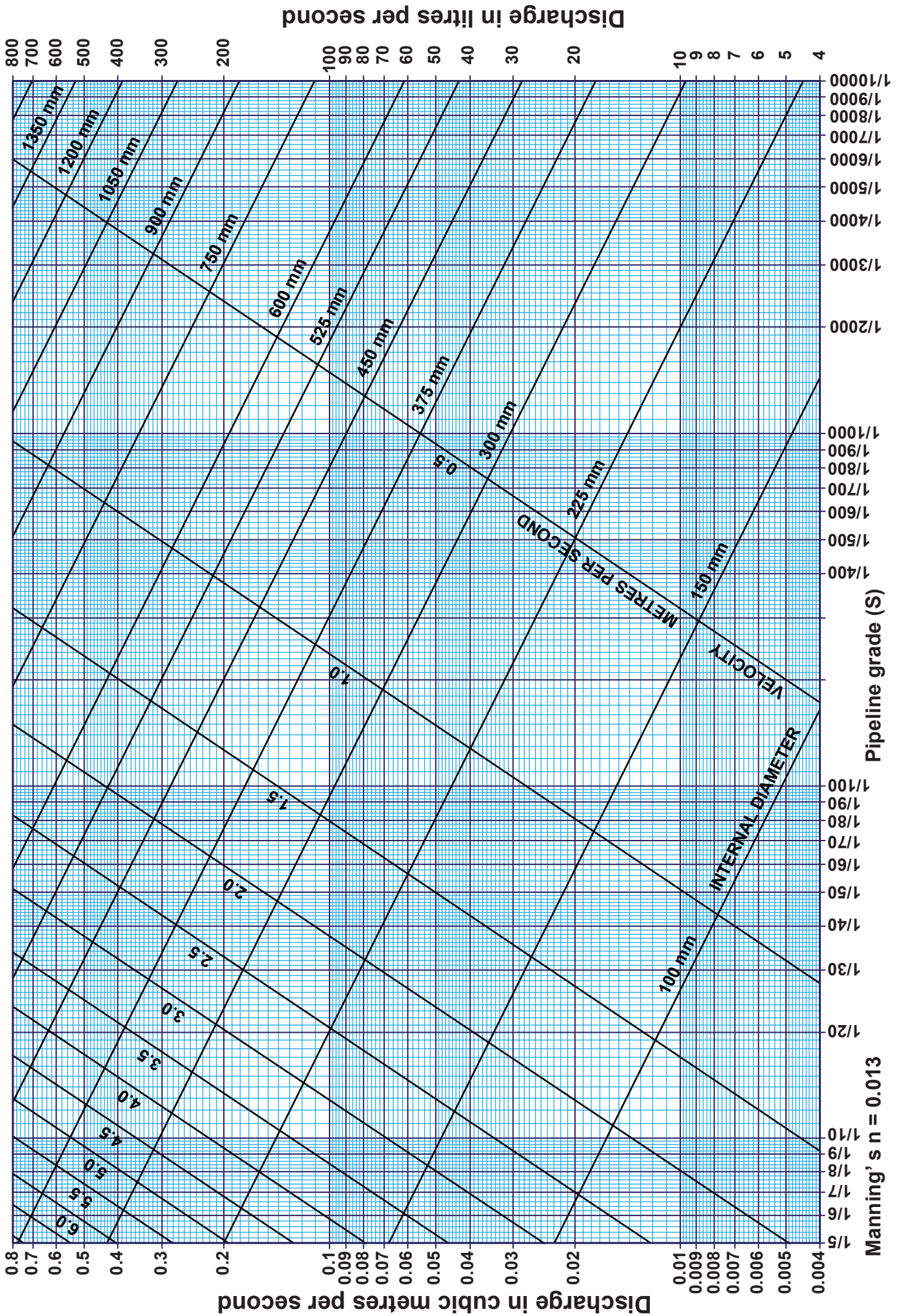
Full flow elements: Q_o, V_o, A_o, R_o Partial full elements: Q, V, A, R

% Value of full section (Q/Q_o, V/V_o, A/A_o, R/R_o)

% Relative depth of flow (y/D)

% Discharge

% Area

% Hydraulic radius

% Velocity

D

y
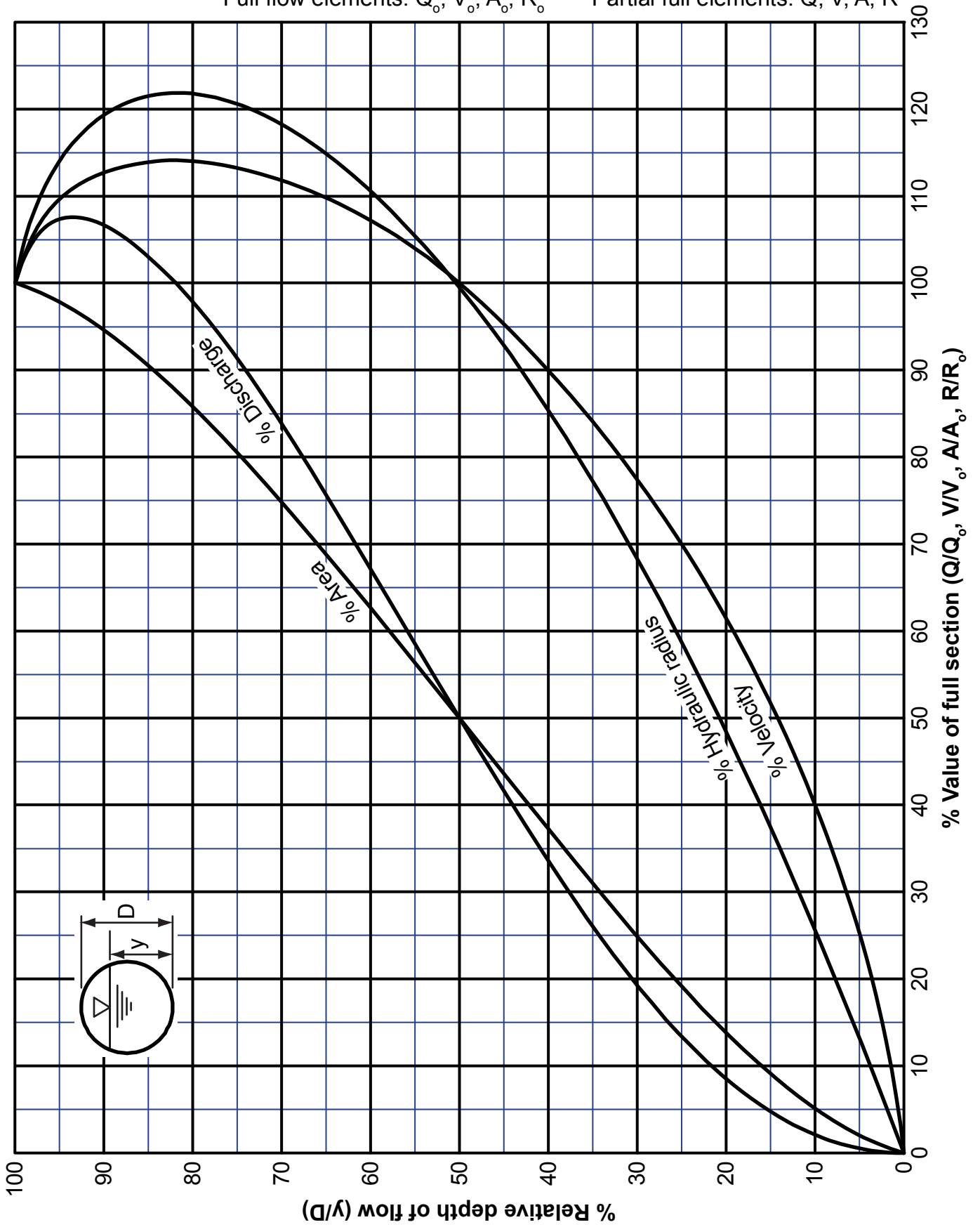
Hydraulic elements for pipes flowing partially full

Chart No. A1-3

Appendix 2 Structure pressure change coefficient charts

A2.1   Introduction

Guidance is given in general to the use of the design charts presented in the appendix. In order that designers may gain the skills necessary to apply the charts to 'non-standard' situations, it is expected that they will become familiar with the charts, how they respond to such things as changes to D_u, D_o, D_L and S/D_o and how they might also respond to changes to shape.

A2.2   General guidance

A2.2.1   Effect of structure shape

It is to be expected that the greater the separation of the outlet to the upstream pipe from the inlet to the downstream pipe, the greater the expansion of the jet from the upstream pipe and therefore the greater the pressure loss. However, the Hare Charts (Hare, 1980) already take this factor into consideration and have taken the place of the Missouri Charts (Sangster et al. 1958) for which reduction in this separation reduced the pressure change coefficient.

The Hare Charts, Missouri Charts and the Cade and Thompson Charts (Cade and Thompson, 1982) have been prepared predominantly for values of B/D_o approximately equal to 2. Based upon their testing and observations, the designer should ensure that $B/D_o \leq 2$, where practical.

Where $B/D_o > 2$, it can be expected that values of K_u and K_w will be greater than those given by the charts. Where $B/D_o < 2$, values of K_u and K_w can be expected to decrease. This is not usually of significant benefit (especially with circular chambers) since other factors will usually dominate.

In the usual range ($0.8 < D_u/D_o < 1.0$) K_u and K_w will decrease by up to 0.4 as B/D_o is reduced from 2 to 1 (except as noted in A4.2.2 below for intersection on the downstream face).

Warning: This general rule does not apply to access chamber with lateral pipes.

A2.2.2   Coefficients K_u and K_w (Hare charts)

The following are points worth noting in relation to the performance of the Hare charts:

- Where the intersection point is on the downstream face K_u may be taken as the same as K_w. Otherwise $K_w > K_u$.

- Where $D_u/D_o < 1$, K_u and K_w may be negative, i.e. there is a pressure gain at the structure not a loss.

- Where the intersection point is on the downstream face, structure size and shape has no influence on K_u or K_w.

- Where the intersection point is not on the downstream face, K_u may be higher than values derived from past practice using the Missouri charts. In addition, the actual WSE may be up to $0.3(V_u^2/2g)$ higher than the HGL in this situation.

Note: for very low submergence ratios ($S/D_o \leq 1.5$) the WSE may be up to $0.8(V_u^2/2g)$ above the HGL so that this situation must be avoided wherever possible, particularly at critical structures.

A2.2.3   Hare charts v. Cade and Thompson

The Hare charts have been based on tests of square structures with B/D_o approximately equal to 2. Cade and Thompson used circular structures with B/D_o approximately equal to 2 but did not

provide for gully inflow as did Hare. Where the Hare Charts are used with Q_g/Q_o = 0, the results can be compared with those of Cade and Thompson.

For the same values of D_u, D_o, Q_u & Q_o, three situations can be examined:

1. Hare – intersection point on downstream face.
2. Hare – intersection point not on downstream face.
3. Cade and Thompson – intersection point at centre of structure.
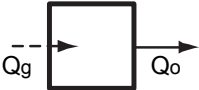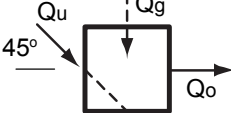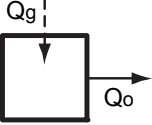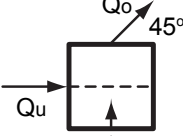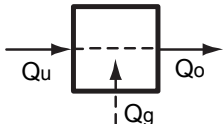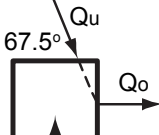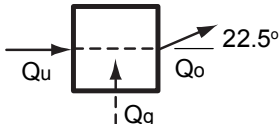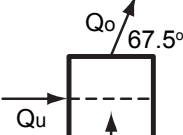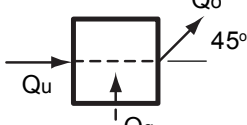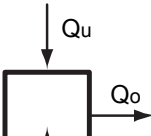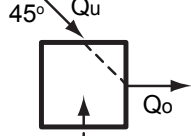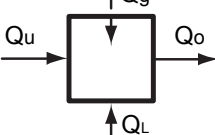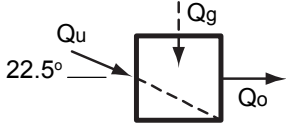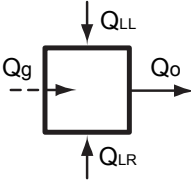
For the examples examined, the value in case 3 was almost always approximately equal to the average of cases 1 and 2.

It is more likely in practice that intersection at the access chamber centre-line will occur and that B/D_o will be less than 2.

Qudm

Structure schematic diagram	Applicable chart		Structure schematic diagram	Applicable chart	
	Ku	Kw		Ku	Kw
Qg → Qo **Kg = Kw**	A2-3	A2-3	45° Qu, Qg → Qo **Ku ≠ Kw**	A2-12 and A2-13	A2-14 and A2-15
Qg ↓ Qo **Kg = Kw**	A2-3	A2-3	Qo 45°, Qu →, Qg ↑ **Ku ≠ Kw**	A2-16 and A2-17	A2-18 and A2-19
Qu → Qo, Qg ↑ **Ku = Kw**	A2-4	A2-4	67.5° Qu, Qo →, Qg ↑ **Ku ≠ Kw**	A2-20 and A2-21	A2-22 and A2-23
Qu →, 22.5° Qo, Qg ↑ **Ku = Kw**	A2-5	A2-5	Qo 67.5°, Qu →, Qg ↑ **Ku ≠ Kw**	A2-24 and A2-25	A2-26 and A2-27
Qo 45°, Qu →, Qg ↑ **Ku = Kw**	A2-6	A2-6	Qu ↓, Qo →, Qg ↑ **Ku ≠ Kw**	A2-28 and A2-29	A2-30 and A2-31
45° Qu, Qo →, Qg ↑ **Ku = Kw**	A2-7	A2-7	Qg ↓, Qu → Qo →, QL ↑ **Ku = Kw**	A2-32	A2-32
Qg ↓, Qu 22.5° →, Qo → **Ku ≠ Kw**	A2-8 and A2-9	A2-10 and A2-11	QLL ↓, Qg → Qo →, QLR ↑	A2-33	A2-33

Index to pressure change coefficient charts

Chart No. A2-1

Structure schematic diagram	Applicable chart		Structure schematic diagram	Applicable chart	
	K_u	K_w		K_u	K_w
Q_n / Q_g / Q_o / Q_f $K_u \neq K_w$	A2-34	A2-34	Q_u $\rightarrow$ Q_o, $45°$ Q_L	A2-42	N/A
Q_u / $\theta°$ / Q_o $K_u \neq K_w$	A2-35	A2-35	Q_u $\rightarrow$ Q_o, $60°$ Q_L	A2-43	N/A
D_u / HGL / D_o / t_i $K_u \neq K_w$	A2-36	A2-36	Q_u $\rightarrow$ Q_o, $90°$ Q_L	A2-44	N/A
Q_L / Q_u / Q_o $K_u = K_w$	A2-37 and A2-38	A2-37 and A2-38			
Q_L / Q_u / Q_o $K_u = K_w$ For $Q_u/Q_o \times D_o/D_u > 1$ For $D_L/D_o < 0.6$	A2-39	A2-39			
Q_u $\rightarrow$ Q_o, $15°$ Q_L	A2-40	N/A			
Q_u $\rightarrow$ Q_o, $30°$ Q_L	A2-41	N/A			

☐ Denotes a structure with grate (gutter) inflow   ◯ Denotes a structure with no grate (gutter) inflow

Index to pressure and energy loss analysis charts

Chart No. A2-2

Chart A2-3

This chart has been derived from the original chart of Sangster et al. (1958) based upon work by Ipswich City Council to give a direct solution to the value of K_g rather than by iterative methods using the original chart.

The curves prepared by Sangster et al. represent envelopes below which all test results reside. Further testing at Queensland University of Technology for Brisbane City Council supports these envelopes although concern has been expressed at the narrow result range for each inlet size/outlet pipe configuration. Sangster et al. did report a tendency for even higher values of K_g for very low submergence ratios (S/D_o) and that the tests were unstable and the data difficult to observe. Those wishing to use the original chart and the iterative solution are referred to Department of Transport (1992).

The coefficient presented in this modified chart is used to determine the WSE in a rectangular inlet with all inflow entering through a grate or side inlet. The coefficient K_g depends on the outflow pipe direction relative to the direction of inflow and the depth of water in the inlet.

To use Chart A2-3:
1. Note whether the direction of the outflow pipe is parallel to the direction of the inflow (Curve A) or at right angles to the direction of inflow (Curve B). Note that for an angle of 15° or more, Curve B should be used. There should be no interpolation between the two curves.
2. Determine the outflow pipe HGL elevation.
3. Calculate the distance between the outflow pipe obvert and the HGL elevation for the outflow pipe (H).
4. Calculate the ratio (H/D_o).
5. Calculate the outfall pipe velocity head ($V_o^2/2g$).
6. Calculate the ratio ($V_o^2/(2g.D_o)$).
7. Locate the appropriate ($V_o^2/(2g.D_o)$) line (which will pass through the Pivot Point).
8. Construct a line parallel to this line at a distance (H/D_o) from the Pivot Point.
9. At the intersection of this line with curves A or B as appropriate, determine the value of K_g
10. Calculate $h_g = K_g.V_o^2/2g$
11. Determine WSE = HGL (outflow pipe) + h_g
12. Check that the WSE meets the freeboard criteria.

Example 1:
> D_o = 450 mm
> V_o = 1.5 m/s
1. Outflow pipe at 90° to inflow – use Curve B
2. HGL at pipe obvert
3. $H = 0$
4. $H/D_o = 0$
5. $V_o^2/2g$ = 0.115 m
6. $V_o^2/(2g.D_o)$ = 0.26
7. Locate $V_o^2/(2g.D_o)$ = 0.26 line
8. $H/D_o = 0$; use line from step 7
9. K_g = 5.22
10. h_g = 5.22 $V_o^2/2g$ = 0.600 m

11. WSE = 1.050 m (i.e. $D_o + H + h_g$) above pipe invert

Example 2:

D_o = 450 mm

V_o = 1.5 m/s

1. Outflow pipe at 90° to inflow – use Curve B
2. HGL at a height of 0.45 m above invert
3. H = 0.45
4. H/D_o = 1.0
5. $V_o^2/2g$ = 0.115 m
6. $V_o^2/(2g.D_o)$ = 0.26
7. Locate line for: $V_o^2/(2g.D_o)$ = 0.26
8. Draw line through H/D_o = 1.0 parallel to line from step 7
9. K_g = 3.75
10. h_g = 3.75 $V_o^2/2g$ = 0.431 m
11. WSE = 1.331 m (i.e. $D_o + H + h_g$) above pipe invert
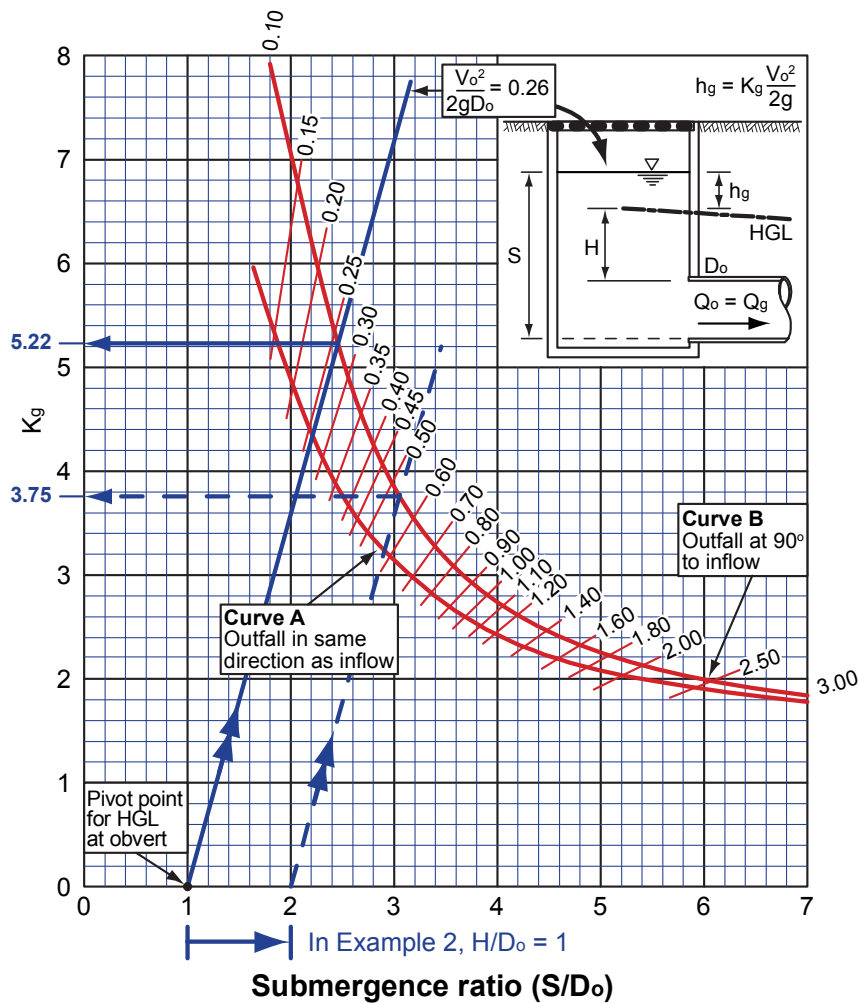
Figure A2-1 – Chart outcomes for examples 1 and 2

Chart elements:

$$\frac{V_o^2}{2gD_o}$$

$$Q_g$$

$$h_g = K_g \frac{V_o^2}{2g}$$

h_g

HGL

S

H

D_o

$Q_o = Q_g$

K_g (vertical axis)

0.10, 0.15, 0.20, 0.25, 0.30, 0.35, 0.40, 0.45, 0.50, 0.60, 0.70, 0.80, 0.90, 1.00, 1.10, 1.20, 1.40, 1.60, 1.80, 2.00, 2.50, 3.00

Curve B
Outfall at 90° to inflow

Curve A
Outfall in same direction as inflow

Pivot point for HGL at obvert

Measure H/D_o from here

Submergence ratio (S/D_o)

Q_g — D_o — Curve A

Q_g — D_o — Curve B
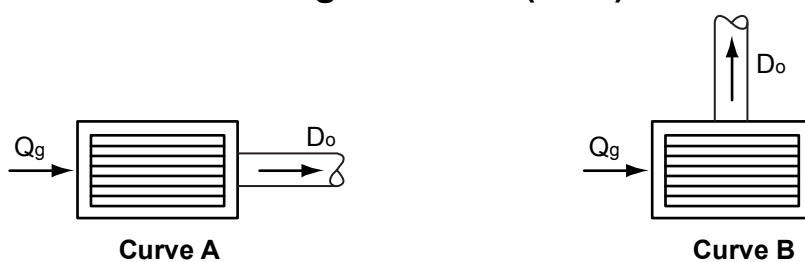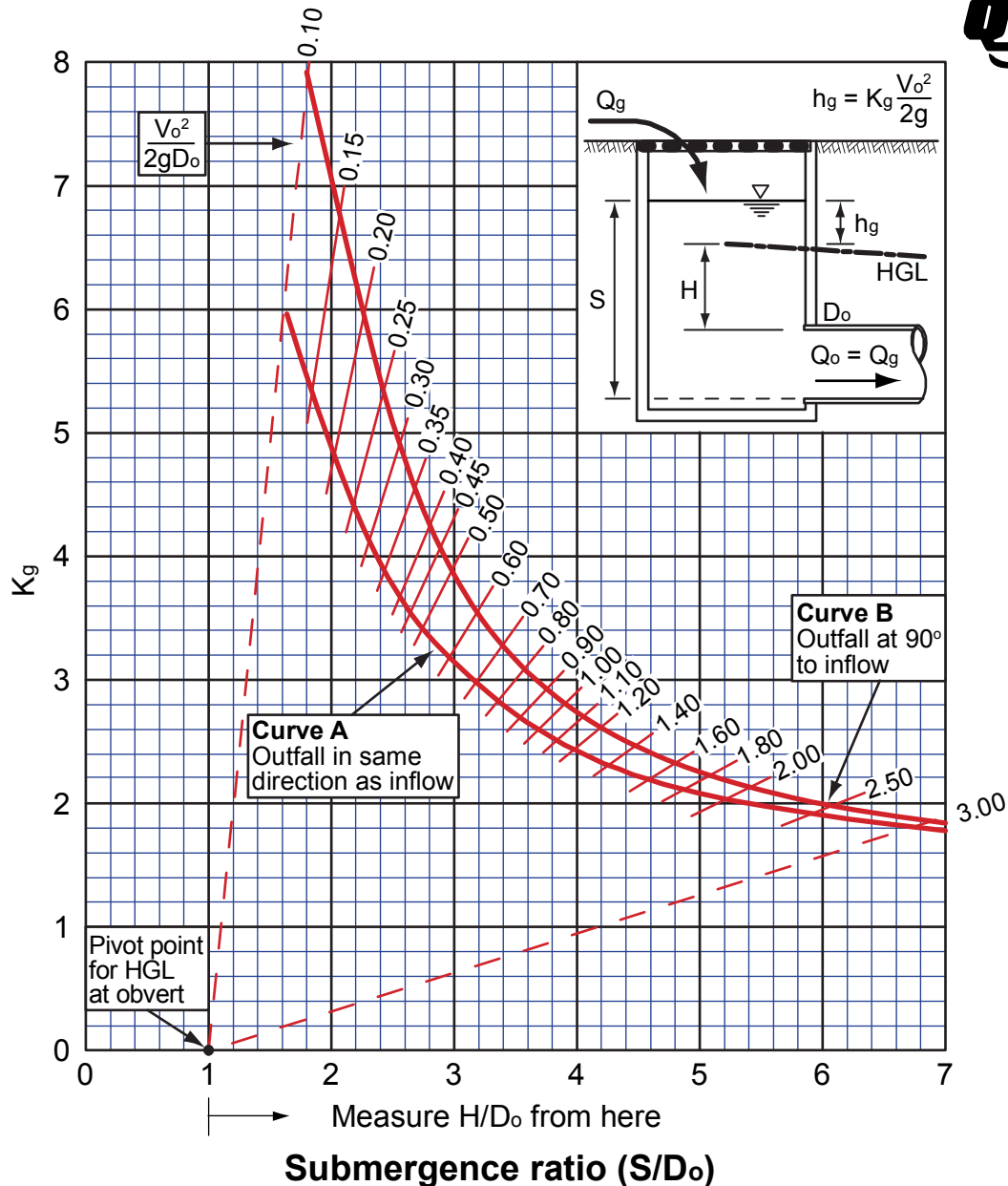
Pressure head change coefficients for rectangular inlet with grate flow only modified from DOT (1992)

Notes:
1. For a *Side inlet,* the inflow direction should be taken as the direction of flow in the kerb and channel.
2. Where the outflow direction is within 15 degrees of the direction of the direction of inflow, use *Curve A*.
3. Where the outflow direction is greater than 15 degrees from the direction of inflow, use *Curve B*.
4. $K_w = K_g$

Chart No. A2-3

Charts A2-4 to A2-7

The coefficients presented in these charts are used to determine the HGL and WSE in a square inlet with inflow both from an upstream pipe and a grate or side inlet. The coefficient depends on the ratio of grate inflow to outflow (Q_g/Q_o), submergence ratio (S/D_o) and deflection angle. The ratio of structure width to outfall diameter (B/D_o) used in the tests was approximately equal to 2. The coefficients K_u and K_w will **not** be greater for B/D_o <2.

A correction factor for K_u and K_w may be obtained from Table A2-1 for submergence ratios (S/D_o) not equal to 2.5. This correction factor is applicable to the pits represented in charts A2-4, A2-5, A2-10 and A2-11.

Table A2.1 – Correction factor for K_u and K_w for submergence ratio (S/D_o) not equal to 2.5

S/D_o	Q_g/Q_o					
	0.00	0.10	0.20	0.30	0.40	0.50
1.5	0.00	0.11	0.22	0.33	0.44	0.55
2.0	0.00	0.04	0.08	0.12	0.16	0.20
2.5	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	-0.03	-0.60	-0.09	-0.12	-0.15
3.5	0.00	-0.04	-0.80	-0.12	-0.16	-0.20
4.0	0.00	-0.05	-0.10	-0.15	-0.20	-0.25

Note (Table A2.1)

[1]   Correction values are to be **added** to K_u and K_w for a submergence ratio (S/D_o) not equal to 2.5.

To use the charts:

1. Determine the outfall pipe HGL elevation
2. Calculate the outfall pipe velocity head ($V_o^2/2g$)
3. Calculate D_u/D_o
4. Calculate Q_g/Q_o
5. Read K_u from charts
6. Calculate S = HGL (outfall) - I.L. Outfall Pipe + $K_u .V_o^2/2g$
7. Calculate S/D_o, then add the adjustment from Table A2-1 to K_u and K_w
8. Calculate $h_u = K_u .V_o^2/2g$

    $h_w = K_w .V_o^2/2g$
9. Calculate WSE = HGL (outfall) + h_w
10. Check that WSE meets the freeboard criteria
11. Calculate HGL (upstream) = HGL (outfall) + h_u

Chart No. A2-4
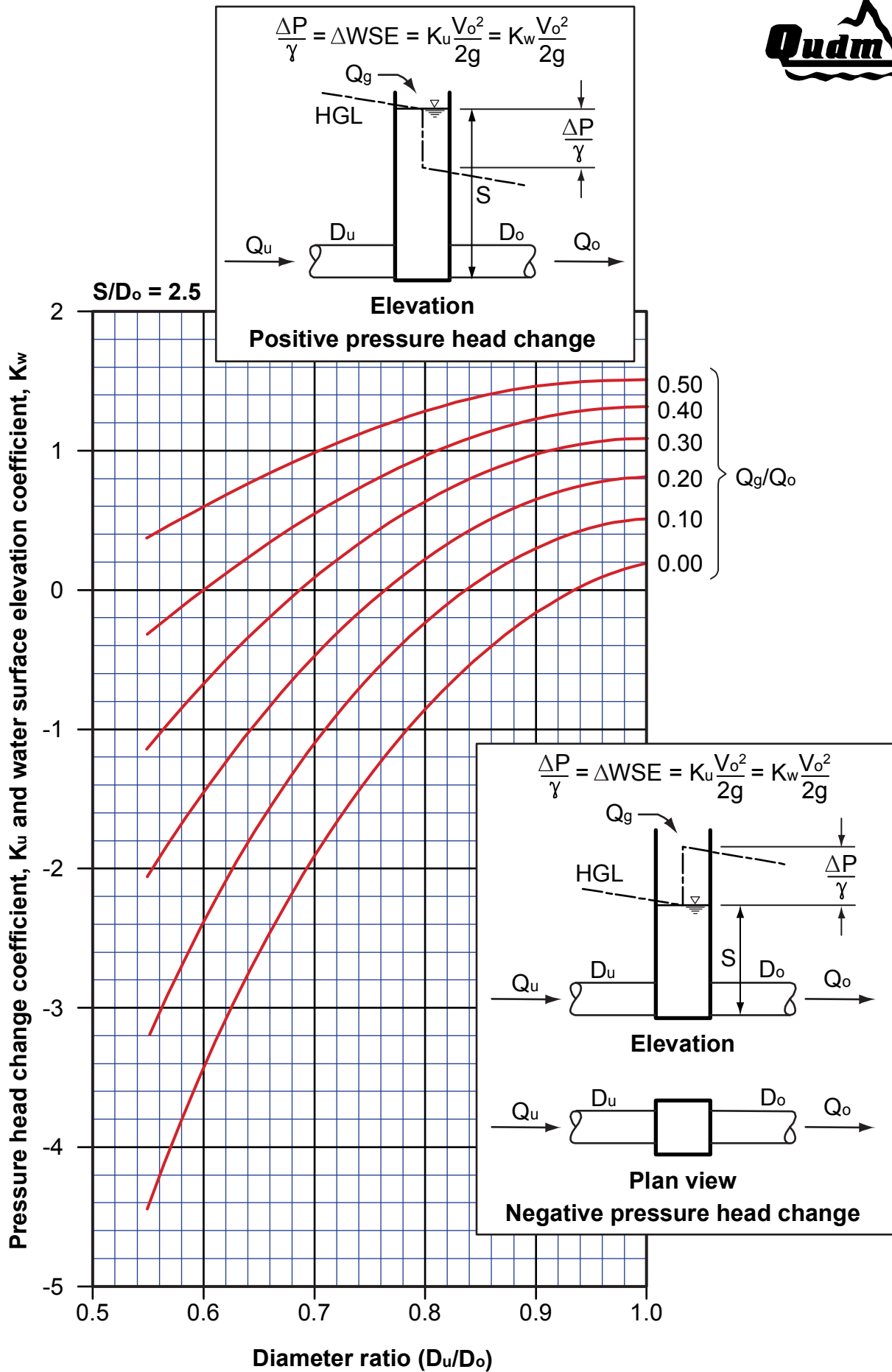
Pressure head change and water surface elevation coefficients for straight through flow for submergence ratio, $S/D_o = 2.5$ (Source: Hare, 1980)

The chart axes and labels:

- Y-axis (left): Pressure head change coefficient, K_u and water surface elevation coefficient, K_w, ranging from -5 to 2
- X-axis (bottom): Diameter ratio (D_u/D_o), ranging from 0.5 to 1.0
- Top-left label: $S/D_o = 2.5$

Curve labels (right side), Q_g/Q_o:
- 0.50
- 0.40
- 0.30
- 0.20
- 0.10
- 0.00

Top inset (Positive pressure head change):

$$\frac{\Delta P}{\gamma} = \Delta WSE = K_u \frac{V_o^2}{2g} = K_w \frac{V_o^2}{2g}$$

Labels: Q_g, HGL, $\frac{\Delta P}{\gamma}$, S, Q_u, D_u, D_o, Q_o

Elevation
Positive pressure head change

Bottom inset (Negative pressure head change):

$$\frac{\Delta P}{\gamma} = \Delta WSE = K_u \frac{V_o^2}{2g} = K_w \frac{V_o^2}{2g}$$

Labels: Q_g, HGL, $\frac{\Delta P}{\gamma}$, S, Q_u, D_u, D_o, Q_o

Elevation

Plan view
Negative pressure head change

Qudm

Pressure head change and water surface elevation coefficients for 22.5° bends at pit junctions, with branch point on downstream face of pit, and for a submergence ratio S/D$_o$ = 2.5 (Source: Hare, 1980)
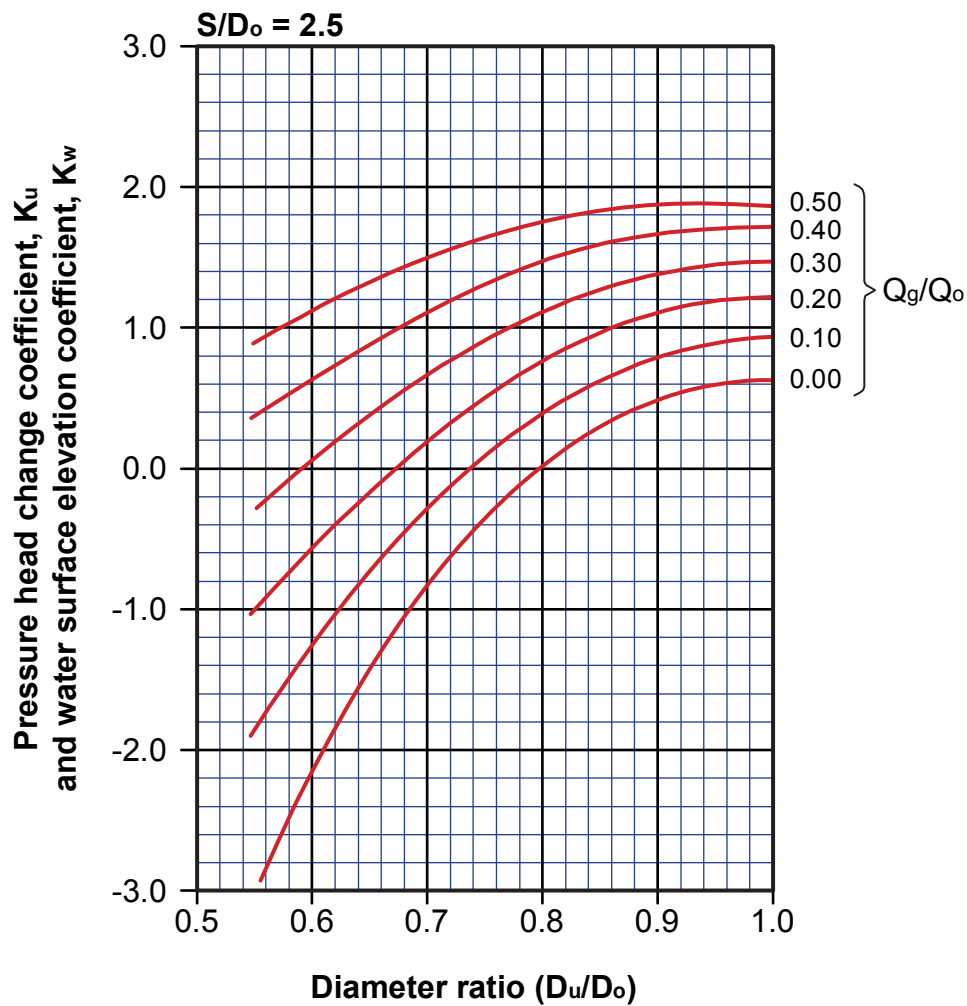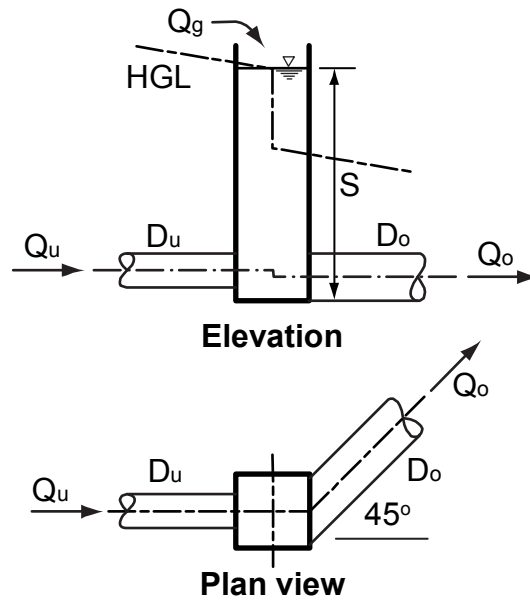
Chart No. A2-5

Elevation

Plan view

S/Dₒ = 2.5

Pressure head change coefficient, K_u and water surface elevation coefficient, K_w

Diameter ratio (D_u/D_o)

0.50
0.40
0.30
0.20
0.10
0.00

Q_g/Q_o

Pressure head change and water surface elevation coefficients for 45° bends at pit junctions with branch point located on downstream face of pit for a submergence ratio, S/Dₒ = 2.5 (Source: Hare, 1980)

Chart No. A2-6

Elevation

Plan view

Pressure head change and water surface elevation coefficients for 45° bends at pit junctions with branch point located on downstream face of pit for a submergence ratio, S/Do = 2.5 (Source: Hare, 1980)

Chart No. A2-7
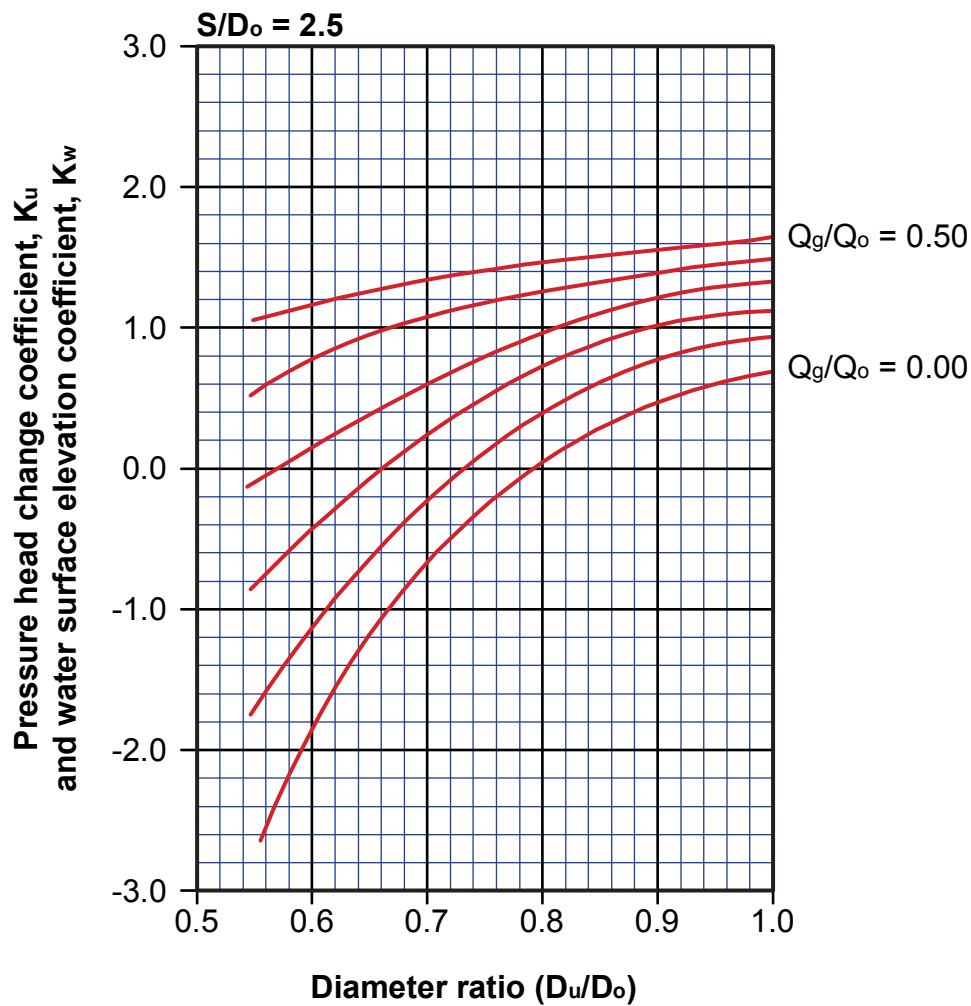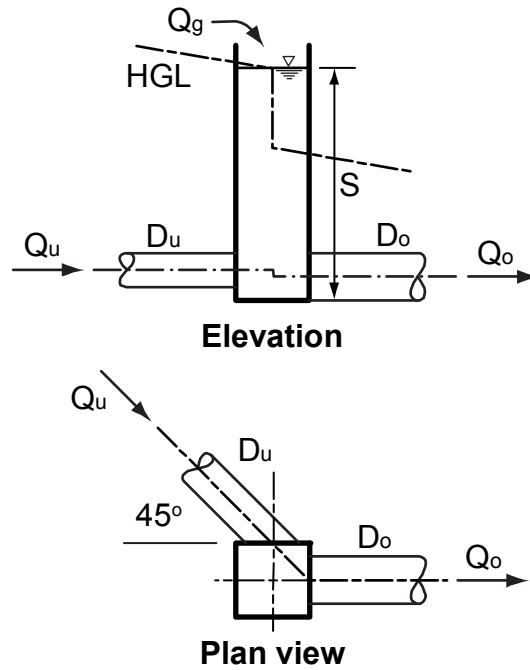
Charts A2-8 to A2-31

The following sets of design charts apply to junction chambers where the expected water surface elevation (WSE) sits above the expected hydraulic grade line (HGL), thus $K_w \neq K_u$. Each type of junction chamber has four related design charts, two for the assessment of K_u and two for the assessment of K_w.

Table A2.2 – Design chart groupings

Junction chamber description	HGL analysis	WSE analysis
22.5° junction with branch point located on upstream face of pit	A2-8 & A2-9	A2-10 & A2-11
45° junction with branch point located on upstream face of pit	A2-12 & A2-13	A2-14 & A2-15
45° junction with branch point located on upstream face of pit (alt)	A2-16 & A2-17	A2-18 & A2-19
67.5° junction with branch point located on downstream face of pit	A2-20 & A2-21	A2-22 & A2-23
67.5° junction with branch point located on upstream face of pit	A2-24 & A2-25	A2-26 & A2-27
90° pit junction	A2-28 & A2-29	A2-30 & A2-31

HGL analysis charts (A2-8, A2-9; A2-12, A2-13; A2-16, A2-17; A2-20, A2-21; A2-24, A2-25; and A2-28, A2-29)

The coefficients presented in these charts are used to determine HGL in a square inlet with inflow from both an upstream pipe and a grate or side inlet. The coefficient depends on the ratio of grate inflow to outflow (Q_g/Q_o), submergence ratio (S/D_o) and deflection angle. The ratio of structure width to outfall diameter (B/D_o) used in the tests was approximately 2. The coefficient K_u will not be greater for $B/D_o < 2$.

To interpolate these charts for $0.0 < Q_g/Q_o < 0.5$ the following equation is used:

$$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where $C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Otherwise, for values of C_g refer to Table A2.3.

Table A2.3 – Values of coefficient C_g

Q_g/Q_o	Q_u/Q_o	C_g	Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00	0.20	0.80	0.48
0.45	0.55	0.93	0.15	0.85	0.37
0.40	0.60	0.85	0.10	0.90	0.25
0.35	0.65	0.77	0.05	0.95	0.13
0.30	0.70	0.68	0.00	1.00	0.00
0.25	0.75	0.58			

To use the charts:

1.   Determine the outfall pipe HGL elevation

2.   Calculate the outfall pipe velocity head ($V_o^2/2g$)

3.   Calculate D_u/D_o

4.   Calculate Q_g/Q_o

5.   Calculate S = HGL (outfall) - I.L. Outfall Pipe + $K_w . V_o^2/2g$ (i.e. K_w must be estimated)

6. Enter the chart for the S/D_o closest to the calculated value to determine the value of K_u interpolating for the appropriate Q_g/Q_o ratio in accordance with the interpolation table (Table A2.2).

7. Calculate $h_u = K_u . V_o^2/2g$

8. Calculate HGL (upstream) = HGL (outfall) + h_u

9. Check HGL elevation ≤ WSE

WSE analysis charts (A2-10, A2-11; A2-14, A2-15; A2-18, A2-19; A2-22, A2-23; A2-26, A2-27; and A2-30, A2-31)

The coefficients presented in these charts are used to determine WSE in a square inlet with inflow from both an upstream pipe and a grate or side inlet. The coefficient depends on the ratio of grate inflow to outflow (Q_g/Q_o), submergence ratio (S/D_o) and deflection angle. The ratio of structure width to outfall diameter (B/D_o) used in the tests was approximately 2. The coefficient K_w will not be greater for $B/D_o < 2$.

To interpolate these charts for $0.0 < Q_g/Q_o < 0.5$ the following equation is used:

$$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where $C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Otherwise, for values of C_g refer to Table A2.3.

To use the charts:

1. Determine the outfall pipe HGL elevation

2. Calculate the outfall pipe velocity head ($V_o^2/2g$)

3. Calculate D_u/D_o

4. Calculate Q_g/Q_o

5. Assume $K_w = 1.0$

6. Calculate S = HGL (outfall) - I.L. Outfall Pipe + $K_w . V_o^2/2g$

7. Calculate S/D_o. Enter the chart for the S/D_o closest to the calculated value to determine the value of K_w interpolating for the appropriate Q_g/Q_o ratio in accordance with the interpolation table (Table A2.2).

8. Iterate steps 6 and 7 to improve the estimation of S and therefore K_w

9. Calculate $h_w = K_w . V_o^2/2g$

10. Determine WSE = HGL (outfall) + h_w

11. Check that WSE meets the freeboard criteria

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

**Pressure head change coefficients (K_u)
for 22.5° bends at pit junctions with branch
point located on the upstream face of pit
for a submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-8

Pressure head change coefficients (K_u) for
22.5° bends at pit junctions with branch point
located on the upstream face of pit for
submergence ratios S/D_o = 1.5, 2.0, 3.0 and 4.0
(Source: Hare, 1980)

Chart No. A2-9

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Water surface elevation coefficients (K_w) for 22.5° bends at pit junctions with branch point located on the upstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

Chart No. A2-10

Water surface elevation coefficients (Kw) for 22.5° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-11

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Pressure head change coefficients (K_u) for 45° bends at pit junctions with branch point located on the upstream face of pit for a submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)

Chart No. A2-12

Pressure head change coefficients (K_u) for 45° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios S/D_o = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-13

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

**Water surface elevation coefficients (K_w) for
45° bends at pit junctions with branch point
located on the upstream face of pit for a
submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-14

Water surface elevation coefficients (Kw) for 45° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-15

To interpolate the Hare charts for 0.0 < Q_g/Q_o < 0.5
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

**Pressure head change coefficients (K_u) for
45° bends at pit junctions with branch point
located on the upstream face of pit for a
submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-16

Pressure head change coefficients (Ku) for 45° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-17

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

**Water surface elevation coefficients (K_w) for
45° bends at pit junctions with branch point
located on the upstream face of pit for a
submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-18

S/Dₒ = 1.5

$Q_g/Q_o = 0.50$
$Q_g/Q_o = 0.00$

S/Dₒ = 2.0

$Q_g/Q_o = 0.00$
$Q_g/Q_o = 0.50$

Water surface elevation coefficient K_w

S/Dₒ = 3.0

$Q_g/Q_o = 0.00$
$Q_g/Q_o = 0.50$

S/Dₒ = 4.0

$Q_g/Q_o = 0.00$
$Q_g/Q_o = 0.50$

Diameter ratio D_u/D_o

Water surface elevation coefficients (K_w) for 45° bends at pit junctions with branch point located on the upstream face of pit for submergence ratios S/Dₒ = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-19

To interpolate the Hare charts for 0.0 < Q_g/Q_o < 0.5
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

**Pressure head change coefficients (K_u)
for 67.5° bends at pit junctions with branch
point located near the downstream face of
pit for a submergence ratio S/D_o = 2.5
(Source: Hare, 1980)**

Chart No. A2-20

Pressure head change coefficients (Ku) for 67.5° bends at pit junctions with branch point located near the downstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-21

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$ the following equation is used:

$$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Water surface elevation coefficients (K_w) for 67.5° bends at pit junctions with branch point located near the downstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

Chart No. A2-22

Water surface elevation coefficients (Kw) for 67.5° bends at pit junctions with branch point located near the downstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-23

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

**Pressure head change coefficients (K_u)
for 67.5° bends at pit junctions with branch
point located near the upstream face of
pit for a submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-24

Pressure head change coefficients (K_u) for 67.5° bends at pit junctions with branch point located near the upstream face of pit for submergence ratios S/D_o = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-25

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$
the following equation is used:

$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Water surface elevation coefficients (K_w) for 67.5° bends at pit junctions with branch point located near the upstream face of pit for a submergence ratio $S/D_o = 2.5$ (Source: Hare, 1980)

Chart No. A2-26

Water surface elevation coefficients (Kw) for 67.5° bends at pit junctions with branch point located near the upstream face of pit for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-27

To interpolate the Hare charts for 0.0 < Q_g/Q_o < 0.5
the following equation is used:

$K_u = K_{0.0} + C_g (K_{0.5} - K_{0.0})$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_u$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_u$ for $Q_g/Q_o = 0.5$

| Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

**Pressure head change coefficients (K_u)
for 90° bends at pit junctions for a
submergence ratio $S/D_o = 2.5$
(Source: Hare, 1980)**

Chart No. A2-28

Pressure head change coefficents (K_u) for 90° bends at pit junctions for submergence ratios S/D_o = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-29

To interpolate the Hare charts for $0.0 < Q_g/Q_o < 0.5$ the following equation is used:

$$K_w = K_{0.0} + C_g (K_{0.5} - K_{0.0})$$

where:

$C_g = 1.33 [1 - (Q_u/Q_o)^2]$

$K_{0.0} = K_w$ for $Q_g/Q_o = 0.0$

$K_{0.5} = K_w$ for $Q_g/Q_o = 0.5$

| Q_g/Q_o	Q_u/Q_o	C_g
0.50	0.50	1.00
0.45	0.55	0.93
0.40	0.60	0.85
0.35	0.65	0.77
0.30	0.70	0.68
0.25	0.75	0.58
0.20	0.80	0.48
0.15	0.85	0.37
0.10	0.90	0.25
0.05	0.95	0.13
0.00	1.00	0.00

Elevation

Plan view

S/D$_o$ = 2.5

$Q_g/Q_o = 0.50$

$Q_g/Q_o = 0.00$

Water surface elevation coefficient (K$_w$)

Diameter ratio (D$_u$/D$_o$)

Water surface elevation coefficients (K$_w$) for 90° bends at pit junctions for a submergence ratio, S/D$_o$ = 2.5 (Source: Hare, 1980)

Chart No. A2-30

Water surface elevation coefficients (Kw) for 90° bends at pit junctions for submergence ratios S/Do = 1.5, 2.0, 3.0 and 4.0 (Source: Hare, 1980)

Chart No. A2-31

Chart A2-32

The coefficients presented in this chart are used to determine HGL for the upstream and the lateral pipes and the WSE for the structure with inflow from an upstream and a lateral pipe and a grate or side inlet. The coefficients depend on the ratio of grate inflow to outflow (Q_g/Q_o), submergence ratio S/D_o) and diameter ratio (D_u/D_o). The main graph of the chart applies directly for no flow into the inlet through the grate. Increments of K_u and K_L for grate flow conditions are shown in the supplementary graphs of the upper portion of the chart.

To use the chart:

1. Determine the outfall pipe HGL elevation

2. Calculate the velocity head in the outfall pipe ($V_o^2/2g$)

3. Calculate the ratios D_u/D_o, Q_u/Q_o, and Q_g/Q_o

4. If no grate flow is involved, use charts A1-24 or A1-25 as appropriate

5. With grate flow, assume K_u (or K_L) = 1.5

6. Calculate S = HGL (outfall) - I.L. Outfall Pipe + K_u (or K_L). $V_o^2/2g$

7. Calculate S/D_o

8. Determine K_u (or K_L) as in step 4 from the lower graph for Q_g/Q_o = 0

9. Using the appropriate upper graph for the particular S/D_o nearest that estimated in step 7 for the appropriate values of D_u/D_o and Q_g/Q_o, and determine the increment of K_u (or K_L). This increment accounts for the effects of grate flow and is always a positive value, even when K_u (or K_L) of step 8 is negative.

10. K_u (or K_L) = sum of K_u (step 8) and K_u (or K_L) increment (step 9). Note that in unusual cases the total value of K_u may be negative.

11. Calculate $h_u = h_L = K_u$ (or K_L) .$V_o^2/2g$

12. HGL (upstream) = HGL (outfall) + h_L

13. WSE = HGL (upstream)

14. Repeat steps 6 to 12 to improve the estimate of S, S/D_o and K_u (or K_L)

15. Check that WSE meets the freeboard criteria
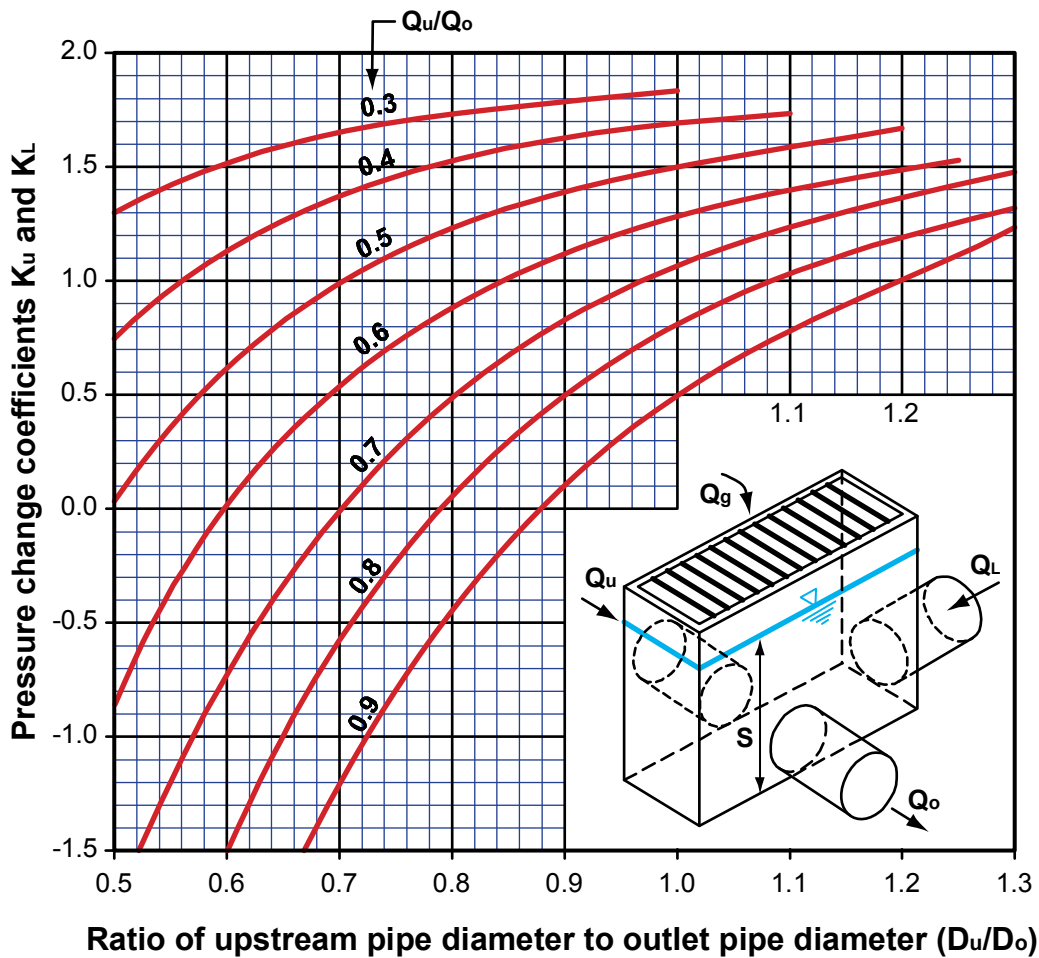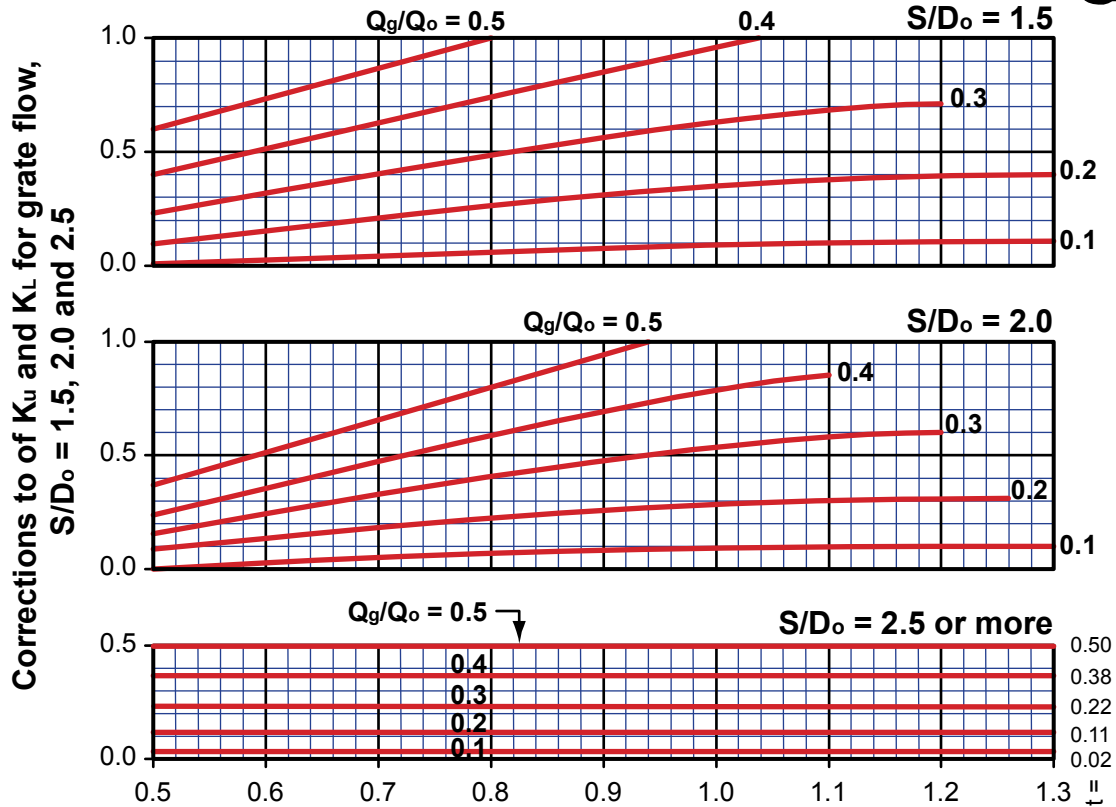
Rectangular inlet with in-line upstream main and 90° lateral pipe, with or without grate flow (Source: DOT, 1992)

Chart No. A2-32

Chart A2-33 (opposing laterals)

The coefficients presented in this chart are used to determine the HGL for each lateral pipe and the WSE for the structure with inflow from two directly opposing laterals and a grate or side inlet. The coefficients depend on the ratios of lateral inflow to outflow (Q_{HV}/Q_o, Q_{LV}/Q_o) and the diameter ratios (D_{HV}/D_o, D_{LV}/D_o, D_{HV}/D_{LV}) and on whether there is grate inflow.

D_{HV} = diameter of lateral pipe with the higher velocity flow
Q_{HV} = flow rate in lateral pipe with the higher velocity flow
D_{LV} = diameter of lateral pipe with the lower velocity flow
Q_{LV} = flow rate in lateral pipe with the lower velocity flow

The pressure change coefficient for the higher-velocity lateral is a constant (K_{HV} = 1.8 with grate flow involved, and with no grate flow, K_{HV} = 1.6) and so is not read from the chart.

To find K_{LR} or K_{LL} for the right or left lateral pipe with flow at a lesser velocity than the other lateral, read 'H' for higher velocity lateral, then read 'L' for the lower velocity lateral, then:

K_{LR} (or K_{LL}) = H – L

Note: When Q_g = 0.0, reduce K_{LL} and K_{LR} by 0.2.

Note: The left lateral (K_{LL}) could be either the 'higher velocity' lateral (in which case K_{LL} = K_{HV}) or the 'lower velocity' lateral (in which case K_{LL} = K_{LV}).

WSE = Elevation of HGL for the higher velocity lateral.

For this type of inlet and junction, the coefficients are not modified by the depth of water in the inlet.

To use the chart:

1. Determine the outfall pipe HGL elevation

2. Calculate the velocity head in the outfall pipe ($V_o^2/2g$)

3. Calculate the velocities in each of the laterals to determine which is the higher velocity and which is the lower velocity lateral

4. Calculate the ratios Q_{HV}/Q_o, Q_{LV}/Q_o, D_{HV}/D_o, D_{LV}/D_o and D_{HV}/D_{LV}

5. Determine H from the left-hand graph on the chart. Enter the graph at the diameter ratio D_{HV}/D_o (note the two scales) and read H for the appropriate value of Q_{HV}/Q_o. In entering the graph, note that unequal size laterals (D_{HV}/D_{LV} not equal to 1.0) effect an offset of the scale for D_{HV}/D_o.

    For D_{HV}/D_{LV} > 1.0, use the scale for D_{HV}/D_{LV} = 1.0

    For 0.8 < D_{HV}/D_{LV} < 1.0, linearly interpolate between the scales

    For D_{HV}/D_{LV} < 0.8 use the scale for D_{HV}/D_{LV} = 0.8

6. Determine L from the right-hand graph on the chart. Enter the graph at the diameter ratio D_{LV}/D_o (note only one scale is involved) and read L for the appropriate value of Q_{LV}/Q_o.

7. Calculate K_{LV} = H – L with grate flow involved. With no grate flow, K_{LV} = (H – L) – 0.2

8. K_{HV} = 1.8 with grate flow involved, and with no grate flow, K_{HV} = 1.6

9. Calculate $h_{LV} = K_{LV} . V_o^2/2g$

    $$h_{HV} = K_{HV} . V_o^2/2g$$

10. HGL (lower velocity lateral) = HGL (outfall) + h_{LV}
    HGL (higher velocity lateral) = HGL (outfall) + h_{HV}
    Apply to left or right laterals as appropriate
11. WSE = HGL (higher velocity lateral)
12. Check that WSE meets the freeboard criteria

Left top chart: Pressure factor (H) vs Ratio of diameter of highest velocity lateral to the outfall diameter ($D_{H.V}/D_O$) for $D_{H.V}/D_{L.V} = 0.8$. Curves labelled $= Q_{H.V}/Q_O$: 0.9, 0.8, 0.7, 0.6, 0.5, 0.4, 0.3

Right top chart: Pressure factor (H) vs Ratio of diameter of highest velocity lateral to the outfall diameter ($D_{H.V}/D_O$) for $D_{H.V}/D_{L.V} = 1.0$. Curves labelled $= Q_{H.V}/Q_O$: 0.9, 0.8, 0.7, 0.6, 0.5, 0.4, 0.3

Bottom left chart: Pressure factor (L) vs Ratio of diameter of the lesser velocity lateral to the outlet diameter ($D_{L.V}/D_O$) for all values of $D_{H.V}/D_{L.V}$. Top curves labelled 0.5, 0.6, 0.7, 0.8 $= Q_{L.V}/Q_O$. Right side curves labelled $= Q_{L.V}/Q_O$: 0.8, 0.7, 0.6, 0.5, 0.4, 0.3, 0.2. Left curves labelled 0.4, 0.3, 0.2.

Notes:

When $Q_g = 0.0$, reduce K_{LL} and K_{LR} by 0.2

K_{LR} or K_{LL} for the lateral pipe with higher velocity flow is always 1.8

To find K_{LR} (or K_{LL}) for the lateral pipe with the lesser flow velocity, read 'H' for the higher velocity lateral and 'L' for the lower velocity lateral pipe, then:
K_{LR} (or K_{LL}) = H - L

$h_{LL} = K_{LL} (V_o^2/2g)$ and $h_{LR} = K_{LR} (V_o^2/2g)$

WSE = HGL for higher velocity lateral

**Rectangular pit with opposed lateral pipes each at 90°
to outlet, with or without grate inflow (Source: DOT, 1992)**

Chart No. A2-33

Chart A2-34 (offset laterals)

The coefficients presented in this chart are used to determine the HGL for each lateral pipe and the WSE for the structure with inflow from two opposing laterals (offset in plan by not less than the sum of the diameters) and a grate or side inlet. The coefficients depend on the diameter ratios (D_n/D_o) and D_f/D_o) and the ratios (Q_n/Q_o).(D_o/D_n) and (Q_f/Q_o).(D_o/D_f) and whether there is grate inflow.

For this type of inlet the pressure changes are not modified by the depth of water in the inlet.

To use the chart:

1. Determine the horizontal distance between the centrelines of the opposed flow laterals at the inlet. If more than the sum of the pipe diameters, this chart will apply.

2. Determine the outfall pipe HGL elevation

3. Calculate the velocity head in the outfall pipe ($V_o^2/2g$)

4. Calculate the ratios Q_f/Q_o, Q_n/Q_o, D_f/D_o, and D_n/D_o

5. Calculate the ratios (Q_f/Q_o).(D_o/D_f) and (Q_n/Q_o).(D_o/D_n)

6. Determine K_n and K_f for the values of the appropriate ratios determined in steps 4 and 5. These values of K_n and K_f are for an inlet with grate flow.

7. For a junction without grate flow, reduce K_n and K_f from step 6 by 0.2

8. Calculate $h_f = K_f . V_o^2/2g$

   $h_n = K_n . V_o^2/2g$

9. HGL (near lateral) = HGL (outfall) + h_n

   HGL (far lateral) = HGL (outfall) + h_f

10. WSE = HGL (far lateral)

11. Check that WSE meets the freeboard criteria

Near lateral inflow pipe

Far lateral inflow pipe
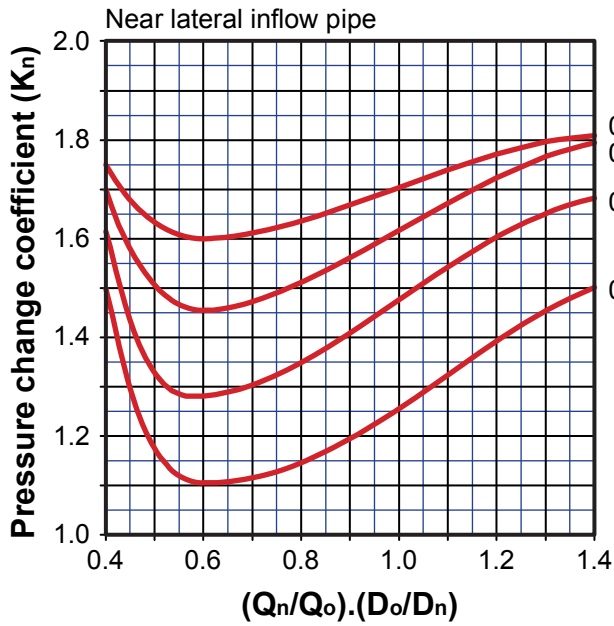
$h_n = K_n \dfrac{V_o^2}{2g}$
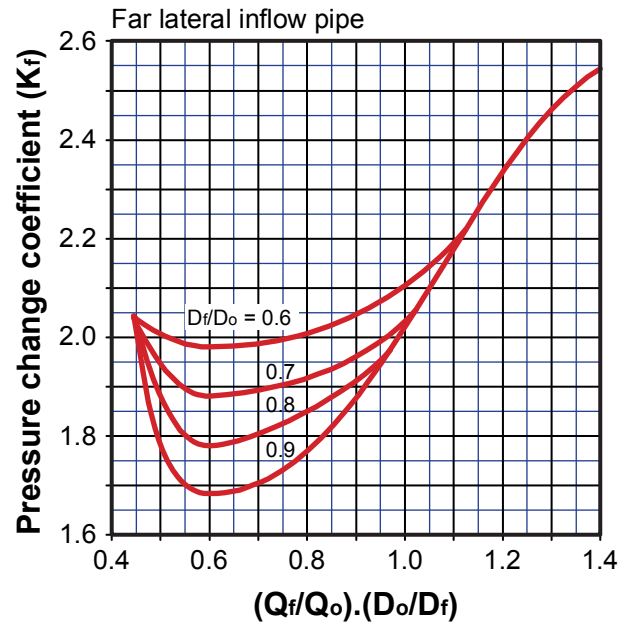
$h_f = K_f \dfrac{V_o^2}{2g}$

WSE = HGL for the far lateral
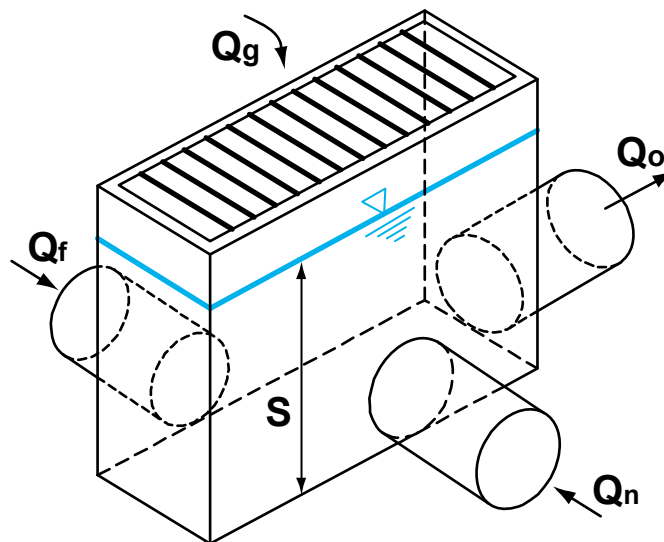
$Q_o = Q_f + Q_n + Q_g$

**Rectangular pit with offset opposed lateral pipes each at 90°
to outlet, with or without grate inflow (Source: DOT, 1992)**

Chart No. A2-34

Chart A2-35

The coefficient presented in this chart is used to determine the HGL and WSE in a circular junction pit with the upstream pipe entering at any angle between 0° and 90° to the direction of the outfall pipe. The coefficients depend on the diameter ratio (D_u/D_o) and the deflection angle.
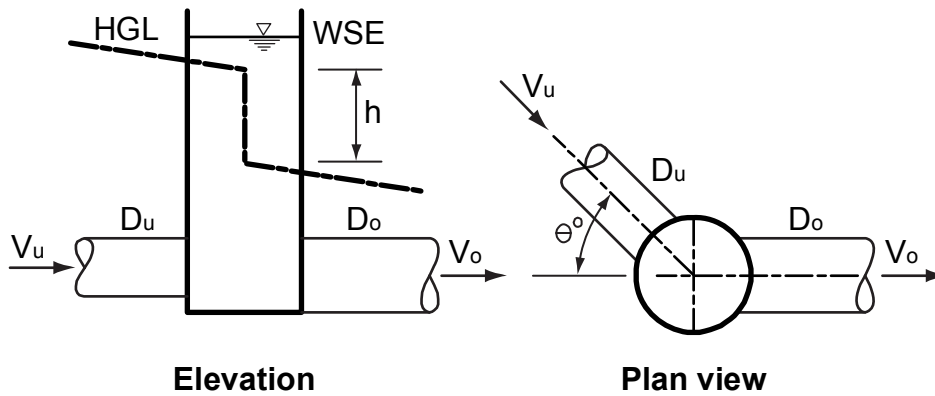
Figure A2.2 – Layout of junction pit

WSE = K_u ($V_o^2/2g$) + K_w' ($V_u^2/2g$) above the elevation of the outfall HGL

Table A2.4 – Values of K_w'

Pipeline deflection (θ)	K_w'	Pipeline deflection (θ)	K_w'
0	0.05	15	0.20
1	0.11	30	0.23
2	0.13	45	0.25
3	0.14	60	0.27
4	0.15	75	0.29
5	0.16	90	0.30
10	0.18		

To use the charts:

1. Determine the outfall pipe HGL elevation
2. Calculate the outfall velocity head ($V_o^2/2g$)
3. Calculate D_u/D_o
4. Read K_u from the chart for the appropriate deflection angle
5. Determine K_w' for the appropriate deflection angle
6. Calculate WSE = $K_u . V_o^2/2g$ + $K_w'.Vu2/2g$
7. Check that WSE meets the freeboard criteria
8. Calculate HGL (upstream) = HGL (outfall) + $K_u . V_o^2/2g$

![Chart A2-35: Pressure loss coefficients for a circular junction pit](chart)

Ratio of upstream pipe diameter to outlet diameter (D_u/D_o)

Pressure head change coefficient (K_u)

K_u

K_u

Ratio of upstream pipe diameter to outlet diameter (D_u/D_o)
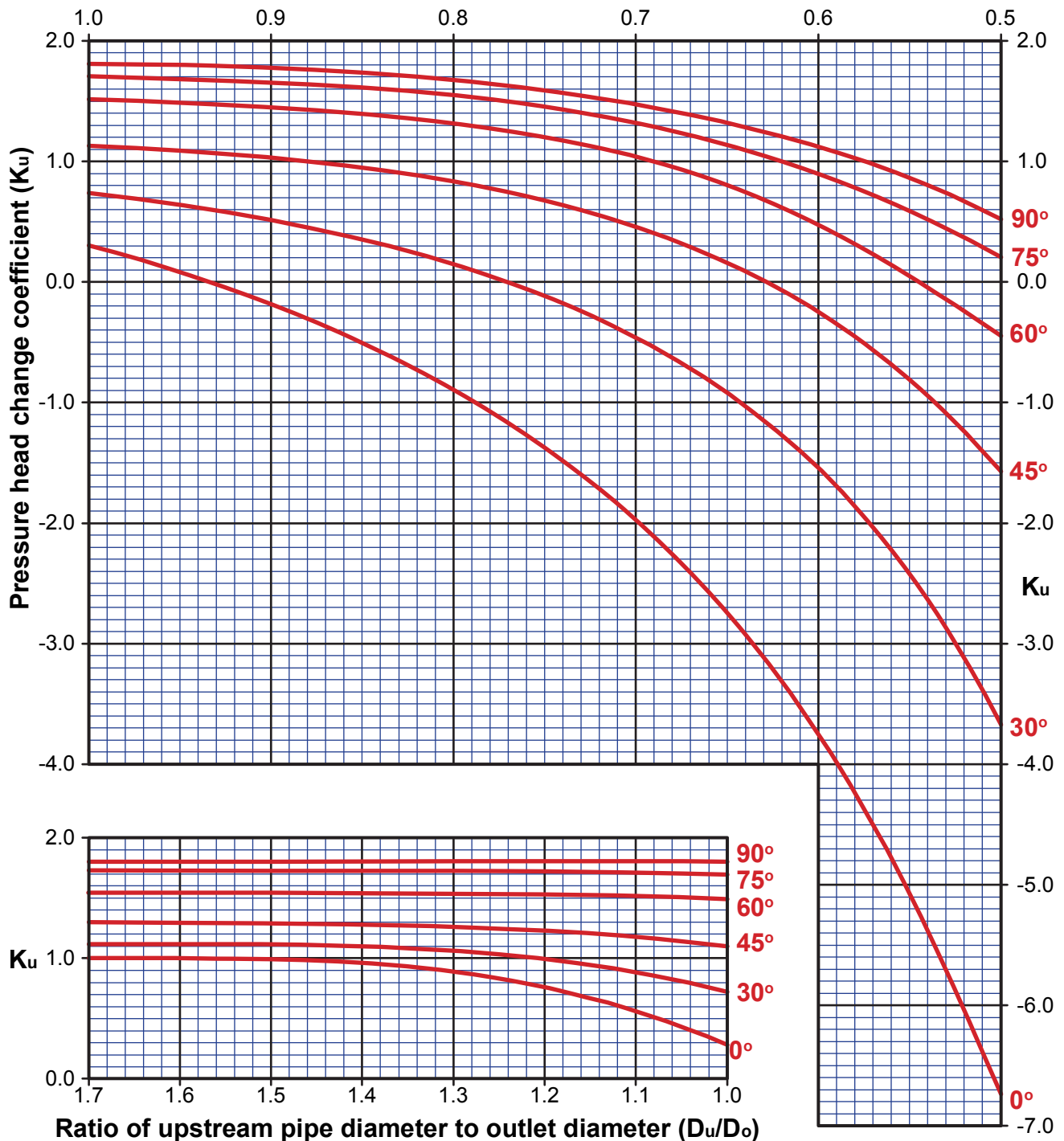
Notes:

WSE = $K_u (V_o^2/2g) + K_w' (V_u^2/2g)$ above the elevation of the outfall HGL

Refer to table in attached text for values of K_w'

**Pressure loss coefficients for a circular junction pit
with upstream pipe entering at angles from 0° to 90°
(Source: Cade and Thompson, 1982)**

Chart No. A2-35

Chart A2-36

The coefficient presented in this chart is used to determine the HGL and WSE in a circular junction pit with the upstream pipe entering at any angle between $0°$ and $90°$ to the direction of the outfall pipe and with a drop in invert between the upstream pipe and the outfall pipe of more than the difference in their diameters. The coefficients depend on the diameter ratio (D_u/D_o), drop height, deflection angle and the outfall diameter.

Elevation   **Plan view**
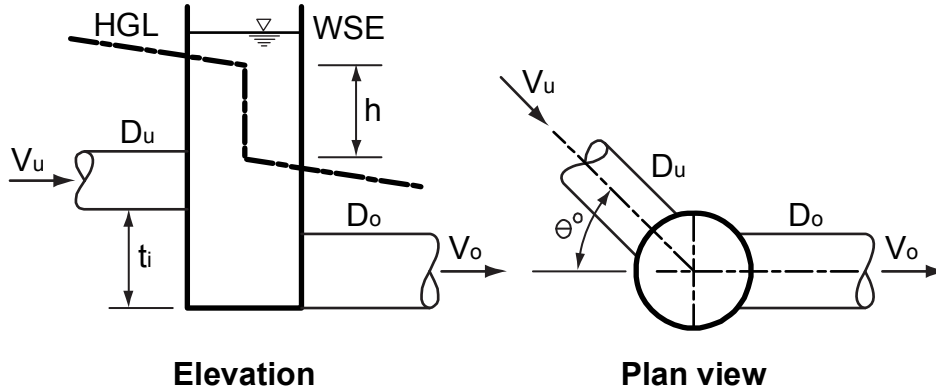
Figure A2.3 – Layout of junction pit with drop in invert from inflow to outflow

$$r/D_o = [(t_i/D_o)^2 + (\theta/57.8)^2]^{0.5}$$

where: $h = K_u (V_o^2/2g)$
θ = degrees
$t_i \geq D_o - D_u$

and WSE $= K_u (V_o^2/2g) + K_w' (V_u^2/2g)$ above the elevation of the outfall HGL

Table A2.5 – Values of K_w'

Pipeline deflection (θ)	K_w'	Pipeline deflection (θ)	K_w'
0	0.05	15	0.20
1	0.11	30	0.23
2	0.13	45	0.25
3	0.14	60	0.27
4	0.15	75	0.29
5	0.16	90	0.30
10	0.18		

To use the chart:
1. Determine the outfall pipe HGL elevation
2. Calculate the outfall velocity head ($V_o^2/2g$)
3. Calculate $r/D_o = [(t_i/D_o)^2 + (\theta/57.8)^2]^{0.5}$
4. Calculate (D_u/D_o)
5. Read K_u from the chart
6. Determine K_w' for the appropriate deflection angle
7. Calculate WSE $= K_u . V_o^2/2g + K_w'. V_o^2/2g$
8. Check that WSE meets the freeboard criteria
9. Calculate HGL (upstream) = HGL (outfall) $+ K_u . V_o^2/2g$
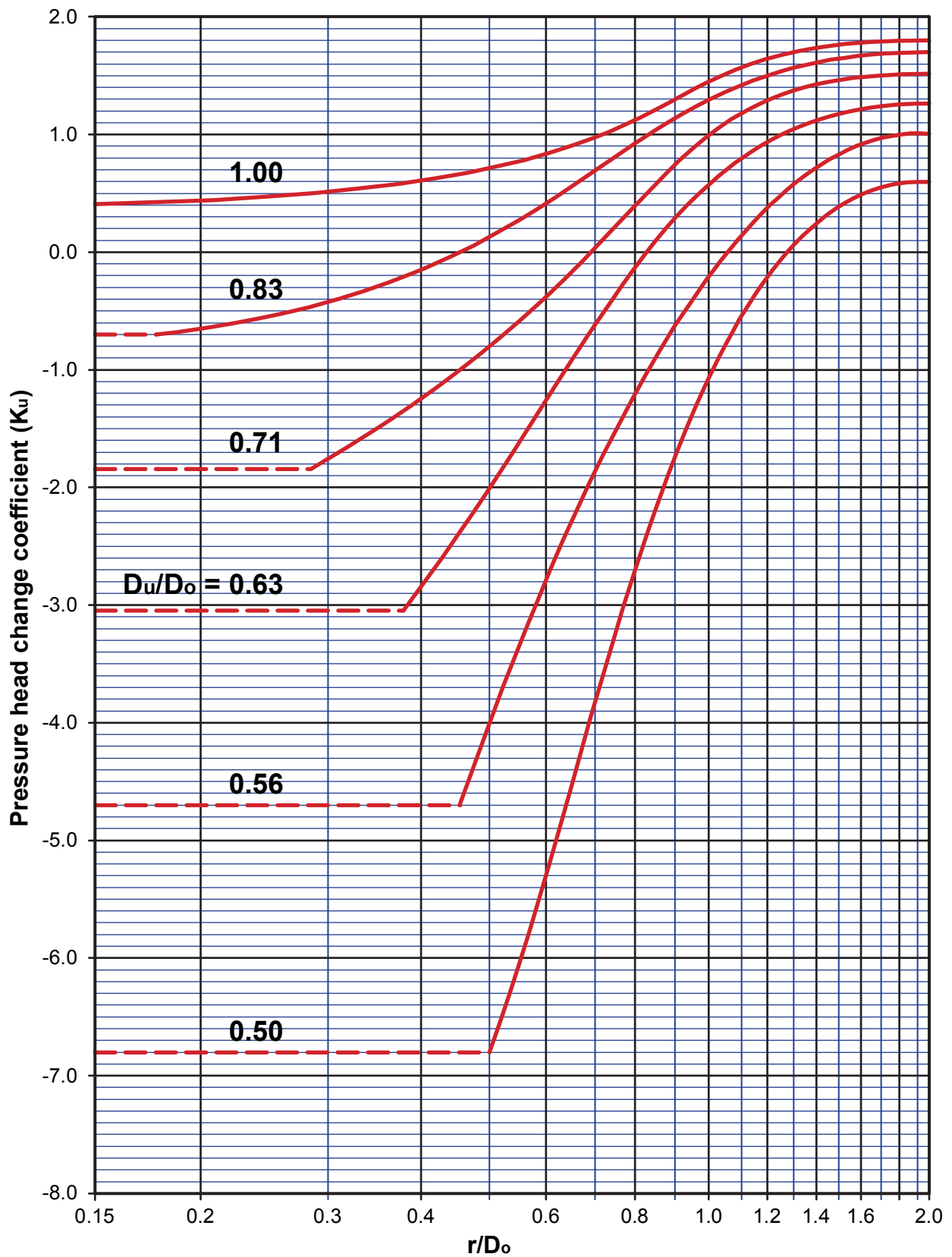
**Pressure head change coefficients for drop chambers with upstream
pipe entering above the outfall pipe and at angles of 0° to 90°
(Source: Cade and Thompson, 1982)**

Charts A2-37 and A2-38

The coefficients presented in these charts are used to determine the HGL for upstream and lateral pipes and the W.S.E for the structure with inflow from an upstream pipe and a lateral pipe. The coefficients depend on the diameter ratio D_L/D_o and ratio $(Q_u/Q_o).(D_o/D_u)$ and the manhole width to outfall pipe diameter ratio (B/D_o).

These charts are not applicable for:

- $(Q_u/Q_o).(D_o/D_u) > 1$; or

- $(D_L/D_o) < 0.6$, in which case **Chart A2-39** should be used.

These charts should not be used for $Q_u = 0$. In such cases charts A2-28, A2-29, A2-30, A2-31 and A2-35 should be used.

To use the charts:

1. Determine the outfall HGL elevation

2. Calculate the velocity head in the outfall pipe $(V_o^2/2g)$

3. Calculate the ratios Q_u/Q_o, D_u/D_o, and D_L/D_o. If D_L/D_o is less than 0.6, use Chart A2-39

4. Calculate the ratio B/Do and note if the outfall entrance is rounded

5. Calculate the ratio $(Q_u/Q_o).(D_o/D_u)$. If this is greater than 1, use Chart A2-39

For lateral pipe:

6. Determine $\overline{K}_L$ from the appropriate chart for the calculated values of D_L/D_o and B/D_o

7. For a rounded outfall pipe entrance or one formed by a pipe socket, reduce the value of $\overline{K}_L$ determined in step 6 by 0.2

8. Determine the factor M_L by entering the upper graph at the value of the ratio $(Q_u/Q_o).(D_o/D_u)$ and for the appropriate value of D_L/D_o

9. Calculate $K_L = M_L\ \overline{K}_L$

10. Calculate $h_L = K_L\ .V_o^2/2g$

11. HGL (lateral) = HGL (outfall) + h_L

For upstream in-line pipe:

12. Determine $\overline{K}_u$ from the appropriate chart for the calculated values of D_L/D_o

13. For a rounded entrance to the outfall pipe or one formed by a pipe socket, reduce the value of $\overline{K}_u$ determined in step 12 by 0.2

14. Determine the factor M_u from the upper graph

15. Calculate $K_u = M_u.\ \overline{K}_u$

16. Calculate $h_u = K_u\ .V_o^2/2g$

17. HGL (upstream) = HGL (outfall) + h_u

18. WSE = HGL (upstream)

19. Check that WSE meets the freeboard criteria

Axis labels (top chart):
$(Q_u/Q_o)(D_o/D_u)$

0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0

$D_L/D_o = 0.6$, 0.7, 0.8, 0.9, 1.0, 1.1, 1.2

Multiplier factor (M_L)
1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0.0

Lower chart:
B/D_o
2.00, 1.75, 1.50, 1.35, 1.20, 1.10, 1.00

$\overline{K_L}$
1.8 1.6 1.4 1.2 1.0 0.8 0.6

Deflectors angled 5° to 15°

Curved deflectors or deflectors angled 45°

D_L/D_o
0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3
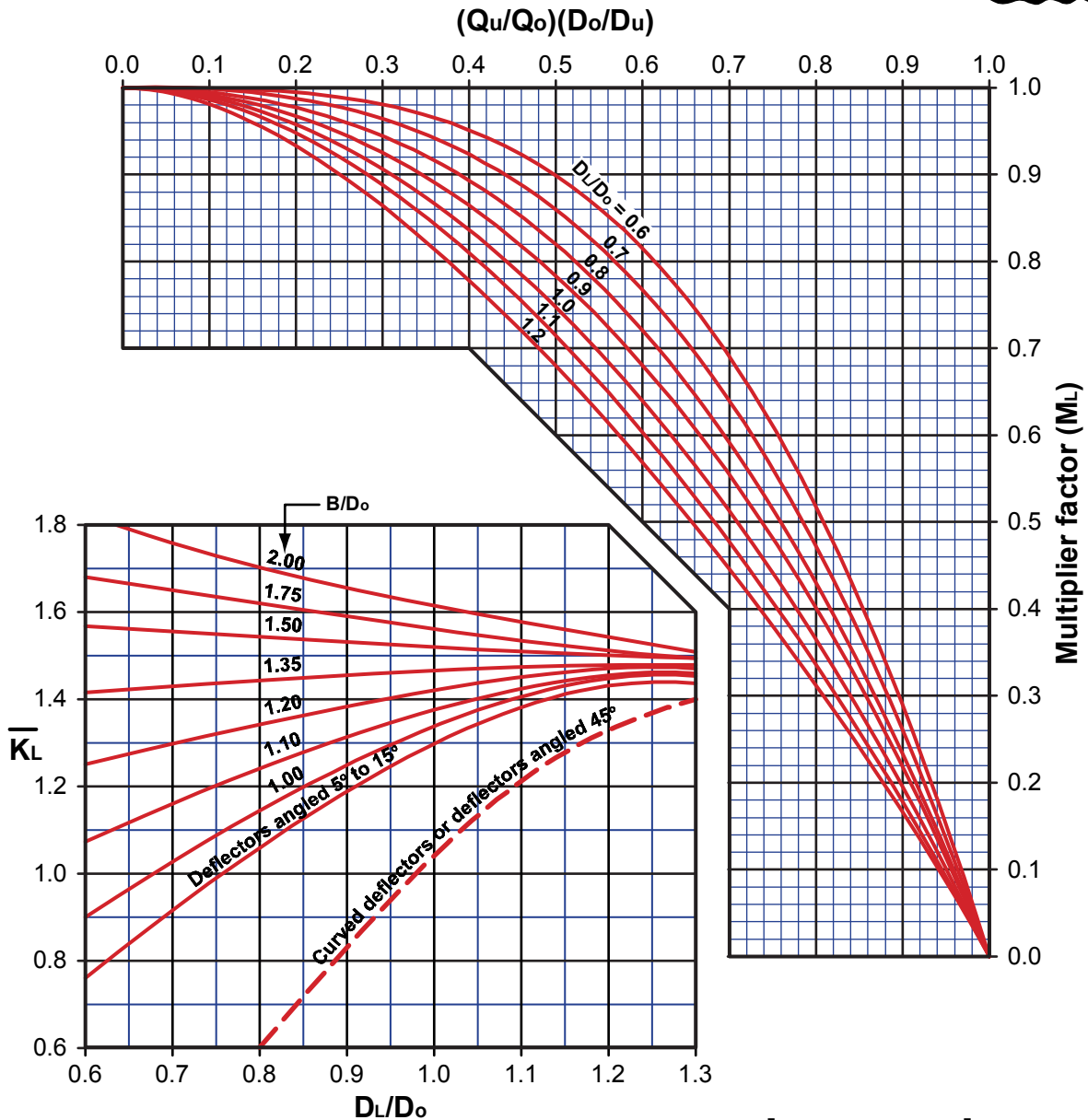
$$M_L = 1 - [(Q_u/Q_o)(D_o/D_u)]^2(D_o/D_L)$$
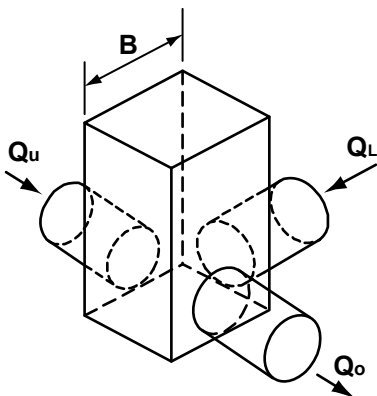
Notes:

$K_L = \overline{K_L} \times M_L$

Dashed curve in lower chart is used for curved or 45° angle deflectors without upstream in-line pipe (i.e. treat the upstream pipe as a 'lateral' pipe).

Chart may be used for round or rectangular pits.

For rounded entrance to outlet pipe, reduce chart values of $\overline{K_L}$ by 0.2 for combining flow.

For $(Q_u/Q_o)(D_o/D_u) > 1$, use Chart A2-39.

For $D_L/D_o < 0.6$, use Chart A2-39.

WSE = elevation of upstream HGL.

Do not use for $Q_u = 0.0$

B

Q_u **Q_L**

Q_o

Elevation sketch

D_L

D_u D_o
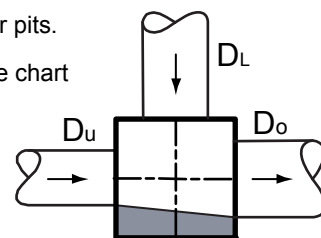
Plan view of pit with angled deflector

Pressure head change coefficients (K_L) for 90° lateral inflow pipe (Source: DOT, 1992)

Chart No. A2-37

(Qu/Qo)(Do/Du)

$(Q_u/Q_o)(D_o/D_u)$

0.0 0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0

1.0 0.9 0.8 0.7 0.6 0.5 0.4 0.3 0.2 0.1 0.0

Multiplier factor (Mu)

B/Do

2.00 1.75 1.50 1.35 1.20 1.10 1.00

$\overline{K_u}$

2.4 2.2 2.0 1.8 1.6 1.4

0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3

DL/Do
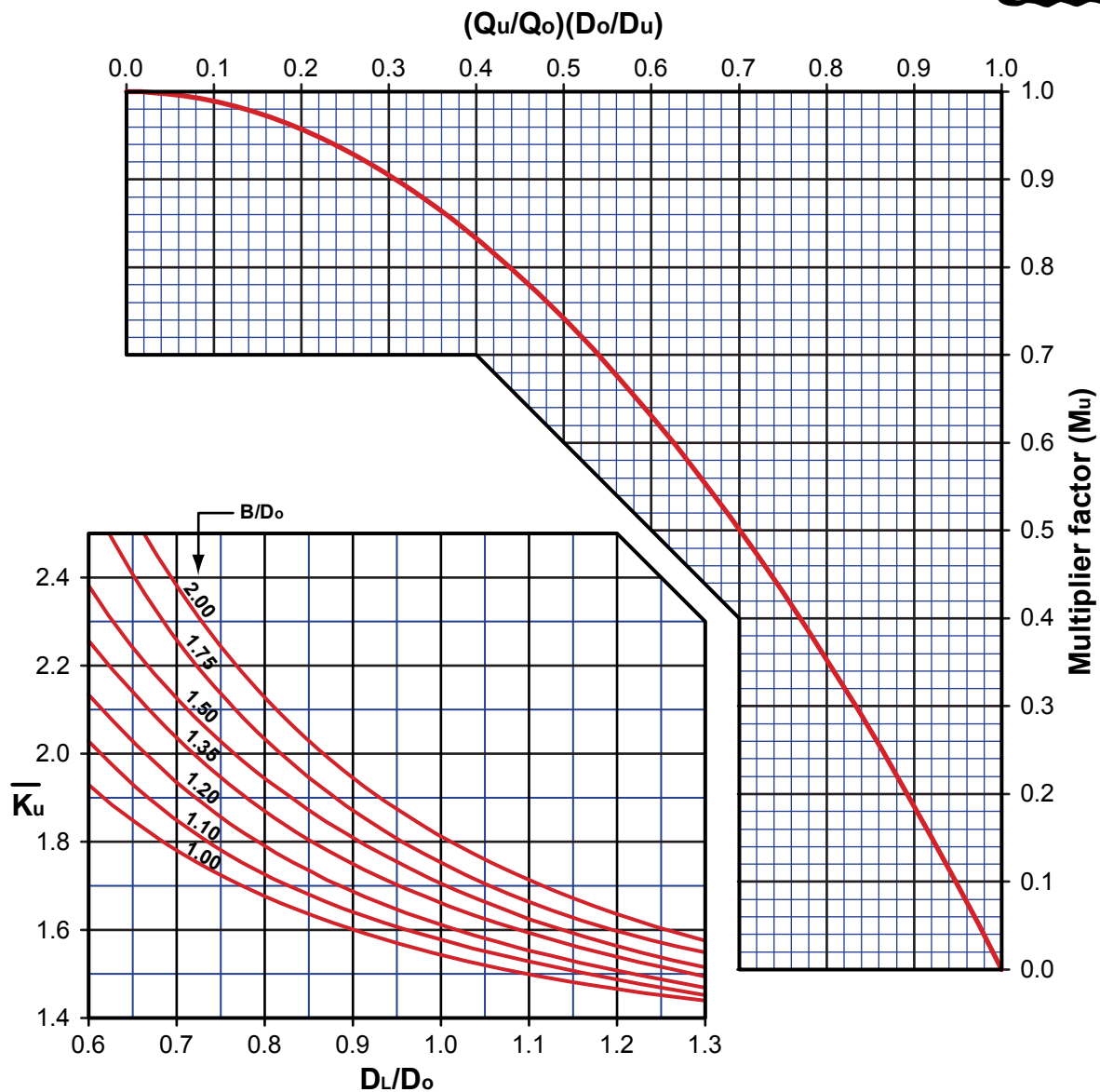
$$M_u = 1 - [(Q_u/Q_o)(D_o/D_u)]^2$$
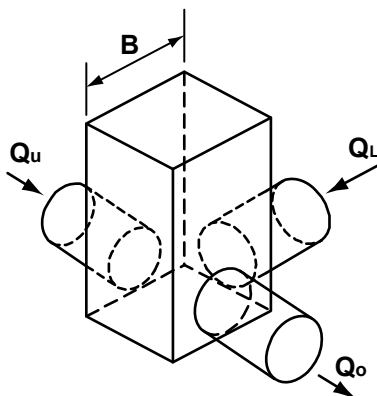
Notes:

$K_u = \overline{K_u} \times M_u$

For junction pits with deflectors at 0° to 15°, read K_u from curve for $B/D_o = 1.0$.

Chart may be used for round or rectangular pits.

For rounded entrance to outlet pipe, reduce chart values of $\overline{K_u}$ by 0.2 for combining flow.

For $(Q_u/Q_o)(D_o/D_u) > 1$, use Chart A2-39.

For $D_L/D_o < 0.6$, use Chart A2-39.

WSE = elevation of upstream HGL.

Do not use for $Q_u = 0.0$

B

Q_u Q_L

Q_o

Elevation sketch

D_L

D_u D_o

Plan view of pit with angled deflector

Pressure head change coefficients (Ku) for through flow pipeline at junction of 90° lateral inflow pipe (Source: DOT, 1992)

Chart No. A2-38
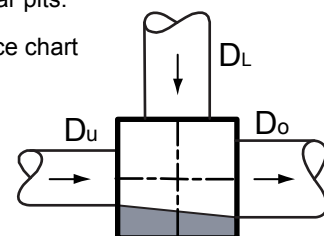
Chart A2-39

The coefficients presented in this chart are used to determine the HGL for the upstream and lateral pipes (the same in this case) and the WSE for the structure with inflow from an upstream pipe and a lateral pipe. The coefficients depend on the diameter ratio (D_u/D_o) and flow ratio (Q_u/Q_o).

This chart is used instead of charts A2-37 and A2-38 where:

- $(Q_u/Q_o).(D_o/D_u) > 1$; or
- $(D_L/D_o) < 0.6$

The values of K_u and K_L are not affected by the structure size or shape or by the inclusion of deflectors.

Note: Chart A2-39 should not be used for $Q_u/Q_o < 0.7$ if other solutions are possible.

To use the chart:

1. Determine the outfall pipe HGL elevation.
2. Calculate the velocity head in the outfall pipe ($V_o^2/2g$).
3. Calculate the ratios D_u/D_o, and Q_u/Q_o. Note that use of Chart A1-24 is advisable if the size and flow factors are within their range. Chart A2-39 should not be used for $Q_u/Q_o < 0.7$ if other solutions are possible.
4. Note whether the outfall entrance is to be rounded or formed by a pipe socket.
5. Determine K_u (or K_L) for the appropriate values of D_u/D_o and Q_u/Q_o
6. $K_w = K_u$ (or K_L) + 0.4
7. If $(Q_u/Q_o).(D_o/D_u)$ was found to be greater than 1 in an attempt to use Chart A1-24, K_u from step 5 will be negative.
8. For rounded entrance from the manhole to the outfall pipe use the reduced values from the chart.
9. Calculate $h_u = h_L = K_u .V_o^2/2g$

    $h_w = K_w .V_o^2/2g$
10. HGL (upstream and lateral) = HGL (outfall) + h_u (or h_L).
11. WSE = HGL (outfall) + h_w.
12. Check that WSE meets the freeboard criteria.
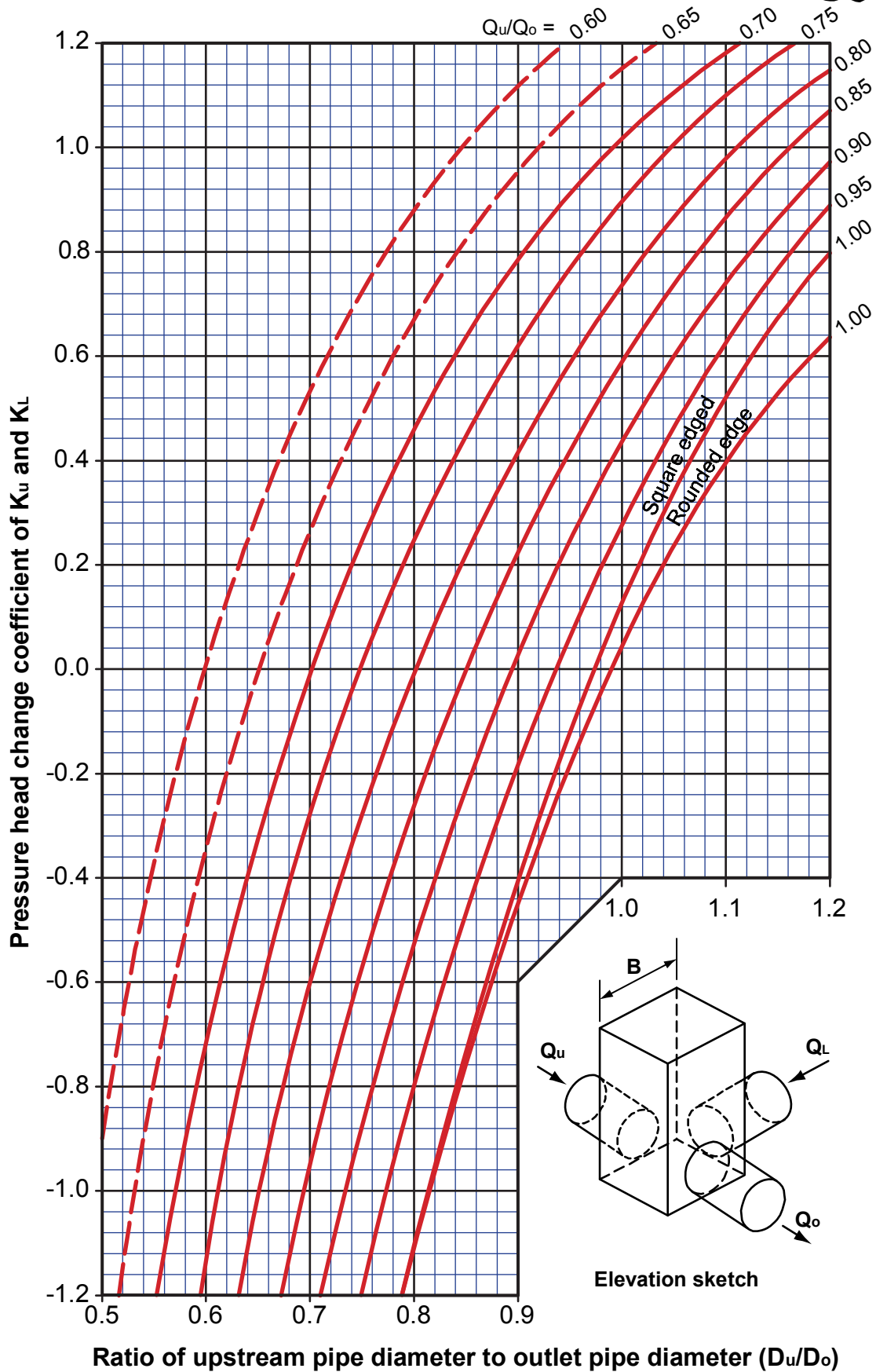
Pressure head change coefficients (K_u & K_L) for through flow pipeline at junction of 90° lateral inflow pipe for conditions outside the range of Charts A2-37 & 38 (Source: DOT, 1992)

Chart No. A2-39

Charts A2-40 to A2-44

It is important to note that these charts present 'energy loss' coefficients, not pressure head change coefficients.

The **energy loss** coefficients presented in these charts are used to determine the HGL for the upstream and lateral pipes with inflow only from the upstream and lateral pipes in this closed system (i.e no junction chamber exists).

The coefficients depend on the flow ratio (Q_L/Q_o) and the area ratio (A_L/A_o or D_L^2/D_o^2).

Note: In all cases, $A_u = A_o$.

To use the charts:

1. Determine the outfall pipe HGL elevation.
2. Calculate the velocity head in the outfall pipe ($V_o^2/2g$).
3. Calculate the ratios Q_L/Q_o and A_L/A_o (or D_L^2/D_o^2).
4. Determine K_u (or K_L) from the appropriate chart for the values calculated in step 3.
5. Calculate **energy loss** values: $H_u = K_u . V_o^2/2g$

$$H_L = K_L . V_o^2/2g$$

6. Calculate HGL (upstream) = HGL (outfall) + H_u + $V_o^2/2g$ - $V_u^2/2g$

HGL (lateral) = HGL (outfall) + H_L + $V_o^2/2g$ - $V_L^2/2g$

‘Energy loss’ coefficients for lateral pipe (K_L) for a non-chamber junction with branch angle of 15º (Source: Miller, 1978)

‘Energy loss’ coefficients for upstream pipe (K_u) for a non-chamber junction with branch angle of 15º (Source: Miller, 1978)

Chart No. A2-40

'Energy loss' coefficients for lateral pipe (K_L) for a non-chamber junction with branch angle of 30° (Source: Miller, 1978)

'Energy loss' coefficients for upstream pipe (K_u) for a non-chamber junction with branch angle of 30° (Source: Miller, 1978)

Chart No. A2-41

'Energy loss' coefficients for lateral pipe (K_L) for a non-chamber junction with branch angle of 45° (Source: Miller, 1978)

'Energy loss' coefficients for upstream pipe (K_u) for a non-chamber junction with branch angle of 45° (Source: Miller, 1978)

Chart No. A2-42

'Energy loss' coefficients for lateral pipe (K_L) for a non-chamber junction with branch angle of 60° (Source: Miller, 1978)

'Energy loss' coefficients for upstream pipe (K_u) for a non-chamber junction with branch angle of 60° (Source: Miller, 1978)

Chart No. A2-43

'Energy loss' coefficients for lateral pipe (K_L) for a non-chamber junction with branch angle of $90°$ (Source: Miller, 1978)

'Energy loss' coefficients for upstream pipe (K_u) for a non-chamber junction with branch angle of $90°$ (Source: Miller, 1978)

Chart No. A2-44

Appendix 3 Road flow capacity charts

Road flow capacity tables are presented in this appendix for four road widths.

The tables presented here provide the designer with road flow capacity information in chart form. The data for 6 metre width roads is common for all roads of greater width up to depth of flow at crown for the 6 metre road, but not for greater depths.

Table A3.1 – List of road flow capacity charts

2.5% crossfall, Manning's n = 0.015		
Chart No.	K&C type [1]	Width of road [2]
A3-1 and A3-2	M1	6.0
A3-3 and A3-4	M1	7.0
A3-5 and A3-6	M1	8.0
A3-7 and A3-8	B1	12.0

Notes (Table A3.1):

[1]   M1 = Mountable K&C (Type M1)

        B1 = Barrier K&C (Type B1)

[2]   Measured invert to invert.
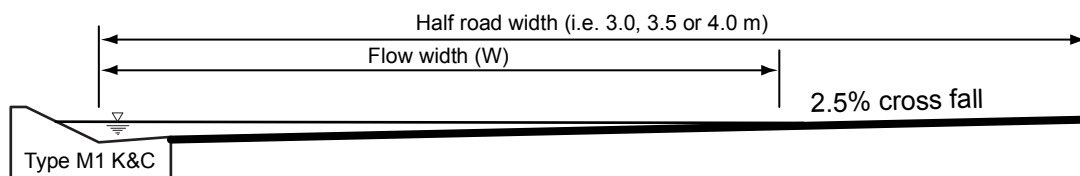
Figure A3.1 – Half road profile for the 6, 7 and 8 m road widths
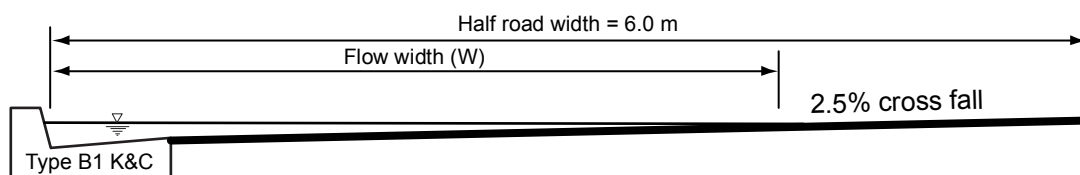
Figure A3.2 – Half road profile for the 12 m road width

Chart notes:

- The solid 'red' line indicated the depth*velocity limit, $d_g.V = 0.4$.

- No allowance has been made in the charts for future overlay of pavement surfacing.

- All design tables have been prepared using a *Flow Correction Factor* of 0.9 for the Izzard equation (refer to section 7.4.2 (d) of this manual).

ROAD FLOW CAPACITY TABLE

6.0 m road (invert to invert)
1/2 road flows only

Crossfall of 2.5% (1 in 40)
dg = 0.03 to 0.15 m

K & C Type M1
K & C Manning's n = 0.013
Pavement Manning's n = 0.015

Widths (W) and depths (dg) are measured from the invert of K & C

Flows below crown of road / Flows above crown of road

Grade %	dg = 0.03 A=0.006 W=0.47			dg = 0.04 A=0.014 W=0.87			dg = 0.05 A=0.025 W=1.27			dg = 0.06 A=0.041 W=1.67			dg = 0.07 A=0.061 W=2.07			dg = 0.08 A=0.085 W=2.47			dg = 0.09 A=0.114 W=2.87			dg = 0.10 A=0.145 W=3.00			dg = 0.11 A=0.177 W=3.00			dg = 0.12 A=0.210 W=3.00			dg = 0.13 A=0.242 W=3.00			dg = 0.14 A=0.275 W=3.00			dg = 0.15 A=0.308 W=3.00		
	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s
0.3	0.002	0.26	0.01	0.004	0.28	0.01	0.008	0.31	0.02	0.014	0.35	0.02	0.023	0.38	0.03	0.035	0.41	0.03	0.051	0.45	0.04	0.070	0.48	0.05	0.093	0.53	0.06	0.119	0.57	0.07	0.148	0.61	0.08	0.180	0.65	0.09	0.214	0.70	0.10
0.5	0.002	0.34	0.01	0.005	0.37	0.01	0.010	0.40	0.02	0.018	0.45	0.03	0.030	0.49	0.03	0.046	0.54	0.04	0.066	0.58	0.05	0.091	0.63	0.06	0.121	0.68	0.07	0.154	0.73	0.09	0.191	0.79	0.10	0.232	0.84	0.12	0.276	0.90	0.13
1.0	0.003	0.48	0.01	0.007	0.52	0.02	0.014	0.57	0.03	0.026	0.63	0.04	0.042	0.69	0.05	0.065	0.76	0.06	0.093	0.82	0.07	0.129	0.89	0.09	0.170	0.96	0.11	0.218	1.04	0.12	0.271	1.12	0.15	0.328	1.19	0.17	0.391	1.27	0.19
1.5	0.044	0.59	0.02	0.009	0.64	0.03	0.018	0.70	0.03	0.032	0.77	0.05	0.052	0.85	0.06	0.079	0.93	0.07	0.114	1.00	0.09	0.158	1.08	0.11	0.209	1.18	0.13	0.267	1.27	0.15	0.331	1.37	0.18	0.402	1.46	0.20	0.479	1.56	0.23
2.0	0.004	0.68	0.02	0.010	0.73	0.03	0.020	0.81	0.04	0.037	0.89	0.05	0.060	0.98	0.07	0.091	1.07	0.09	0.132	1.16	0.10	0.182	1.25	0.13	0.241	1.36	0.15	0.308	1.47	0.18	0.383	1.58	0.21	0.464	1.69	0.24	0.553	1.80	0.27
2.5	0.005	0.76	0.02	0.011	0.82	0.03	0.023	0.90	0.05	0.041	1.00	0.06	0.067	1.10	0.08	0.102	1.20	0.10	0.147	1.30	0.12	0.204	1.40	0.14	0.270	1.52	0.17	0.345	1.64	0.20	0.428	1.77	0.23	0.519	1.89	0.26	0.618	2.01	0.30
3.0	0.005	0.83	0.02	0.012	0.90	0.04	0.025	0.99	0.05	0.045	1.09	0.07	0.074	1.20	0.08	0.112	1.31	0.10	0.161	1.42	0.13	0.223	1.53	0.15	0.295	1.66	0.18	0.377	1.80	0.22	0.469	1.94	0.25	0.569	2.07	0.29	0.677	2.20	0.33
3.5	0.006	0.90	0.03	0.013	0.97	0.04	0.027	1.07	0.05	0.049	1.18	0.07	0.079	1.30	0.09	0.121	1.42	0.11	0.174	1.53	0.14	0.241	1.66	0.17	0.319	1.80	0.20	0.408	1.94	0.23	0.506	2.09	0.27	0.614	2.24	0.31	0.731	2.38	0.36
4.0	0.006	0.96	0.03	0.014	1.04	0.04	0.029	1.14	0.06	0.052	1.26	0.08	0.085	1.39	0.10	0.129	1.51	0.12	0.186	1.64	0.15	0.257	1.77	0.18	0.341	1.92	0.21	0.436	2.08	0.25	0.541	2.24	0.29	0.657	2.39	0.33	0.782	2.54	0.38
4.5	0.006	1.02	0.03	0.015	1.10	0.04	0.031	1.21	0.06	0.055	1.34	0.08	0.090	1.47	0.10	0.137	1.61	0.13	0.198	1.74	0.16	0.273	1.88	0.19	0.362	2.04	0.22	0.462	2.20	0.26	0.574	2.37	0.31	0.697	2.53	0.35	0.829	2.70	0.40
5.0	0.007	1.07	0.03	0.016	1.16	0.05	0.032	1.28	0.06	0.058	1.41	0.08	0.095	1.55	0.11	0.145	1.69	0.14	0.208	1.83	0.16	0.288	1.98	0.20	0.381	2.15	0.24	0.487	2.32	0.28	0.605	2.50	0.32	0.734	2.67	0.37	0.874	2.84	0.43
6.0	0.007	1.17	0.04	0.017	1.27	0.05	0.035	1.40	0.07	0.064	1.55	0.09	0.104	1.70	0.12	0.158	1.86	0.15	0.228	2.01	0.18	0.315	2.17	0.22	0.418	2.35	0.26	0.534	2.55	0.31	0.663	2.74	0.36	0.804	2.93	0.41	0.957	3.11	0.47
7.0	0.008	1.27	0.04	0.019	1.37	0.05	0.038	1.51	0.08	0.069	1.67	0.10	0.112	1.84	0.13	0.171	2.00	0.16	0.247	2.17	0.20	0.341	2.34	0.23	0.451	2.54	0.28	0.577	2.75	0.33	0.716	2.96	0.38	0.869	3.16	0.44	1.034	3.36	0.50
8.0	0.008	1.35	0.04	0.020	1.47	0.06	0.041	1.62	0.08	0.073	1.79	0.11	0.120	1.96	0.14	0.183	2.14	0.17	0.264	2.32	0.21	0.364	2.50	0.25	0.482	2.72	0.30	0.616	2.94	0.35	0.765	3.16	0.41	0.929	3.38	0.47	1.105	3.59	0.54
9.0	0.009	1.44	0.04	0.021	1.56	0.06	0.043	1.71	0.09	0.078	1.89	0.11	0.127	2.08	0.15	0.194	2.27	0.18	0.280	2.46	0.22	0.386	2.66	0.27	0.511	2.88	0.32	0.654	3.12	0.37	0.812	3.35	0.44	0.985	3.58	0.50	1.172	3.81	0.57
10.0	0.009	1.51	0.05	0.022	1.64	0.07	0.046	1.81	0.09	0.082	2.00	0.12	0.134	2.20	0.15	0.204	2.40	0.19	0.295	2.59	0.23	0.407	2.80	0.28	0.539	3.04	0.33	0.689	3.29	0.39	0.856	3.53	0.46	1.038	3.78	0.53	1.236	4.02	0.60
12.0	0.010	1.66	0.05	0.024	1.80	0.07	0.050	1.98	0.10	0.090	2.19	0.13	0.147	2.41	0.17	0.224	2.62	0.21	0.323	2.84	0.26	0.446	3.07	0.31	0.591	3.33	0.37	0.755	3.60	0.43	0.938	3.87	0.50	1.137	4.14	0.58	1.354	4.40	0.66
14.0	0.011	1.79	0.05	0.026	1.94	0.08	0.054	2.14	0.11	0.097	2.36	0.14	0.159	2.60	0.18	0.242	2.83	0.23	0.349	3.07	0.28	0.482	3.31	0.33	0.638	3.59	0.40	0.815	3.89	0.47	1.013	4.18	0.54	1.229	4.47	0.63	1.462	4.75	0.71
16.0	0.012	1.91	0.06	0.028	2.07	0.08	0.058	2.28	0.11	0.104	2.53	0.15	0.170	2.78	0.19	0.259	3.03	0.24	0.373	3.28	0.30	0.515	3.54	0.35	0.682	3.84	0.42	0.872	4.16	0.50	1.083	4.47	0.58	1.313	4.78	0.67	1.563	5.08	0.76

Chart No. A3-1

ROAD FLOW CAPACITY TABLE

6.0 m road (invert to invert)

1/2 road flows only

Widths and depths are measured from the invert of K & C

Crossfall of 2.5% (1 in 40)

dg = 0.16 to 0.28 m

K & C Type M1

K & C Manning's n = 0.013

Pavement Manning's n = 0.015

— Flows above crown of road —

Grade %	dg = 0.16 A=0.340 W=3.00			dg = 0.17 A=0.373 W=3.00			dg = 0.18 A=0.406 W=3.00			dg = 0.19 A=0.439 W=3.00			dg = 0.20 A=0.471 W=3.00			dg = 0.21 A=0.504 W=3.00			dg = 0.22 A=0.537 W=3.00			dg = 0.23 A=0.570 W=3.00			dg = 0.24 A=0.602 W=3.00			dg = 0.25 A=0.635 W=3.00			dg = 0.26 A=0.668 W=3.00			dg = 0.27 A=0.701 W=3.00			dg = 0.28 A=0.733 W=3.00		
	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s	Q m³/s	V m/s	dg.V m²/s
0.3	0.251	0.74	0.12	0.290	0.78	0.13	0.331	0.82	0.15	0.375	0.86	0.16	0.421	0.89	0.18	0.469	0.93	0.20	0.519	0.97	0.21	0.572	1.00	0.23	0.626	1.04	0.25	0.682	1.07	0.27	0.741	1.11	0.29	0.801	1.14	0.31	0.863	1.18	0.33
0.5	0.324	0.95	0.15	0.374	1.00	0.17	0.428	1.05	0.19	0.484	1.10	0.21	0.544	1.15	0.23	0.606	1.20	0.25	0.671	1.25	0.27	0.738	1.30	0.30	0.808	1.34	0.32	0.881	1.39	0.35	0.956	1.43	0.37	1.034	1.48	0.40	1.114	1.52	0.43
1.0	0.458	1.35	0.22	0.529	1.42	0.24	0.605	1.49	0.27	0.685	1.56	0.30	0.769	1.63	0.33	0.857	1.70	0.36	0.948	1.77	0.39	1.044	1.83	0.42	1.143	1.90	0.46	1.246	1.96	0.49	1.352	2.02	0.53	1.462	2.09	0.56	1.575	2.15	0.60
1.5	0.561	1.65	0.26	0.648	1.74	0.30	0.741	1.83	0.33	0.839	1.91	0.36	0.941	2.00	0.40	1.049	2.08	0.44	1.161	2.16	0.48	1.278	2.24	0.52	1.400	2.32	0.56	1.526	2.40	0.60	1.656	2.48	0.64	1.790	2.56	0.69	1.929	2.63	0.74
2.0	0.647	1.90	0.30	0.748	2.01	0.34	0.856	2.11	0.38	0.968	2.21	0.42	1.087	2.31	0.46	1.211	2.40	0.50	1.341	2.50	0.55	1.476	2.59	0.60	1.616	2.68	0.64	1.762	2.77	0.69	1.912	2.86	0.74	2.067	2.95	0.80	2.228	3.04	0.85
2.5	0.724	2.13	0.34	0.837	2.24	0.38	0.957	2.36	0.42	1.083	2.47	0.47	1.215	2.58	0.52	1.354	2.69	0.56	1.499	2.79	0.61	1.650	2.90	0.67	1.807	3.00	0.72	1.970	3.10	0.78	2.138	3.20	0.83	2.311	3.30	0.89	2.491	3.40	0.95
3.0	0.793	2.33	0.37	0.917	2.46	0.42	1.048	2.58	0.46	1.186	2.70	0.51	1.331	2.82	0.56	1.484	2.94	0.62	1.642	3.06	0.67	1.808	3.17	0.73	1.980	3.29	0.79	2.158	3.40	0.85	2.342	3.51	0.91	2.532	3.61	0.98	2.728	3.72	1.04
3.5	0.857	2.52	0.40	0.990	2.65	0.45	1.132	2.79	0.50	1.281	2.92	0.56	1.438	3.05	0.61	1.602	3.18	0.67	1.774	3.30	0.73	1.953	3.43	0.79	2.138	3.55	0.85	2.330	3.67	0.92	2.529	3.79	0.98	2.735	3.90	1.05	2.947	4.02	1.13
4.0	0.916	2.69	0.43	1.059	2.84	0.48	1.210	2.98	0.54	1.370	3.12	0.59	1.537	3.26	0.65	1.713	3.40	0.71	1.896	3.53	0.78	2.087	3.66	0.84	2.286	3.79	0.91	2.491	3.92	0.98	2.704	4.05	1.05	2.924	4.17	1.13	3.150	4.30	1.20
4.5	0.971	2.85	0.46	1.123	3.01	0.51	1.283	3.16	0.57	1.453	3.31	0.63	1.631	3.46	0.69	1.817	3.60	0.76	2.012	3.75	0.82	2.214	3.89	0.89	2.424	4.03	0.97	2.642	4.16	1.04	2.868	4.29	1.12	3.101	4.43	1.20	3.342	4.56	1.28
5.0	1.024	3.01	0.48	1.183	3.17	0.54	1.353	3.33	0.60	1.531	3.49	0.66	1.719	3.65	0.73	1.915	3.80	0.80	2.120	3.95	0.87	2.334	4.10	0.94	2.556	4.24	1.02	2.785	4.39	1.10	3.023	4.53	1.18	3.269	4.67	1.26	3.522	4.80	1.34
6.0	1.121	3.30	0.53	1.296	3.47	0.59	1.482	3.65	0.66	1.677	3.82	0.73	1.883	4.00	0.80	2.098	4.16	0.87	2.323	4.33	0.95	2.557	4.49	1.03	2.799	4.65	1.12	3.051	4.80	1.20	3.312	4.96	1.29	3.581	5.11	1.38	3.858	5.26	1.47
7.0	1.211	3.56	0.57	1.400	3.75	0.64	1.601	3.94	0.71	1.812	4.13	0.78	2.034	4.32	0.86	2.266	4.50	0.94	2.509	4.67	1.03	2.761	4.85	1.12	3.024	5.02	1.20	3.296	5.19	1.30	3.577	5.36	1.39	3.868	5.52	1.49	4.168	5.68	1.59
8.0	1.295	3.80	0.61	1.497	4.01	0.68	1.711	4.22	0.76	1.937	4.42	0.84	2.174	4.61	0.92	2.423	4.81	1.01	2.682	5.00	1.10	2.952	5.18	1.19	3.233	5.37	1.29	3.523	5.55	1.39	3.824	5.73	1.49	4.135	5.90	1.59	4.455	6.08	1.70
9.0	1.373	4.04	0.65	1.588	4.26	0.72	1.815	4.47	0.80	2.054	4.68	0.89	2.306	4.89	0.98	2.570	5.10	1.07	2.845	5.30	1.17	3.131	5.50	1.26	3.429	5.69	1.37	3.737	5.88	1.47	4.056	6.07	1.58	4.386	6.26	1.69	4.726	6.44	1.80
10.0	1.448	4.25	0.68	1.674	4.49	0.76	1.913	4.71	0.85	2.166	4.94	0.94	2.431	5.16	1.03	2.709	5.37	1.13	2.999	5.59	1.23	3.301	5.79	1.33	3.614	6.00	1.44	3.939	6.20	1.55	4.275	6.40	1.66	4.623	6.60	1.78	4.981	6.79	1.90
12.0	1.586	4.66	0.75	1.833	4.91	0.84	2.096	5.16	0.93	2.372	5.41	1.03	2.663	5.65	1.13	2.967	5.89	1.24	3.285	6.12	1.35	3.616	6.35	1.46	3.959	6.57	1.58	4.315	6.79	1.70	4.684	7.01	1.82	5.064	7.23	1.95	5.457	7.44	2.08
14.0	1.713	5.03	0.81	1.980	5.31	0.90	2.264	5.58	1.00	2.562	5.84	1.11	2.876	6.10	1.22	3.205	6.36	1.34	3.548	6.61	1.45	3.905	6.86	1.58	4.276	7.10	1.70	4.661	7.34	1.83	5.059	7.58	1.97	5.470	7.81	2.11	5.894	8.04	2.25
16.0	1.831	5.38	0.86	2.117	5.67	0.96	2.420	5.96	1.07	2.739	6.25	1.19	3.075	6.52	1.30	3.426	6.80	1.43	3.793	7.07	1.55	4.175	7.33	1.69	4.571	7.59	1.82	4.983	7.85	1.96	5.408	8.10	2.11	5.848	8.35	2.25	6.301	8.59	2.41

Qudm

Chart No. A3-2

ROAD FLOW CAPACITY TABLE

Qudm

K & C Type M1

7.0 m road (invert to invert) Crossfall of 2.5% (1 in 40)

1/2 road flows only $d_g = 0.03$ to 0.15

K & C Manning's n = 0.013

Pavement Manning's n = 0.015

Widths (W) and depths (d_g) are measured from the invert of K & C

— Flows below crown of road —   — Flows above crown of road —

Grade %	$d_g=0.03$ A=0.006 W=0.47 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.04$ A=0.014 W=0.87 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.05$ A=0.025 W=1.27 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.06$ A=0.041 W=1.67 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.07$ A=0.061 W=2.07 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.08$ A=0.085 W=2.47 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.09$ A=0.114 W=2.87 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.10$ A=0.146 W=3.27 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.11$ A=0.183 W=3.50 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.12$ A=0.220 W=3.50 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.13$ A=0.257 W=3.50 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.14$ A=0.295 W=3.50 Q m³/s	V m/s	d_g.V m²/s	$d_g=0.15$ A=0.333 W=3.50 Q m³/s	V m/s	d_g.V m²/s
0.3	0.002	0.26	0.01	0.004	0.28	0.01	0.008	0.31	0.02	0.014	0.35	0.02	0.023	0.38	0.03	0.035	0.41	0.03	0.051	0.45	0.04	0.071	0.48	0.05	0.094	0.52	0.06	0.122	0.55	0.07	0.153	0.59	0.08	0.188	0.64	0.09	0.225	0.68	0.10
0.5	0.002	0.34	0.01	0.005	0.37	0.01	0.010	0.40	0.02	0.018	0.45	0.03	0.030	0.49	0.04	0.046	0.54	0.05	0.066	0.58	0.05	0.091	0.62	0.06	0.122	0.67	0.07	0.157	0.72	0.09	0.198	0.77	0.10	0.242	0.82	0.11	0.291	0.87	0.13
1.0	0.003	0.48	0.01	0.007	0.52	0.02	0.014	0.57	0.03	0.026	0.63	0.04	0.042	0.69	0.05	0.065	0.76	0.06	0.093	0.82	0.07	0.129	0.88	0.09	0.172	0.94	0.10	0.223	1.01	0.12	0.280	1.09	0.14	0.343	1.16	0.16	0.412	1.24	0.19
1.5	0.044	0.59	0.02	0.009	0.64	0.03	0.018	0.70	0.03	0.032	0.77	0.05	0.052	0.85	0.06	0.079	0.93	0.07	0.114	1.00	0.09	0.158	1.08	0.11	0.211	1.15	0.13	0.273	1.24	0.15	.0343	1.33	0.17	0.420	1.42	0.20	0.504	1.51	0.23
2.0	0.004	0.68	0.02	0.010	0.73	0.03	0.020	0.81	0.04	0.037	0.89	0.05	0.060	0.98	0.07	0.091	1.07	0.09	0.132	1.16	0.10	0.182	1.25	0.12	0.243	1.33	0.15	0.315	1.43	0.17	0.396	1.54	0.20	0.485	1.64	0.23	0.582	1.75	0.26
2.5	0.005	0.76	0.02	0.011	0.82	0.03	0.023	0.90	0.05	0.041	1.00	0.06	0.067	1.10	0.08	0.102	1.20	0.10	0.147	1.30	0.12	0.204	1.39	0.14	0.272	1.49	0.16	0.352	1.60	0.19	0.442	1.72	0.22	0.542	1.84	0.26	0.651	1.96	0.29
3.0	0.005	0.83	0.02	0.012	0.90	0.04	0.025	0.99	0.05	0.045	1.09	0.06	0.074	1.20	0.08	0.112	1.31	0.10	0.161	1.42	0.13	0.223	1.52	0.15	0.298	1.63	0.18	0.386	1.75	0.21	0.484	1.88	0.24	.0594	2.01	0.28	0.713	2.14	0.32
3.5	0.006	0.90	0.03	0.013	0.97	0.04	0.027	1.07	0.05	0.049	1.18	0.07	0.079	1.30	0.09	0.121	1.42	0.11	0.174	1.53	0.14	0.241	1.65	0.16	0.322	1.76	0.19	0.417	1.89	0.23	0.523	2.03	0.26	0.641	2.17	0.30	0.770	2.31	0.35
4.0	0.006	0.96	0.03	0.014	1.04	0.04	0.029	1.14	0.06	0.052	1.26	0.08	0.085	1.39	0.10	0.129	1.51	0.12	0.186	1.64	0.15	0.258	1.76	0.18	0.344	1.88	0.21	0.445	2.02	0.24	0.559	2.17	0.28	0.686	2.32	0.33	0.823	2.47	0.37
4.5	0.006	1.02	0.03	0.015	1.10	0.04	0.031	1.21	0.06	0.055	1.34	0.08	0.090	1.47	0.10	0.137	1.61	0.13	0.198	1.74	0.16	0.273	1.87	0.19	0.365	2.00	0.22	0.472	2.15	0.26	0.593	2.30	0.30	0.727	2.46	0.34	0.873	2.62	0.39
5.0	0.007	1.07	0.03	0.016	1.16	0.05	0.032	1.28	0.06	0.058	1.41	0.08	0.095	1.55	0.11	0.145	1.69	0.14	0.208	1.83	0.16	0.288	1.97	0.20	0.385	2.11	0.23	0.498	2.26	0.27	0.625	2.43	0.32	0.767	2.60	0.36	0.920	2.77	0.41
6.0	0.007	1.17	0.04	0.017	1.27	0.05	0.035	1.40	0.07	0.064	1.55	0.09	0.104	1.70	0.12	0.158	1.86	0.15	0.228	2.01	0.18	0.316	2.16	0.22	0.422	2.31	0.25	0.545	2.48	0.30	0.685	2.66	0.35	0.840	2.85	0.40	1.008	3.03	0.45
7.0	0.008	1.27	0.04	0.019	1.37	0.05	0.038	1.51	0.08	0.069	1.67	0.10	0.112	1.84	0.13	0.171	2.00	0.16	0.247	2.17	0.20	0.341	2.33	0.23	0.455	2.49	0.27	0.589	2.68	0.32	0.740	2.87	0.37	0.907	3.07	0.43	1.089	3.27	0.49
8.0	0.008	1.35	0.04	0.020	1.47	0.06	0.041	1.62	0.08	0.073	1.79	0.11	0.120	1.96	0.14	0.183	2.14	0.17	0.264	2.32	0.21	0.364	2.49	0.25	0.487	2.66	0.29	0.630	2.86	0.34	0.791	3.07	0.40	0.970	3.29	0.46	1.164	3.50	0.52
9.0	0.009	1.44	0.04	0.021	1.56	0.06	0.043	1.71	0.09	0.078	1.89	0.11	0.127	2.08	0.15	0.194	2.27	0.18	0.280	2.46	0.22	0.387	2.64	0.26	0.516	2.83	0.31	0.668	3.04	0.36	0.839	3.26	0.42	1.028	3.48	0.49	1.235	3.71	0.56
10.0	0.009	1.51	0.05	0.022	1.64	0.07	0.046	1.81	0.09	0.082	2.00	0.12	0.134	2.20	0.15	0.204	2.40	0.19	0.295	2.59	0.23	0.408	2.78	0.28	0.544	2.98	0.33	0.704	3.20	0.38	0.885	3.44	0.45	1.084	3.67	0.51	1.302	3.91	0.59
12.0	0.010	1.66	0.05	0.024	1.80	0.07	0.050	1.98	0.10	0.090	2.19	0.13	0.147	2.41	0.17	0.224	2.62	0.21	0.323	2.84	0.26	0.446	3.05	0.30	0.596	3.26	0.36	0.771	3.51	0.42	0.969	3.76	0.49	1.188	4.02	0.56	1.426	4.28	0.64
14.0	0.011	1.79	0.05	0.026	1.94	0.08	0.054	2.14	0.11	0.097	2.36	0.14	0.159	2.60	0.18	0.242	2.83	0.23	0.349	3.07	0.28	0.482	3.29	0.33	0.644	3.52	0.39	0.833	3.79	0.45	1.047	4.06	0.53	1.283	4.35	0.61	1.540	4.63	0.69
16.0	0.012	1.91	0.06	0.028	2.07	0.08	0.058	2.28	0.11	0.104	2.53	0.15	0.170	2.78	0.19	0.259	3.03	0.24	0.373	3.28	0.30	0.515	3.52	0.35	0.689	3.77	0.41	0.891	4.05	0.49	1.119	4.35	0.56	1.371	4.65	0.65	1.647	4.95	0.74

Chart No. A3-3

ROAD FLOW CAPACITY TABLE

Qudm

7.0 m road (invert to invert) Crossfall of 2.5% (1 in 40) **K & C Type M1**

1/2 road flows only d_g = 0.16 to 0.28 m K & C Manning's n = 0.013

Widths (W) and depths (d_g) are measured from the invert of K & C Pavement Manning's n = 0.015

Flows above crown of road

Grade %	d_g=0.16 A=0.371 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.17 A=0.408 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.18 A=0.446 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.19 A=0.484 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.20 A=0.522 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.21 A=0.559 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.22 A=0.597 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.23 A=0.635 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.24 A=0.673 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.25 A=0.710 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.26 A=0.748 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.27 A=0.786 W=3.50 Q m³/s	V m/s	d_g.V m²/s	d_g=0.28 A=0.824 W=3.50 Q m³/s	V m/s	d_g.V m²/s
0.3	0.266	0.72	0.11	0.310	0.76	0.13	0.356	0.80	0.14	0.405	0.84	0.16	0.457	0.88	0.18	0.511	0.91	0.19	0.568	0.95	0.21	0.627	0.99	0.23	0.688	1.02	0.25	0.752	1.06	0.26	0.819	1.09	0.28	0.887	1.13	0.30	0.958	1.16	0.33
0.5	0.344	0.93	0.15	0.400	0.98	0.17	0.460	1.03	0.19	0.523	1.08	0.21	0.590	1.13	0.23	0.660	1.18	0.25	0.733	1.23	0.27	0.809	1.27	0.29	0.889	1.32	0.32	0.971	1.37	0.34	1.057	1.41	0.37	1.145	1.46	0.39	1.236	1.50	0.42
1.0	0.486	1.31	0.21	0.566	1.38	0.24	0.650	1.46	0.26	0.740	1.53	0.29	0.834	1.60	0.32	0.933	1.67	0.35	1.037	1.74	0.38	1.145	1.80	0.41	1.257	1.87	0.45	1.374	1.93	0.48	1.494	2.00	0.52	1.619	2.06	0.56	1.748	2.12	0.59
1.5	0.595	1.61	0.26	0.693	1.70	0.29	0.796	1.78	0.32	0.906	1.87	0.36	1.022	1.96	0.39	1.143	2.04	0.43	1.270	2.13	0.47	1.402	2.21	0.51	1.540	2.29	0.55	1.682	2.37	0.59	1.830	2.45	0.64	1.983	2.52	0.68	2.141	2.60	0.73
2.0	0.687	1.85	0.30	0.800	1.96	0.33	0.919	2.06	0.37	1.046	2.16	0.41	1.180	2.26	0.45	1.320	2.36	0.50	1.466	2.46	0.54	1.619	2.55	0.59	1.778	2.64	0.63	1.943	2.73	0.68	2.113	2.82	0.73	2.290	2.91	0.79	2.473	3.00	0.84
2.5	0.768	2.07	0.33	0.894	2.19	0.37	1.028	2.30	0.41	1.170	2.42	0.46	1.319	2.53	0.51	1.475	2.64	0.55	1.639	2.74	0.60	1.810	2.85	0.66	1.987	2.95	0.71	2.172	3.06	0.76	2.363	3.16	0.82	2.560	3.26	0.88	2.764	3.36	0.94
3.0	0.842	2.27	0.36	0.979	2.40	0.41	1.126	2.52	0.45	1.281	2.65	0.50	1.445	2.77	0.55	1.616	2.89	0.61	1.796	3.01	0.66	1.983	3.12	0.72	2.177	3.24	0.78	2.379	3.35	0.84	2.588	3.46	0.90	2.805	3.57	0.96	3.028	3.68	1.03
3.5	0.909	2.45	0.39	1.058	2.59	0.44	1.216	2.73	0.49	1.384	2.86	0.54	1.560	2.99	0.60	1.746	3.12	0.66	1.939	3.25	0.71	2.141	3.37	0.78	2.352	3.50	0.84	2.570	3.62	0.90	2.796	3.74	0.97	3.030	3.86	1.04	3.271	3.97	1.11
4.0	0.972	2.62	0.42	1.131	2.77	0.47	1.300	2.91	0.52	1.479	3.06	0.58	1.668	3.20	0.64	1.866	3.34	0.70	2.073	3.47	0.76	2.289	3.61	0.83	2.514	3.74	0.90	2.747	3.87	0.97	2.989	4.00	1.04	3.239	4.12	1.11	3.497	4.25	1.19
4.5	1.031	2.78	0.45	1.200	2.94	0.50	1.379	3.09	0.56	1.569	3.24	0.62	1.769	3.39	0.68	1.979	3.54	0.74	2.199	3.68	0.81	2.428	3.82	0.88	2.666	3.96	0.95	2.914	4.10	1.03	3.170	4.24	1.10	3.435	4.37	1.18	3.709	4.50	1.26
5.0	1.087	2.93	0.47	1.265	3.10	0.53	1.454	3.26	0.59	1.654	3.42	0.65	1.865	3.58	0.72	2.086	3.73	0.78	2.318	3.88	0.85	2.559	4.03	0.93	2.811	4.18	1.00	3.072	4.32	1.08	3.342	4.47	1.16	3.621	4.61	1.24	3.910	4.75	1.33
6.0	1.190	3.21	0.51	1.385	3.39	0.58	1.593	3.57	0.64	1.812	3.74	0.71	2.043	3.92	0.78	2.286	4.09	0.86	2.539	4.25	0.94	2.804	4.42	1.02	3.079	4.58	1.10	3.365	4.74	1.18	3.661	4.89	1.27	3.967	5.05	1.36	4.283	5.20	1.46
7.0	1.286	3.47	0.56	1.496	3.66	0.62	1.720	3.86	0.69	1.957	4.04	0.77	2.207	4.23	0.85	2.469	4.41	0.93	2.743	4.59	1.01	3.028	4.77	1.10	3.326	4.94	1.19	3.634	5.12	1.28	3.954	5.29	1.37	4.285	5.45	1.47	4.626	5.62	1.57
8.0	1.374	3.71	0.59	1.600	3.92	0.67	1.839	4.12	0.74	2.092	4.32	0.82	2.359	4.52	0.90	2.639	4.72	0.99	2.932	4.91	1.08	3.238	5.10	1.17	3.555	5.29	1.27	3.885	5.47	1.37	4.227	5.65	1.47	4.580	5.83	1.57	4.945	6.00	1.68
9.0	1.458	3.93	0.63	1.697	4.15	0.71	1.950	4.37	0.79	2.219	4.59	0.87	2.502	4.80	0.96	2.799	5.00	1.05	3.110	5.21	1.15	3.434	5.41	1.24	3.771	5.61	1.35	4.121	5.80	1.45	4.483	5.99	1.56	4.858	6.18	1.67	5.245	6.37	1.78
10.0	1.537	4.15	0.66	1.788	4.38	0.74	2.056	4.61	0.83	2.339	4.83	0.92	2.638	5.06	1.01	2.951	5.27	1.11	3.278	5.49	1.21	3.620	5.70	1.31	3.975	5.91	1.42	4.344	6.11	1.53	4.726	6.32	1.64	5.121	6.52	1.76	5.529	6.71	1.88
12.0	1.683	4.54	0.73	1.959	4.80	0.82	2.252	5.05	0.91	2.562	5.30	1.01	2.889	5.54	1.11	3.232	5.78	1.21	3.591	6.01	1.32	3.965	6.25	1.44	4.354	6.47	1.55	4.758	6.70	1.67	5.177	6.92	1.80	5.610	7.14	1.93	6.057	7.35	2.06
14.0	1.818	4.91	0.78	2.116	5.18	0.88	2.433	5.45	0.98	2.768	5.72	1.09	3.121	5.98	1.20	3.491	6.24	1.31	3.879	6.50	1.43	4.283	6.75	1.55	4.703	6.99	1.68	5.140	7.24	1.81	5.592	7.47	1.94	6.059	7.71	2.08	6.542	7.94	2.22
16.0	1.944	5.24	0.84	2.262	5.54	0.94	2.601	5.83	1.05	2.959	6.11	1.16	3.336	6.40	1.28	3.732	6.67	1.40	4.147	6.94	1.53	4.579	7.21	1.66	5.028	7.48	1.79	5.495	7.73	1.93	5.978	7.99	2.08	6.478	8.24	2.23	6.994	8.49	2.38

Chart No. A3-4

ROAD FLOW CAPACITY TABLE

8.0 m road (invert to invert) **Crossfall of 2.5% (1 in 40)** **K & C Type M1**

1/2 road flows only **dg = 0.03 to 0.15** **K & C Manning's n = 0.013**

Widths (W) and depths (dg) are measured from the invert of K & C **Pavement Manning's n = 0.015**

Flows below crown of road → Flows above crown of road

Grade %	dg=0.03 A=0.006 W=0.47 Q m³/s	V m/s	dg.V m²/s	dg=0.04 A=0.014 W=0.87 Q m³/s	V m/s	dg.V m²/s	dg=0.05 A=0.025 W=1.27 Q m³/s	V m/s	dg.V m²/s	dg=0.06 A=0.041 W=1.67 Q m³/s	V m/s	dg.V m²/s	dg=0.07 A=0.061 W=2.07 Q m³/s	V m/s	dg.V m²/s	dg=0.08 A=0.085 W=2.47 Q m³/s	V m/s	dg.V m²/s	dg=0.09 A=0.114 W=2.87 Q m³/s	V m/s	dg.V m²/s	dg=0.10 A=0.146 W=3.27 Q m³/s	V m/s	dg.V m²/s	dg=0.11 A=0.183 W=3.67 Q m³/s	V m/s	dg.V m²/s	dg=0.12 A=0.224 W=4.00 Q m³/s	V m/s	dg.V m²/s	dg=0.13 A=0.267 W=4.00 Q m³/s	V m/s	dg.V m²/s	dg=0.14 A=0.309 W=4.00 Q m³/s	V m/s	dg.V m²/s	dg=0.15 A=0.352 W=4.00 Q m³/s	V m/s	dg.V m²/s
0.3	0.002	0.26	0.01	0.004	0.28	0.01	0.008	0.31	0.02	0.014	0.35	0.02	0.023	0.38	0.03	0.035	0.41	0.03	0.051	0.45	0.04	0.071	0.48	0.05	0.094	0.51	0.06	0.123	0.55	0.07	0.155	0.58	0.08	0.192	0.62	0.09	0.233	0.66	0.10
0.5	0.002	0.34	0.01	0.005	0.37	0.01	0.010	0.40	0.02	0.018	0.45	0.03	0.030	0.49	0.03	0.046	0.54	0.04	0.066	0.58	0.05	0.091	0.62	0.06	0.122	0.66	0.07	0.158	0.71	0.08	0.200	0.75	0.10	0.248	0.80	0.11	0.300	0.85	0.13
1.0	0.003	0.48	0.01	0.007	0.52	0.02	0.014	0.57	0.03	0.026	0.63	0.04	0.042	0.69	0.05	0.065	0.76	0.06	0.093	0.82	0.07	0.129	0.88	0.09	0.172	0.94	0.10	0.224	1.00	0.12	0.284	1.06	0.14	0.351	1.13	0.16	0.425	1.21	0.18
1.5	0.044	0.59	0.02	0.009	0.64	0.03	0.018	0.70	0.04	0.032	0.77	0.05	0.052	0.85	0.06	0.079	0.93	0.07	0.114	1.00	0.09	0.158	1.08	0.11	0.211	1.15	0.13	0.274	1.22	0.15	0.347	1.30	0.17	0.429	1.39	0.19	0.520	1.48	0.22
2.0	0.004	0.68	0.02	0.010	0.73	0.03	0.020	0.81	0.04	0.037	0.89	0.05	0.060	0.98	0.07	0.091	1.07	0.09	0.132	1.16	0.10	0.182	1.25	0.12	0.243	1.33	0.15	0.316	1.41	0.17	0.401	1.50	0.20	0.496	1.60	0.22	0.601	1.71	0.26
2.5	0.005	0.76	0.02	0.011	0.82	0.03	0.023	0.90	0.05	0.041	1.00	0.06	0.067	1.10	0.08	0.102	1.20	0.10	0.147	1.30	0.12	0.204	1.39	0.14	0.272	1.49	0.16	0.354	1.58	0.19	0.448	1.68	0.22	0.554	1.79	0.25	0.671	1.91	0.29
3.0	0.005	0.83	0.02	0.012	0.90	0.04	0.025	0.99	0.05	0.045	1.09	0.07	0.074	1.20	0.08	0.112	1.31	0.10	0.161	1.42	0.13	0.223	1.52	0.15	0.298	1.63	0.18	0.388	1.73	0.21	0.491	1.84	0.24	0.607	1.96	0.28	0.736	2.09	0.31
3.5	0.006	0.90	0.03	0.013	0.97	0.04	0.027	1.07	0.05	0.049	1.18	0.07	0.079	1.30	0.09	0.121	1.42	0.11	0.174	1.53	0.14	0.241	1.65	0.16	0.322	1.76	0.19	0.419	1.87	0.22	0.530	1.99	0.26	0.656	2.12	0.30	0.794	2.26	0.34
4.0	0.006	0.96	0.03	0.014	1.04	0.04	0.029	1.14	0.06	0.052	1.26	0.08	0.085	1.39	0.10	0.129	1.51	0.12	0.186	1.64	0.15	0.258	1.76	0.18	0.344	1.88	0.21	0.448	2.00	0.24	0.567	2.13	0.28	0.701	2.27	0.32	0.849	2.41	0.36
4.5	0.006	1.02	0.03	0.015	1.10	0.04	0.031	1.21	0.06	0.055	1.34	0.08	0.090	1.47	0.10	0.137	1.61	0.13	0.198	1.74	0.16	0.273	1.87	0.19	0.365	2.00	0.22	0.475	2.12	0.25	0.601	2.26	0.29	0.744	2.41	0.34	0.901	2.56	0.38
5.0	0.007	1.07	0.03	0.016	1.16	0.05	0.032	1.28	0.06	0.058	1.41	0.08	0.095	1.55	0.11	0.145	1.69	0.14	0.208	1.83	0.16	0.288	1.97	0.20	0.385	2.11	0.23	0.500	2.23	0.27	0.634	2.38	0.31	0.784	2.54	0.36	0.950	2.70	0.40
6.0	0.007	1.17	0.04	0.017	1.27	0.05	0.035	1.40	0.07	0.064	1.55	0.09	0.104	1.70	0.12	0.158	1.86	0.15	0.228	2.01	0.18	0.316	2.16	0.22	0.422	2.31	0.25	0.548	2.45	0.29	0.695	2.61	0.34	0.859	2.78	0.39	1.040	2.96	0.44
7.0	0.008	1.27	0.04	0.019	1.37	0.05	0.038	1.51	0.08	0.069	1.67	0.10	0.112	1.84	0.13	0.171	2.00	0.16	0.247	2.17	0.20	0.341	2.33	0.23	0.455	2.49	0.27	0.592	2.64	0.32	0.750	2.81	0.37	0.928	3.00	0.42	1.124	3.19	0.48
8.0	0.008	1.35	0.04	0.020	1.47	0.06	0.041	1.62	0.08	0.073	1.79	0.11	0.120	1.96	0.14	0.183	2.14	0.17	0.264	2.32	0.21	0.364	2.49	0.25	0.487	2.66	0.29	0.633	2.82	0.34	0.802	3.01	0.39	0.992	3.21	0.45	1.201	3.41	0.51
9.0	0.009	1.44	0.04	0.021	1.56	0.06	0.043	1.71	0.09	0.078	1.89	0.11	0.127	2.08	0.15	0.194	2.27	0.18	0.280	2.46	0.22	0.387	2.64	0.26	0.516	2.83	0.31	0.671	3.00	0.36	0.851	3.19	0.41	1.052	3.40	0.48	1.274	3.62	0.54
10.0	0.009	1.51	0.05	0.022	1.64	0.07	0.046	1.81	0.09	0.082	2.00	0.12	0.134	2.20	0.15	0.204	2.40	0.19	0.295	2.59	0.23	0.408	2.78	0.28	0.544	2.98	0.33	0.708	3.16	0.38	0.897	3.36	0.44	1.109	3.59	0.50	1.343	3.82	0.57
12.0	0.010	1.66	0.05	0.024	1.80	0.07	0.050	1.98	0.10	0.090	2.19	0.13	0.147	2.41	0.17	0.224	2.62	0.21	0.323	2.84	0.26	0.446	3.05	0.30	0.596	3.26	0.36	0.775	3.46	0.42	0.982	3.68	0.48	1.215	3.93	0.55	1.471	4.18	0.63
14.0	0.011	1.79	0.05	0.026	1.94	0.08	0.054	2.14	0.11	0.097	2.36	0.14	0.159	2.60	0.18	0.242	2.83	0.23	0.349	3.07	0.28	0.482	3.29	0.33	0.644	3.52	0.39	0.837	3.74	0.45	1.061	3.98	0.52	1.312	4.24	0.59	1.589	4.51	0.68
16.0	0.012	1.91	0.06	0.028	2.07	0.08	0.058	2.28	0.11	0.104	2.53	0.15	0.170	2.78	0.19	0.259	3.03	0.24	0.373	3.28	0.30	0.515	3.52	0.35	0.689	3.77	0.41	0.895	3.99	0.48	1.134	4.25	0.55	1.403	4.54	0.64	1.699	4.83	0.72

Chart No. A3-5

ROAD FLOW CAPACITY TABLE

Qudm

8.0 m road (invert to invert)

1/2 road flows only

Widths (W) and depths (d_g) are measured from the invert of K & C

Crossfall of 2.5% (1 in 40)

d_g = 0.16 to 0.28 m

K & C Type M1

K & C Manning's n = 0.013

Pavement Manning's n = 0.015

— Flows above crown of road —

Grade %	d_g=0.16 A=0.395 W=4.00			d_g=0.17 A=0.437 W=4.00			d_g=0.18 A=0.480 W=4.00			d_g=0.19 A=0.523 W=4.00			d_g=0.20 A=0.566 W=4.00			d_g=0.21 A=0.608 W=4.00			d_g=0.22 A=0.651 W=4.00			d_g=0.23 A=0.694 W=4.00			d_g=0.24 A=0.737 W=4.00			d_g=0.25 A=0.779 W=4.00			d_g=0.26 A=0.822 W=4.00			d_g=0.27 A=0.865 W=4.00			d_g=0.28 A=0.908 W=4.00		
	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s	Q m³/s	V m/s	d_g.V m²/s
0.3	0.277	0.70	0.11	0.324	0.74	0.13	0.375	0.78	0.14	0.429	0.82	0.16	0.486	0.86	0.17	0.545	0.90	0.19	0.608	0.93	0.21	0.674	0.97	0.22	0.742	1.01	0.24	0.813	1.04	0.26	0.887	1.08	0.28	0.963	1.11	0.30	1.042	1.15	0.32
0.5	0.357	0.90	0.14	0.418	0.96	0.16	0.484	1.01	0.18	0.553	1.06	0.20	0.627	1.11	0.22	0.704	1.16	0.24	0.785	1.21	0.27	0.870	1.25	0.29	0.958	1.30	0.31	1.050	1.35	0.34	1.145	1.39	0.36	1.243	1.44	0.39	1.345	1.48	0.41
1.0	0.505	1.28	0.20	0.592	1.35	0.23	0.684	1.43	0.26	0.783	1.50	0.28	0.886	1.57	0.31	0.996	1.64	0.34	1.110	1.71	0.38	1.230	1.77	0.41	1.355	1.84	0.44	1.484	1.90	0.48	1.619	1.97	0.51	1.758	2.03	0.55	1.902	2.10	0.59
1.5	0.619	1.57	0.25	0.725	1.66	0.28	0.838	1.75	0.31	0.958	1.83	0.35	1.086	1.92	0.38	1.220	2.00	0.42	1.360	2.09	0.46	1.506	2.17	0.50	1.659	2.25	0.54	1.818	2.33	0.58	1.983	2.41	0.63	2.153	2.49	0.67	2.330	2.57	0.72
2.0	0.714	1.81	0.29	0.837	1.91	0.33	0.968	2.02	0.36	1.107	2.12	0.40	1.254	2.22	0.44	1.408	2.31	0.49	1.570	2.41	0.53	1.739	2.51	0.58	1.916	2.60	0.62	2.099	2.69	0.67	2.289	2.78	0.72	2.486	2.87	0.78	2.690	2.96	0.83
2.5	0.799	2.02	0.32	0.936	2.14	0.36	1.082	2.25	0.41	1.237	2.37	0.45	1.402	2.48	0.50	1.574	2.59	0.54	1.756	2.70	0.59	1.945	2.80	0.64	2.142	2.91	0.70	2.347	3.01	0.75	2.560	3.11	0.81	2.780	3.21	0.87	3.077	3.31	0.93
3.0	0.875	2.22	0.35	1.025	2.34	0.40	1.185	2.47	0.44	1.356	2.59	0.49	1.535	2.71	0.54	1.725	2.83	0.60	1.923	2.95	0.65	2.130	3.07	0.71	2.346	3.19	0.76	2.571	3.30	0.82	2.804	3.41	0.89	3.045	3.52	0.95	3.294	3.63	1.02
3.5	0.945	2.39	0.38	1.107	2.53	0.43	1.280	2.67	0.48	1.464	2.80	0.53	1.658	2.93	0.59	1.863	3.06	0.64	2.077	3.19	0.70	2.301	3.32	0.76	2.534	3.44	0.83	2.777	3.56	0.89	3.029	3.68	0.96	3.289	3.80	1.03	3.558	3.92	1.10
4.0	1.010	2.56	0.41	1.183	2.71	0.46	1.369	2.85	0.51	1.565	2.99	0.57	1.773	3.13	0.63	1.992	3.27	0.69	2.221	3.41	0.75	2.460	3.54	0.82	2.709	3.68	0.88	2.969	3.81	0.95	3.238	3.94	1.02	3.516	4.07	1.10	3.804	4.19	1.17
4.5	1.071	2.71	0.43	1.255	2.87	0.49	1.452	3.02	0.54	1.660	3.17	0.60	1.880	3.32	0.66	2.112	3.47	0.73	2.355	3.62	0.80	2.609	3.76	0.86	2.874	3.90	0.94	3.149	4.04	1.01	3.434	4.18	1.09	3.730	4.31	1.16	4.035	4.45	1.24
5.0	1.129	2.86	0.46	1.323	3.02	0.51	1.530	3.19	0.57	1.750	3.35	0.64	1.982	3.50	0.70	2.227	3.66	0.77	2.483	3.81	0.84	2.750	3.96	0.91	3.029	4.11	0.99	3.319	4.26	1.06	3.620	4.40	1.14	3.931	4.55	1.23	4.253	4.69	1.31
6.0	1.237	3.13	0.50	1.449	3.31	0.56	1.676	3.49	0.63	1.917	3.67	0.70	2.171	3.84	0.77	2.439	4.01	0.84	2.720	4.18	0.92	3.013	4.34	1.00	3.318	4.50	1.08	3.636	4.66	1.17	3.965	4.82	1.25	4.307	4.98	1.34	4.659	5.13	1.44
7.0	1.336	3.39	0.54	1.566	3.58	0.61	1.810	3.77	0.68	2.071	3.96	0.75	2.345	4.15	0.83	2.635	4.33	0.91	2.938	4.51	0.99	3.254	4.69	1.08	3.584	4.87	1.17	3.927	5.04	1.26	4.283	5.21	1.35	4.652	5.38	1.45	5.032	5.54	1.55
8.0	1.429	3.62	0.58	1.674	3.83	0.65	1.935	4.03	0.73	2.214	4.23	0.80	2.507	4.43	0.89	2.816	4.63	0.97	3.140	4.82	1.06	3.479	5.01	1.15	3.832	5.20	1.25	4.199	5.39	1.35	4.579	5.57	1.45	4.973	5.75	1.55	5.380	5.93	1.66
9.0	1.515	3.84	0.61	1.775	4.06	0.69	20.53	4.28	0.77	2.348	4.49	0.85	2.659	4.70	0.94	2.987	4.91	1.03	3.331	5.12	1.13	3.690	5.32	1.22	4.064	5.52	1.32	4.453	5.71	1.43	4.857	5.91	1.54	5.274	6.10	1.65	5.706	6.29	1.76
10.0	1.597	4.05	0.65	1.871	4.28	0.73	2.164	4.51	0.81	2.475	4.73	0.90	2.803	4.96	0.99	3.149	5.18	1.09	3.511	5.39	1.19	3.890	5.61	1.29	4.284	5.82	1.40	4.694	6.02	1.51	5.119	6.23	1.62	5.560	6.43	1.74	6.015	6.63	1.86
12.0	1.750	4.43	0.71	2.050	4.69	0.80	2.370	4.94	0.89	2.711	5.16	0.98	3.071	5.43	1.09	3.449	5.67	1.19	3.846	5.91	1.30	4.261	6.14	1.41	4.693	6.37	1.53	5.142	6.60	1.65	5.608	6.82	1.77	6.090	7.04	1.90	6.589	7.26	2.03
14.0	1.890	4.79	0.77	2.214	5.06	0.86	2.560	5.33	0.96	2.938	5.60	1.06	3.317	5.86	1.17	3.726	6.12	1.29	4.154	6.38	1.40	4.602	6.63	1.53	5.069	6.88	1.65	5.554	7.13	1.78	6.057	7.37	1.92	6.578	7.61	2.05	7.117	7.84	2.20
16.0	2.020	5.12	0.82	2.367	5.41	0.92	2.737	5.70	1.03	3.130	5.99	1.14	3.546	6.27	1.25	3.983	6.55	1.37	4.441	6.82	1.50	4.920	7.09	1.63	5.419	7.36	1.77	5.938	7.62	1.90	6.476	7.88	2.05	7.033	8.13	2.20	7.608	8.38	2.35

Chart No. A3-6

ROAD FLOW CAPACITY TABLE

Qudm — K & C Type B1

12.0 m road (invert to invert)   Crossfall of 2.5% (1 in 40)

1/2 road flows only d_g = 0.04 to 0.15   K & C Manning's n = 0.013

Widths (W) and depths (d_g) are measured from the invert of K & C   Pavement Manning's n = 0.015

— Flows below crown of road —

Grade %	d_g=0.04 A=0.010 W=0.55 Q m³/s	V m/s	d_g.V m²/s	d_g=0.05 A=0.017 W=0.95 Q m³/s	V m/s	d_g.V m²/s	d_g=0.06 A=0.029 W=1.35 Q m³/s	V m/s	d_g.V m²/s	d_g=0.07 A=0.044 W=1.75 Q m³/s	V m/s	d_g.V m²/s	d_g=0.08 A=0.064 W=2.15 Q m³/s	V m/s	d_g.V m²/s	d_g=0.09 A=0.087 W=2.55 Q m³/s	V m/s	d_g.V m²/s	d_g=0.10 A=0.115 W=2.95 Q m³/s	V m/s	d_g.V m²/s	d_g=0.11 A=0.146 W=3.35 Q m³/s	V m/s	d_g.V m²/s	d_g=0.12 A=0.182 W=3.75 Q m³/s	V m/s	d_g.V m²/s	d_g=0.13 A=0.221 W=4.15 Q m³/s	V m/s	d_g.V m²/s	d_g=0.14 A=0.265 W=4.55 Q m³/s	V m/s	d_g.V m²/s	d_g=0.15 A=0.312 W=4.95 Q m³/s	V m/s	d_g.V m²/s
0.3	0.003	0.33	0.01	0.006	0.35	0.02	0.011	0.37	0.02	0.018	0.40	0.03	0.027	0.43	0.03	0.040	0.46	0.04	0.057	0.49	0.05	0.077	0.53	0.06	0.101	0.56	0.07	0.130	0.59	0.08	0.164	0.62	0.09	0.203	0.65	0.10
0.5	0.004	0.43	0.02	0.008	0.46	0.02	0.014	0.48	0.03	0.023	0.52	0.04	0.035	0.55	0.04	0.052	0.60	0.05	0.073	0.64	0.06	0.099	0.68	0.07	0.131	0.72	0.09	0.168	0.76	0.10	0.212	0.80	0.11	0.263	0.84	0.13
1.0	0.006	0.60	0.02	0.011	0.64	0.03	0.020	0.68	0.04	0.032	0.73	0.05	0.050	0.78	0.06	0.073	0.84	0.08	0.103	0.90	0.09	0.140	0.96	0.11	0.185	1.02	0.12	0.238	1.08	0.14	0.300	1.13	0.16	0.371	1.19	0.18
1.5	0.007	0.74	0.03	0.014	0.79	0.04	0.024	0.84	0.05	0.040	0.89	0.06	0.061	0.96	0.08	0.090	1.03	0.09	0.127	1.10	0.11	0.172	1.18	0.13	0.227	1.25	0.15	0.292	1.32	0.17	0.367	1.39	0.19	0.455	1.46	0.22
2.0	0.008	0.85	0.03	0.016	0.91	0.05	0.028	0.96	0.06	0.046	1.03	0.07	0.071	1.11	0.09	0.104	1.19	0.11	0.146	1.27	0.13	0.198	1.36	0.15	0.262	1.44	0.17	0.337	1.52	0.20	0.424	1.60	0.22	0.525	1.68	0.25
2.5	0.009	0.95	0.04	0.017	1.02	0.05	0.031	1.08	0.06	0.051	1.15	0.08	0.079	1.24	0.10	0.116	1.33	0.12	0.163	1.42	0.14	0.222	1.52	0.17	0.293	1.61	0.19	0.376	1.70	0.22	0.474	1.79	0.25	0.587	1.88	0.28
3.0	0.010	1.04	0.04	0.019	1.12	0.06	0.034	1.18	0.07	0.056	1.26	0.09	0.087	1.36	0.11	0.127	1.46	0.13	0.179	1.56	0.16	0.243	1.66	0.18	0.321	1.76	0.21	0.412	1.86	0.24	0.520	1.96	0.27	0.643	2.06	0.31
3.5	0.011	1.13	0.05	0.021	1.20	0.06	0.037	1.28	0.08	0.060	1.37	0.10	0.093	1.47	0.12	0.137	1.58	0.14	0.193	1.69	0.17	0.263	1.80	0.20	0.346	1.91	0.23	0.445	2.01	0.26	0.561	2.12	0.30	0.695	2.23	0.33
4.0	0.012	1.21	0.05	0.022	1.29	0.06	0.039	1.36	0.08	0.065	1.46	0.10	0.100	1.57	0.13	0.147	1.68	0.15	0.207	1.80	0.18	0.281	1.92	0.21	0.370	2.04	0.24	0.476	2.15	0.28	0.600	2.27	0.32	0.743	2.38	0.36
4.5	0.012	1.28	0.05	0.023	1.37	0.07	0.041	1.45	0.09	0.068	1.55	0.11	0.106	1.66	0.13	0.156	1.79	0.16	0.219	1.91	0.19	0.298	2.04	0.22	0.393	2.16	0.26	0.505	2.28	0.30	0.636	2.40	0.34	0.788	2.52	0.38
5.0	0.013	1.35	0.05	0.025	1.44	0.07	0.044	1.52	0.09	0.072	1.63	0.11	0.112	1.75	0.14	0.164	1.88	0.17	0.231	2.01	0.20	0.314	2.15	0.24	0.414	2.28	0.27	0.532	2.41	0.31	0.671	2.53	0.35	0.831	2.66	0.40
6.0	0.014	1.48	0.06	0.027	1.58	0.08	0.048	1.67	0.10	0.079	1.79	0.13	0.122	1.92	0.15	0.180	2.06	0.19	0.253	2.21	0.22	0.344	2.35	0.26	0.453	2.49	0.30	0.583	2.64	0.34	0.735	2.78	0.39	0.910	2.91	0.44
7.0	0.015	1.59	0.06	0.029	1.70	0.09	0.052	1.80	0.11	0.085	1.93	0.14	0.132	2.08	0.17	0.194	2.23	0.20	0.273	2.38	0.24	0.371	2.54	0.28	0.490	2.69	0.32	0.630	2.85	0.37	0.794	3.00	0.42	0.983	3.15	0.47
8.0	0.017	1.70	0.07	0.031	1.82	0.09	0.055	1.93	0.12	0.091	2.06	0.14	0.141	2.22	0.18	0.208	2.38	0.21	0.292	2.55	0.25	0.397	2.71	0.30	0.523	2.88	0.35	0.673	3.04	0.40	0.849	3.21	0.45	1.051	3.37	0.50
9.0	0.018	1.81	0.07	0.033	1.93	0.10	0.059	2.05	0.12	0.097	2.19	0.15	0.150	2.35	0.19	0.220	2.53	0.23	0.310	2.70	0.27	0.421	2.88	0.32	0.555	3.06	0.37	0.714	3.23	0.42	0.900	3.40	0.48	1.114	3.57	0.54
10.0	0.019	1.91	0.08	0.035	2.04	0.10	0.062	2.16	0.13	0.102	2.31	0.16	0.158	2.48	0.20	0.232	2.66	0.24	0.327	2.85	0.28	0.444	3.04	0.33	0.585	3.22	0.39	0.753	3.40	0.44	0.949	3.58	0.50	1.175	3.76	0.56
12.0	0.020	2.09	0.08	0.038	2.23	0.11	0.068	2.36	0.14	0.112	2.53	0.18	0.173	2.72	0.22	0.254	2.92	0.26	0.358	3.12	0.31	0.486	3.33	0.37	0.641	3.53	0.42	0.825	3.73	0.48	1.039	3.93	0.55	1.287	4.12	0.62
14.0	0.022	2.25	0.09	0.041	2.41	0.12	0.073	2.55	0.15	0.121	2.73	0.19	0.187	2.94	0.23	0.275	3.15	0.28	0.387	3.37	0.34	0.525	3.59	0.40	0.692	3.81	0.46	0.891	4.03	0.52	1.123	4.24	0.59	1.390	4.45	0.67
16.0	0.023	2.41	0.10	0.044	2.58	0.13	0.078	2.73	0.16	0.129	2.92	0.20	0.200	3.14	0.25	0.294	3.37	0.30	0.413	3.60	0.36	0.561	3.84	0.42	0.740	4.07	0.49	0.952	4.31	0.56	1.200	4.53	0.63	1.486	4.76	0.71

Chart No. A3-7

ROAD FLOW CAPACITY TABLE

Qudm

12.0 m road (invert to invert)   Crossfall of 2.5% (1 in 40)   **K & C Type B1**

1/2 road flows only d_g = 0.16 to 0.27 m   K & C Manning's n = 0.013

Widths (W) and depths (d_g) are measured from the invert of K & C   Pavement Manning's n = 0.015

←— Flows below crown of road —→   Flows above crown of road —→

Grade %	d_g=0.16 A=0.364 W=5.35 Q m³/s	V m/s	d_g.V m²/s	d_g=0.17 A=0.419 W=5.75 Q m³/s	V m/s	d_g.V m²/s	d_g=0.18 A=0.478 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.19 A=0.538 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.20 A=0.598 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.21 A=0.658 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.22 A=0.718 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.23 A=0.778 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.24 A=0.838 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.25 A=0.898 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.26 A=0.958 W=6.00 Q m³/s	V m/s	d_g.V m²/s	d_g=0.27 A=1.018 W=6.00 Q m³/s	V m/s	d_g.V m²/s
0.3	0.248	0.68	0.11	0.298	0.71	0.12	0.355	0.74	0.13	0.417	0.77	0.15	0.485	0.81	0.16	0.557	0.85	0.18	0.635	0.88	0.19	0.717	0.92	0.21	0.803	0.96	0.23	0.894	1.00	0.25	0.990	1.03	0.27	1.089	1.07	0.29
0.5	0.320	0.88	0.14	0.385	0.92	0.16	0.458	0.96	0.17	0.538	1.00	0.19	0.626	1.05	0.21	0.719	1.09	0.23	0.819	1.14	0.25	0.925	1.19	0.27	1.037	1.24	0.30	1.155	1.29	0.32	1.278	1.33	0.35	1.406	1.38	0.37
1.0	0.453	1.25	0.20	0.545	1.30	0.22	0.648	1.35	0.24	0.761	1.41	0.27	0.885	1.48	0.30	1.017	1.55	0.32	1.159	1.61	0.35	1.309	1.68	0.39	1.467	1.75	0.42	1.633	1.82	0.45	1.807	1.89	0.49	1.988	1.95	0.53
1.5	0.555	1.53	0.24	0.667	1.59	0.27	0.793	1.66	0.30	0.932	1.73	0.33	1.084	1.81	0.36	1.246	1.89	0.40	1.419	1.98	0.43	1.603	2.06	0.47	1.796	2.14	0.51	2.000	2.23	0.56	2.213	2.31	0.60	2.435	2.39	0.65
2.0	0.640	1.76	0.28	0.770	1.84	0.31	0.916	1.91	0.34	1.077	2.00	0.38	1.251	2.09	0.42	1.439	2.19	0.46	1.639	2.28	0.50	1.851	2.38	0.55	2.074	2.47	0.59	2.309	2.57	0.64	2.555	2.67	0.69	2.812	2.76	0.75
2.5	0.716	1.97	0.32	0.861	2.05	0.35	1.024	2.14	0.39	1.204	2.24	0.42	1.399	2.34	0.47	1.608	2.44	0.51	1.832	2.55	0.56	2.069	2.66	0.61	2.319	2.77	0.66	2.582	2.87	0.72	2.857	2.98	0.77	3.144	3.09	0.83
3.0	0.784	2.16	0.35	0.944	2.25	0.38	1.122	2.35	0.42	1.319	2.45	0.47	1.532	2.56	0.51	1.762	2.68	0.56	2.007	2.79	0.61	2.267	2.91	0.67	2.541	3.03	0.73	2.828	3.15	0.79	3.129	3.27	0.85	3.444	3.38	0.91
3.5	0.847	2.33	0.37	1.019	2.43	0.41	1.212	2.53	0.46	1.424	2.65	0.50	1.655	2.77	0.55	1.903	2.89	0.61	2.168	3.02	0.66	2.448	3.15	0.72	2.744	3.27	0.79	3.055	3.40	0.85	3.380	3.53	0.92	3.720	3.65	0.99
4.0	0.906	2.49	0.40	1.090	2.60	0.44	1.296	2.71	0.49	1.523	2.83	0.54	1.769	2.96	0.59	2.035	3.09	0.65	2.317	3.23	0.71	2.617	3.36	0.77	2.934	3.50	0.84	3.266	3.64	0.91	3.614	3.77	0.98	3.977	3.90	1.05
4.5	0.961	2.64	0.42	1.156	2.76	0.47	1.374	2.87	0.52	1.615	3.00	0.57	1.877	3.14	0.63	2.158	3.28	0.69	2.458	3.42	0.75	2.776	3.57	0.82	3.111	3.71	0.89	3.464	3.86	0.96	3.833	4.00	1.04	4.218	4.14	1.12
5.0	1.013	2.78	0.45	1.218	2.91	0.49	1.448	3.03	0.54	1.702	3.16	0.60	1.978	3.31	0.66	2.275	3.45	0.73	2.591	3.61	0.79	2.926	3.76	0.86	3.280	3.91	0.94	3.651	4.06	1.02	4.040	4.22	1.10	4.446	4.37	1.18
6.0	1.109	3.05	0.49	1.334	3.18	0.54	1.587	3.32	0.60	1.865	3.46	0.66	2.167	3.62	0.72	2.492	3.78	0.79	2.838	3.95	0.87	3.205	4.12	0.95	3.593	4.29	1.03	4.000	4.45	1.11	4.426	4.62	1.20	4.870	4.78	1.29
7.0	1.198	3.29	0.53	1.441	3.44	0.58	1.714	3.58	0.64	2.014	3.74	0.71	2.341	3.91	0.78	2.692	4.09	0.86	3.066	4.27	0.94	3.462	4.45	1.02	3.881	4.63	1.11	4.320	4.81	1.20	4.780	4.99	1.30	5.260	5.17	1.39
8.0	1.281	3.52	0.56	1.541	3.68	0.62	1.832	3.83	0.69	2.153	4.00	0.76	2.502	4.18	0.84	2.877	4.37	0.92	3.277	4.56	1.00	3.701	4.76	1.09	4.149	4.95	1.19	4.618	5.14	1.29	5.110	5.33	1.39	5.624	5.52	1.49
9.0	1.359	3.74	0.60	1.634	3.90	0.66	1.943	4.06	0.73	2.284	4.24	0.81	2.654	4.44	0.89	3.052	4.64	0.97	3.476	4.84	1.06	3.926	5.04	1.16	4.400	5.25	1.26	4.899	5.45	1.36	5.420	5.66	1.47	5.965	5.86	1.58
10.0	1.432	3.94	0.63	1.723	4.11	0.70	2.048	4.28	0.77	2.408	4.47	0.85	2.798	4.68	0.94	3.217	4.89	1.03	3.664	5.10	1.12	4.138	5.32	1.22	4.638	5.53	1.33	5.164	5.75	1.44	5.714	5.96	1.55	6.287	6.17	1.67
12.0	1.569	4.31	0.69	1.887	4.50	0.77	2.244	4.69	0.84	2.637	4.90	0.93	3.065	5.12	1.02	3.524	5.35	1.12	4.014	5.59	1.23	4.533	5.82	1.34	5.081	6.06	1.45	5.656	6.30	1.57	6.259	6.53	1.70	6.888	6.76	1.83
14.0	1.694	4.66	0.75	2.038	4.86	0.83	2.424	5.07	0.91	2.849	5.29	1.01	3.310	5.53	1.11	3.806	5.78	1.21	4.336	6.03	1.33	4.896	6.29	1.45	5.488	6.55	1.57	6.110	6.80	1.70	6.760	7.05	1.83	7.439	7.30	1.97
16.0	1.811	4.98	0.80	2.179	5.20	0.88	2.591	5.42	0.97	3.045	5.66	1.07	3.539	5.91	1.18	4.069	6.18	1.30	4.635	6.45	1.42	5.235	6.72	1.55	5.867	7.00	1.68	6.532	7.27	1.82	7.227	7.54	1.96	7.953	7.81	2.11

Chart No. A3-8