ARR
Australian Rainfall & Runoff

A GUIDE TO FLOOD ESTIMATION

BOOK 9 - RUNOFF IN URBAN AREAS

Version 4.2

Australian Government
ENGINEERS AUSTRALIA

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PREFACE

Since its first publication in 1958, Australian Rainfall and Runoff (ARR) has remained one of the most influential and widely used guidelines published by Engineers Australia (EA). The 3rd edition, published in 1987, retained the same level of national and international acclaim as its predecessors.

With nationwide applicability, balancing the varied climates of Australia, the information and the approaches presented in Australian Rainfall and Runoff are essential for policy decisions and projects involving:

- infrastructure such as roads, rail, airports, bridges, dams, stormwater and sewer systems;
- town planning;
- mining;
- developing flood management plans for urban and rural communities;
- flood warnings and flood emergency management;
- operation of regulated river systems; and
- prediction of extreme flood levels.

However, many of the practices recommended in the 1987 edition of ARR have become outdated, and no longer represent industry best practice. This fact, coupled with the greater understanding of climate and flood hydrology derived from the larger data sets now available to us, has provided the primary impetus for revising these guidelines. It is hoped that this revision will lead to improved design practice, which will allow better management, policy and planning decisions to be made.

One of the major responsibilities of the National Committee on Water Engineering of Engineers Australia is the periodic revision of ARR. While the NCWE had long identified the need to update ARR it had become apparent by 2002 that even with a piecemeal approach the task could not be carried out without significant financial support. In 2008 the revision of ARR was identified as a priority in the National Adaptation Framework for Climate Change which was endorsed by the Council of Australian Governments.

In addition to the update, 21 projects were identified with the aim of filling knowledge gaps. Funding for Stages 1 and 2 of the ARR revision projects were provided by the now Department of the Environment. Stage 3 was funded by Geoscience Australia. Funding for Stages 2 and 3 of Project 1 (Development of Intensity-Frequency-Duration information across Australia) has been provided by the Bureau of Meteorology. The outcomes of the projects assisted the ARR Editorial Team with the compiling and writing of chapters in the revised ARR. Steering and Technical Committees were established to assist the ARR Editorial Team in guiding the projects to achieve desired outcomes.

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Status of this document

This document is a living document and will be regularly updated in the future.

In development of this guidance, and discussed in Book 1 of ARR 1987, it was recognised that knowledge and information availability is not fixed and that future research and applications will develop new techniques and information. This is particularly relevant in applications where techniques have been extrapolated from the region of their development to other regions and where efforts should be made to reduce large uncertainties in current estimates of design flood characteristics.

Therefore, where circumstances warrant, designers have a duty to use other procedures and design information more appropriate for their design flood problem. The Editorial team of this edition of Australian Rainfall and Runoff believe that the use of new or improved procedures should be encouraged, especially where these are more appropriate than the methods described in this publication.

Care should be taken when combining inputs derived using ARR 1987 and methods described in this document.

Change Log

Version 4.2 - Climate Change Chapter Update

In late 2022 the Australian Government Department of Climate Change, Energy, the Environment and Water in partnership with Engineers Australia commenced an 18 month project to update the climate change considerations chapter of the Australian Rainfall and Runoff guidelines (Chapter 6, Book 1) to incorporate the most recent and relevant climate science and projections. The project involved the undertaking of a rigorous literature review of hydroclimatology under climate change relevant to design flood estimation, which was peer reviewed and published in a leading international journal. The findings were used to draft practical flood guidance which was finalised after an extensive process of review and feedback by industry. Funding for this project was received from National Emergency Management Agency under the Disaster Risk Reduction Package. The project report was adapted to replace Book 1 chapter 6.

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This version updates Book 1 Chapter 6 to reflect updates in climate science as discussed above. While no other chapters have been updated some minor amendments were made to remove inconsistencies with the new chapter. FAQs relating to the update are available https://arr.ga.gov.au/contact-us.

Key updates in Version 4.2

Update	Version 4.2
Book 1	Book 1 Chapter 6 Climate change updated
Guideline formats	PDF Web-based version Epub version
User experience	FAQs added to Geoscience Australia Website
Climate change	Reflected best practice as of 2024 and IPCC 6
Other Minor Changes	List the minor changes to the following chapters for consistency Book 1 Chapter 4 Section 15.1 Book 1 Chapter 4 Section 16.1 Book 1 Chapter 5 Section 10.4 Book 2 Chapter 1 Section 3 Book 2 Chapter 3 Section 3 Book 6 Chapter 5 Section 5 Book 8 Chapter 7 Section 7 Book 9 Chapter 6 Section 4.2 Book 9 Chapter 6 Section 4.6

ARR 2019 (now Version 4.1)

Geoscience Australia, on behalf of the Australian Government, asked the National Committee on Water Engineers (NCWE) - a specialist committee of Engineers Australia - to continue overseeing the technical direction of ARR. ARR's success comes from practitioners and researchers driving its development; and the NCWE is the appropriate organisation to oversee this work. The NCWE has formed a sub-committee to lead the ongoing management and development of ARR for the benefit of the Australian community and the profession. The current membership of the ARR management subcommittee includes Mark Babister, Robin Connolly, Rory Nathan and Bill Weeks.

The ARR team have been working hard on finalising ARR since it was released in 2016. The team has received a lot of feedback from industry and practitioners, ranging from substantial feedback to minor typographical errors. Much of this feedback has now been addressed. Where a decision has been made not to address the feedback, advice has been provided as to why this was the case.

A new version of ARR is now available. ARR 2019 is a result of extensive consultation and feedback from practitioners. Noteworthy updates include the completion of Book 9, reflection of current climate change practice and improvements to user experience, including the availability of the document as a PDF.

Key updates in ARR 2019

Update	ARR 2016	ARR 2019
Book 9	Available as “rough” draft	Peer reviewed and completed
Guideline formats	Epub version Web-based version	Following practitioner feedback, a pdf version of ARR 2019 is now available
User experience	Limited functionality in web-based version	Additional pdf format available
Climate change	Reflected best practice as of 2016 Climate Change policies	Updated to reflect current practice
PMF chapter	Updated from the guidance provided in 1998 to include current best practice	Minor edits and reflects differences required for use in dam studies and floodplain management
Examples		Examples included for Book 9
Figures		Updated reflecting practitioner feedback

As of May 2019, this version was considered to be final.

ARR 2016 (now Version 4.0)
Released July 2016

BOOK 9

Runoff in Urban Areas

Runoff in Urban Areas

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Chapter 1. Introduction

Peter Coombes, Steve Roso

Chapter Status	Final
Date last updated	14/5/2019

1.1. Introduction

There have been profound changes to the science and practice of urban hydrology and stormwater management since the last edition of Australian Rainfall and Runoff (ARR) published in 1987 (Pilgrim, 1987). During this period analysis methods have evolved from use of the slide rule to the computer age and beyond. The revision of ARR has aimed for an evidence based approach that incorporates 30 years of additional data, science and knowledge. This includes a move away from simple design rainfall burst event methods towards Ensemble and Monte Carlo approaches to better capture variability. There is less reliance on the rational method, more data available, new Intensity Frequency Duration (IFD) data and better flow estimates for ungauged catchments (refer to Book 3, Chapter 3).

There are new challenges and gaps in knowledge about urban hydrology that is part of an increasingly complex urban water cycle and town planning processes. The designer now aims to retain stormwater within urban landscapes, manage stormwater quality, maximize the potential of the stormwater resource and to slow flows into receiving waterways. Australian Rainfall and Runoff now employs Australian data which ensures that urban designers can better represent real local systems and address these new challenges.

Wherever possible this version of ARR provides information about the uncertainty of methods and inputs. This will better equip urban designers to understand risks in the urban environment. The Urban Book (Book 9 – Runoff in Urban Areas) has been constructed to utilise and complement the broader set of tools in ARR used to manage the water cycle. The over-arching objective of this book is to provide revised and up-to-date guidance for analysis and management of urban stormwater runoff.

1.1.1. Urban Stormwater Runoff

Urban stormwater runoff and associated stormwater management responses are part of a linked urban water cycle which includes stormwater quantity and quality, water supply, sewerage, urban form and waterways. Urban runoff has hydrologic characteristics such as flow rate and volume which differ considerably from natural and rural systems. As a result there is significant potential for impacts on natural processes and on society. These include nuisance flooding, disruption of traffic and business functions, flood disasters and damage, stream erosion, and destruction of natural waterway form and function. These water balance and linked systems issues are discussed in Book 9, Chapter 2 and Book 9, Chapter 3.

Whilst urban runoff can be a problem to be managed, it is also a potential opportunity to be exploited if viewed as an environmental resource. There are urban runoff design and investigation techniques that can be used to achieve better economic, social and environmental outcomes. The discussion of managing urban stormwater runoff in this Book also intersects with managing stormwater quality which is addressed in a number of guidelines throughout Australia such as Australian Runoff Quality. Many of these practices are introduced in this book.

1.1.2. Stormwater Management Infrastructure

Urban runoff was traditionally managed using networks of pipes and channels to convey stormwater rapidly away from urban areas. The definition of drainage has now broadened to incorporate both conveyance and management of stormwater volumes via a wider range of measures, including natural and man-made infrastructure to restore natural flood behavior where possible.

Two classes of stormwater management infrastructure are described in this book; volume management (Book 9, Chapter 4) and conveyance systems (Book 9, Chapter 5).

Volume management includes measures that can store runoff for a period of time, promote infiltration and store harvested stormwater for beneficial uses. Modern best practice aims to achieve a range of hydrologic and water quality objectives within these facilities. Volume management is a key element of stormwater management and flood control which has increased in importance and will continue to evolve into the future. Stormwater volume controls have been subject to substantial and increased research effort since 1987.

Conveyance systems allow runoff to pass through urban areas and provide connections through the catchment. Conveyance systems can be classified in different ways, for example underground versus surface and trunk versus non-trunk. The traditional description of urban stormwater management involves a minor and major event management philosophy where the minor concept involves pipe drainage networks and the major concept addresses flood events that are conveyed as surface flows. A minor versus major design concept is also still relevant in order to efficiently convey urban runoff while mitigating nuisance, damage and disaster. Regardless, the focus for conveyance should be careful management of surface flows and restoration of natural flow behaviour wherever possible.

Volume management facilities and conveyance systems are interlinked to form a network with volume management most often at discrete locations connected by more linear conveyance systems. Both conveyance and volume management can exist at multiple scales from lot scale (source control) to regional scale (end of pipe). In the context of Book 9, natural and semi-natural urban waterways are considered part of the network of conveyance and storage infrastructure.

1.1.3. Modelling

The unique characteristics of urban modelling include measurement and assessment of the hydrologic and hydraulic effect of impervious surfaces, conveyance systems and hydraulic structures including volume management facilities. Analysis of urban areas involves data intensive and complex processes. There is a need for complex computing tasks aided by software to assist with modern investigation. A wide range of computer software is available to the designer. Hand calculations are generally unsuitable for most urban applications other than basic checks and approximations.

Choice of computer software such as urban hydrology and hydraulic models depends on a number of factors including the spatial scale of the investigation area and the magnitude of the floods of interest. Book 9, Chapter 6 provides guidance on how to pick a short list of suitable models based on these factors. The aim should be to best match the selected model with the type of investigation being undertaken.

Once a suitable model has been selected, the challenge is to ensure the model is applied correctly. Book 9, Chapter 6 does not provide guidance on how to use specific modelling software and instead describes the urban modelling process in a software independent manner. Some models can be simplified and the physical resolution reduced, depending on the spatial scale of the investigation and experience of the modelling team. Urban modelling frameworks are described providing guidance for key segments of urban catchments from the behaviour of land uses within sub-catchments that flow to inlet structures, through urban stormwater networks, and into the receiving waterway.

1.1.4. Structure and Purpose of this Book

This Urban Book is a guideline rather than a standard or recipe as Australia is too diverse and the urban practice involves increasing complex combinations of solutions. A primary audience of this book includes readers from multiple disciplines and early career urban designers.

This book focusses on the entire spectrum of runoff events and potential flooding outcomes. Book 9, Chapter 2 provides an overview of the characteristics of urban hydrology. Book 9, Chapter 3 introduces some of the key concepts in urban stormwater management as part of an urban water cycle and urban systems. It is built around Book 9, Chapter 4 and Book 9, Chapter 5 which describe the key stormwater design elements of volume management and conveyance. Book 9, Chapter 6 provides guidance on urban modelling including model selection and application. Two case studies are also provided in Book 9, Chapter 6.

1.1.5. The Future

There is a need to allow changes in interpretation of the stormwater components of this book to accommodate contemporary and integrated approaches to water cycle management in urban areas, which starts with the integration of land and water planning across time horizons and spatial scales. This guidance encourages advances in urban water cycle management, and expects advances in science and professional practice over the next 30 years. There is an enabling framework of guidance in all ARR Books, which encourages and permits advanced analysis techniques and innovative designs. The guidance in ARR does not intend to hold back advances in analysis of integrated solutions.

In some jurisdictions, there has been disproportionate focus on mitigating nuisance in the minor system at the expense of a proper analysis of the major system. Replacement of the minor or major drainage approach with the relativity of mitigating nuisance or disaster may be a future innovation of stormwater management. Allowing space for a major system can help manage large events and provides flexibility for adapting stormwater management to incorporate integrated systems and better management of nuisance.

It is expected that policy frameworks will evolve to further integrate land and water management with design processes at all spatial scales from local to regional and which also applies to urban renewal and asset renewal or replacement choices. Future design methods fomakr integrated solutions are likely to include most of the variability of real rainfall events by using continuous simulation, Monte Carlo frameworks and techniques that consider complete storms, frequency of rainfall volumes and the spatial variability of events.

Good urban runoff management will only be achieved when it is integrated with the complete management of the urban water cycle and includes proper consideration of runoff quality. The guidance in the Urban Book must be linked with Australian Runoff Quality (ARQ) (Engineers Australia, 2006) and other water quality guidelines so that urban stormwater management is an integrated part of the urban water cycle and avoids duplication of infrastructure. An integrated approach to stormwater management should avoid installation of infrastructure to meet separate objectives that, in combination, create unexpected diminished performance. There is a need to consider integrated approaches for future urban water management. Integrated systems have the capacity to produce solutions that respond to multiple objectives including economic, social and environmental criteria.

This Book on Runoff in Urban Areas is part of the evolving story of stormwater management and aims to encourage innovation into the future.

1.2. References

Engineers Australia (2006), Australian Runoff Quality - a guide to Water Sensitive Urban Design, Wong, T.H.F. (Editor-in-Chief), Barton.

Pilgrim, DH (ed) (1987) Australian Rainfall and Runoff - A Guide to Flood Estimation, Institution of Engineers, Australia, Barton, ACT, 1987.

Chapter 2. Aspects of Urban Hydrology

Anthony Ladson

With contributions from the Book 9 editors (Peter Coombes and Steve Roso)

Chapter Status	Final
Date last updated	14/5/2019

2.1. The Urban Hydrologic Cycle

Hydrologic analysis for both urban and non-urban situations begins with the water cycle. In rural areas, hydrologists are concerned with catchment inputs, especially rainfall, outputs such as evaporation and runoff, and water storage. The fundamental processes are the same for urban catchments, however, development profoundly changes water storages and flows (Figure 9.2.1):

- Inputs increase as mains water is supplied to urban catchments along with rainfall.
- Less water is stored within urban catchments. Paved surfaces replace much of the landscape to diminish infiltration of rainfall into soil profiles. Hydraulically efficient conveyance networks rapidly remove surface water from urban areas.
- There are dramatic changes in quantity, quality and timing of water leaving the catchment. Runoff volumes are often substantially increased. Wastewater networks provide an alternative flow path that interacts with stormwater and groundwater. There may be less opportunities for water to evaporate if it can quickly drain from a catchment.

The change in the rate and volume of inputs, outputs and storage explains the hydrologic behaviour in urban areas: the rapid response to rainfall and increased flood magnitude and frequency that correlates with development. This chapter explores aspects of urban hydrology, the impact of development and urban stormwater conveyance networks, focussing on areas where the effect of urbanisation needs to be considered for estimation of floods.

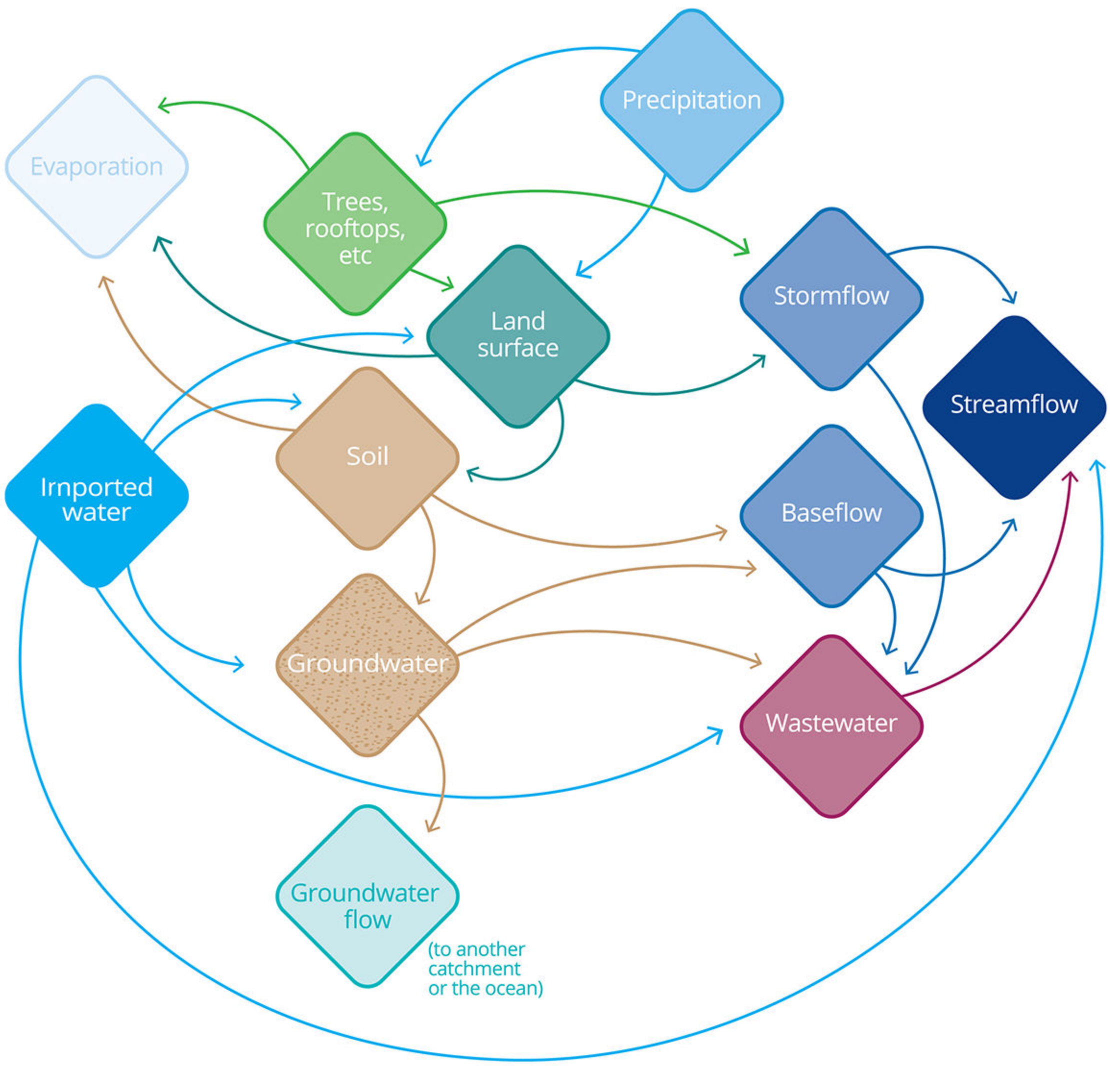

Figure 9.2.1. Simple Model of Water Inputs, Storage and Flows in an Urban Catchment

2.2. Human Impact on the Hydrologic Cycle

2.2.1. Urban Water Balance

The hydrological cycle must be considered at different temporal and spatial scales to gain an insight into urban hydrology. A water balance can identify the influence of imported water on catchment hydrology at the spatial scale of a suburb or city.

The water balance for an urban catchment, during a selected time period, can be expressed by equating the change in the amount of for water stored to the sum of catchment inputs minus the sum of catchment outputs (Mitchell et al., 2003).

$$\Delta S = (P + I) - (E_a + R_s + R_w) \quad (9.2.1)$$

Where:

ΔS is the change in catchment storage

P is precipitation

I is imported water

E_a is actual evapotranspiration

R_s is stormwater runoff

R_w is wastewater discharge

There have been several studies of water balances in the urban areas of Australia including Canberra, Melbourne, Perth, Sydney and South-East Queensland (Table 9.2.1). Although there are substantial differences in climate of these study areas, and the number of selected examples is small, the data provides some insights.

- Wastewater leaving a catchment should be less than 59% to 86% of the amount of water imported, since imported water contributes to stormwater and evapotranspiration. This means that imported water contributes to stormwater and/or evapotranspiration. As a result, stormwater plus evaporation exceeds precipitation, according to all case studies.
- Imported water is about 30% to 39% of precipitation. This means imported water substantially increases catchment inflows.
- The volume of imported water is about the same as, or less than, wastewater plus stormwater. This suggests the potential for augmentation of water supply by some combination of rainwater harvest, stormwater harvest and wastewater reuse.

Table 9.2.1. Annual Water Balance Data from Suburbs of Australian Cities.[a]

Location	Input			Output				Wastewater /Imported Water (%)
	Rainfall (mm)	Imported Water (mm)	Imported Water as a Percentage of Rainfall (%)	Actual Evapo-transpiration (mm)	Storm Water Runoff (mm)	Waste Water Runoff (mm)	Change In Store (Misclose) (mm)[b]	
Curtin, ACT (Mitchell, et al. 2003) (1979-1996)	630	200	32%	508	203	118	1	59%
Sydney (Bell, 1972) (1962-1971)	1150	349[c]	30%	736	501	262	0	75%
Sydney (Kenway et al., 2011) (2004-2005)	952	370	39%	766	281	319	-44	86%

Location	Input			Output				Wastewater /Imported Water (%)
	Rainfall (mm)	Imported Water (mm)	Imported Water as a Percentage of Rainfall (%)	Actual Evapo-transpiration (mm)	Storm Water Runoff (mm)	Waste Water Runoff (mm)	Change In Store (Misclose) (mm)[b]	
Subiaco-Shenton Park Perth (McFarlane, 1985)	788	285 + 96[d]	36%	766	104	154	117[e]	54%
Melbourne (Kenway et al., 2011) (2004-2005)	763	237	31%	688	165	190	-43	80%
South-East Queensland (Kenway et al., 2011) (2004-2005)	1021	374	37%	814	390	179	12	49%

[a]The National Water Accounts reported by the Bureau of Meteorology (Bureau of Meteorology, 2015) contain information on water use in regions that include the urban areas of Adelaide, Canberra, Melbourne, Perth, South East Queensland and Sydney.

[b]See original studies for details

[c]Includes imported water and use of groundwater

[d]Inflow of stormwater from upstream area

[e]Adjusted for change in groundwater storage

Assessment of water balances for cities or urban regions also need to account for the spatial and temporal variation of paramaters throughout an area. For example, the spatial distribution of rainfall depth, frequency (rain days per year) and maximum temperatures are shown for the Greater Melbourne region in Figure 9.2.2, Figure 9.2.3 and Figure 9.2.4.

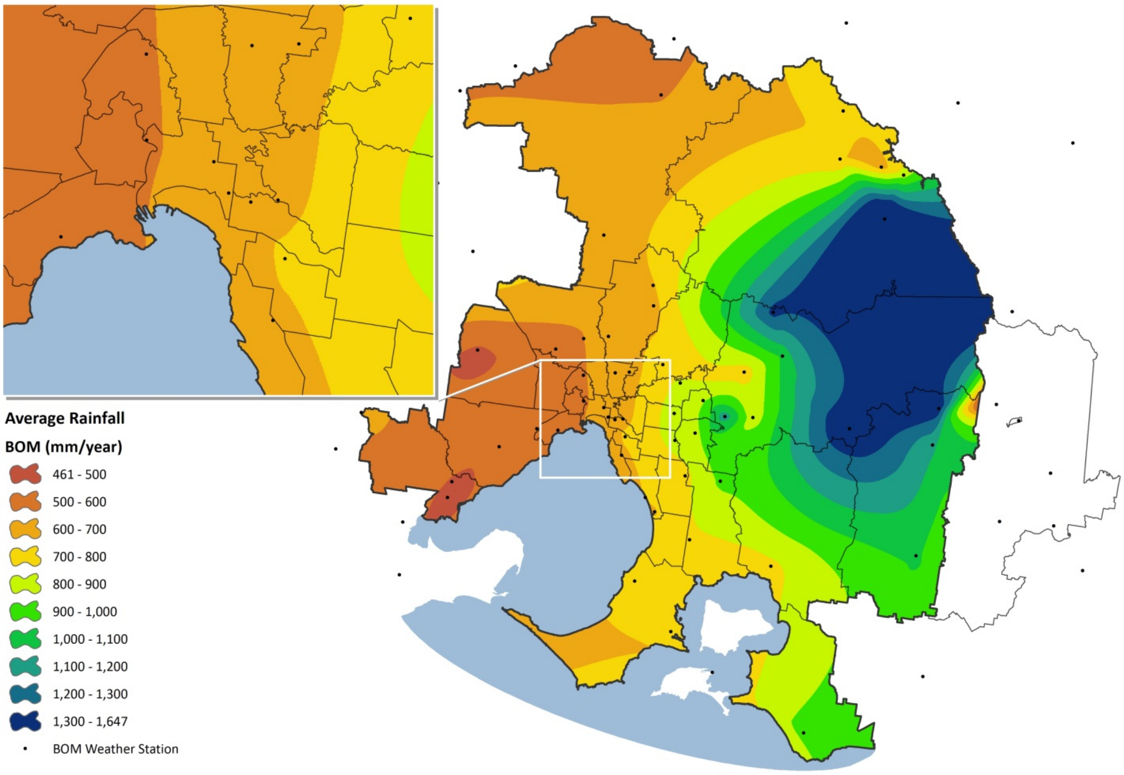

Figure 9.2.2. Spatial Distribution of Average Annual Rainfall Depths for the Greater Melbourne Region (Coombes, 2012)

Figure 9.2.2 demonstrates that average annual rainfall depths range from less than 470 mm to greater than 1640 mm across the Greater Melbourne region. The spatial distribution of rainfall will impact on the assessment of the water balance for the region and also impact on selection of stormwater management strategies. The spatial distribution of the frequency of rainfall will also impact on the determination of a water balance (Figure 9.2.3) (Walsh et al., 2012).

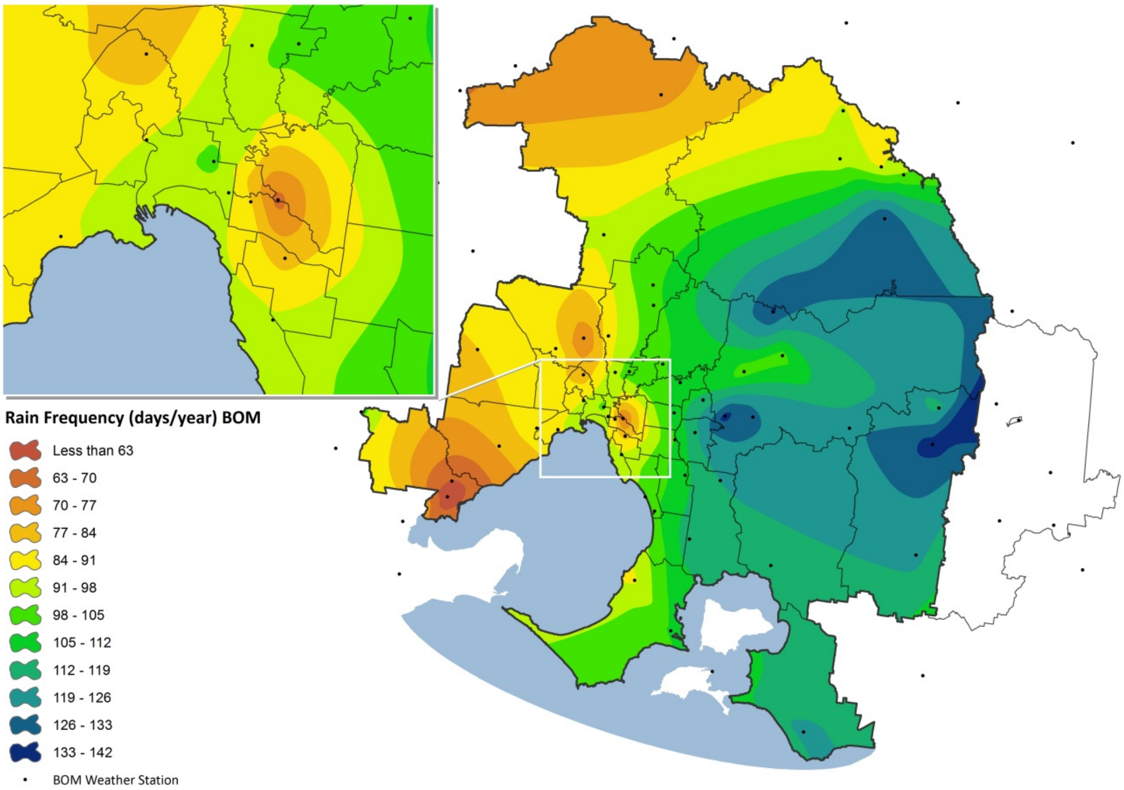

Figure 9.2.3. Spatial Distribution of Average Annual Frequency of Rainfall for the Greater Melbourne Region (Coombes, 2012)

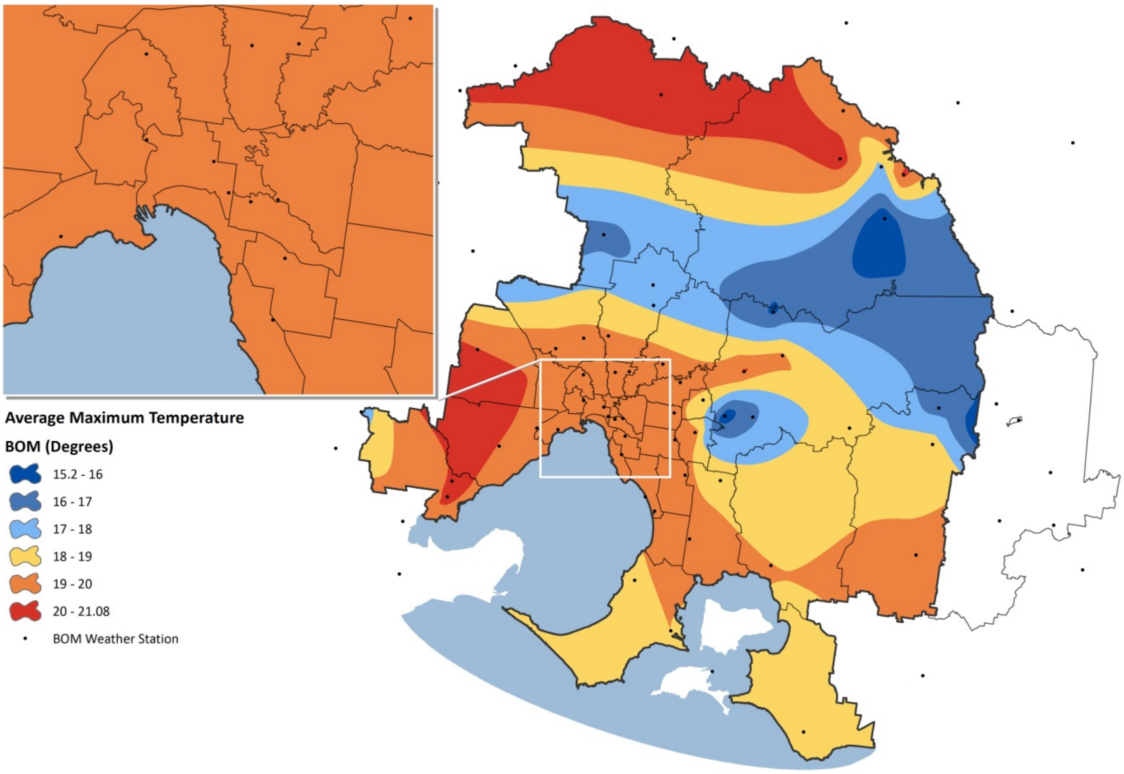

Figure 9.2.4. Spatial Distribution of Average Annual Maximum Temperatures for the Greater Melbourne Region (Coombes, 2012)

A range of recent detailed investigations that also considered the spatial and temporal variation of parameters was used to define water balances for Greater Melbourne, Greater Sydney, Greater Perth, and South-East Queensland regions. Water balances for 2013 were extracted from these studies to provide the examples presented in Table 9.2.2.

Table 9.2.2. Water Balances for Selected Regions

Region	Study	Average Annual Volume (GL)		
		Water	Wastewater	Stormwater
Greater Melbourne	Coombes & Bonacci (2012)	394	381	440[a]
Greater Sydney	Coombes & Barry (2012)	524	497	564
Greater Perth	Coombes & Lucas (2005)	249	131[b]	525
South East QLD	Coombes (2012)	278	265	470

[a]only includes stormwater runoff from urban surfaces. The total runoff volume of 650 GL/annum included open space and parks. These results are similar to the research by Walsh (2018) that estimated a total annual volume of 608 GL.

[b]there are less properties connected to centralised wastewater networks than connections to mains water supply.

Table 9.2.2 demonstrates that each region is subject to substantially greater volumes of stormwater runoff than demands for mains water. In addition, the volumes of wastewater discharges are similar to water demands. However, this result may be misleading as there are less wastewater connections (especially for Perth) than water supply connections in each region. Households in some areas are reliant on local wastewater management measures (such as septic tanks) and receive mains water supplies.

2.2.2. Lessons from a Detailed Water Balance Study at Curtin, ACT

Detailed information about an urban water balance is available for Curtin in ACT where Mitchell et al. (2003) obtained sufficient information to construct an annual water balance between January 1978 and June 1996. This study provides information on the variability in the urban water balance over time and the influence of climate (Table 9.2.3).

Table 9.2.3. Water Balance for Curtin Catchment in Canberra for the Period 1979 – 1995. (Adapted from (Mitchell et al., 2003))

Year	Rainfall (mm)	Imported Water (mm)	Actual Evapotranspiration (mm)	Stormwater Runoff (mm)	Wastewater Discharge (mm)	Change in Storage
Driest	247	269	347	74	107	-12
Average	630	200	508	203	118	1
Wettest	914	141	605	290	126	34

The average annual input and output of the catchment was about 830 mm. Approximately 24% (200 mm) of water was imported to the catchment via the supply system. Precipitation (rainfall) contributed the remaining 630 mm. Outputs included actual evapotranspiration (61%, 508 mm), stormwater runoff (24%, 203 mm) and wastewater discharge (14%, 118 mm).

The volume of imported water exceeded the volume of wastewater in all years and thus contributed to stormwater runoff, and at least in the driest years, to evapotranspiration. More water left the catchment as evapotranspiration and as stormwater runoff than was input via precipitation. In addition, in all but the driest years, wastewater and stormwater were greater than imported water, indicating the potential for harvest of suburban discharges to meet water demands. This highlighted the requirements for water imports under drought conditions.

Climate had a substantial influence on several of the water fluxes. Annual precipitation was highly variable ranging between 214 mm to 914 mm. On average, there was three times as much rainfall as water imports but in the driest year, more water was imported to the catchment than fell as rainfall. In the wettest year, imported water made up only 13% of water input. Figure 9.2.5 shows the relative amounts of precipitation and imported water for the driest, average and wettest years. The area of pie charts are proportional to total input. The proportion of imported water increases in drier years.. The proportion of imported water increases in drier years.

Considering outputs, the largest term is evapotranspiration, which represents 59% or more for each year. Although the total evapotranspiration varies between 347 mm and 605 mm for dry and wet years, the proportion of water lost as evapotranspiration is reasonably constant (59% to 66%). Figure 9.2.6 shows that relative amounts of actual evapotranspiration, stormwater and wastewater for the driest, average and wettest years (area of pie chart is proportional to total output). The proportion of stormwater increases in wetter years.. The proportion of stormwater increases during wetter years. The total volume and percentage of wastewater output does not seem to be greatly influenced by climate, as it is consistent between wet, average and dry years.

Stormwater runoff is highly reliant on climate, changing by a factor of about 4 mm from 74 mm during the driest year to 290 mm during the wettest year. Woolmington and Burgess (1983) demonstrated the direct link between garden watering and augmentation of low flows in Canberra urban streams, although this is moderated by water restrictions.

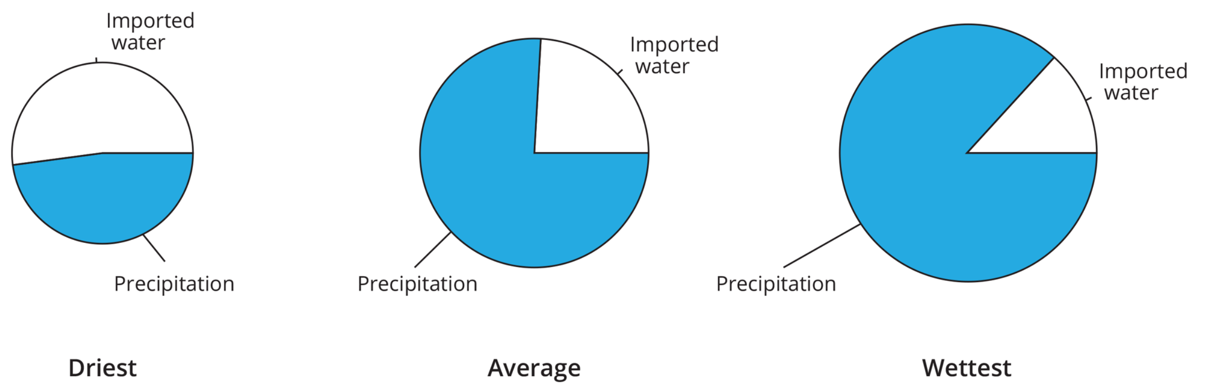

Figure 9.2.5. Total Water Input to Curtin in ACT

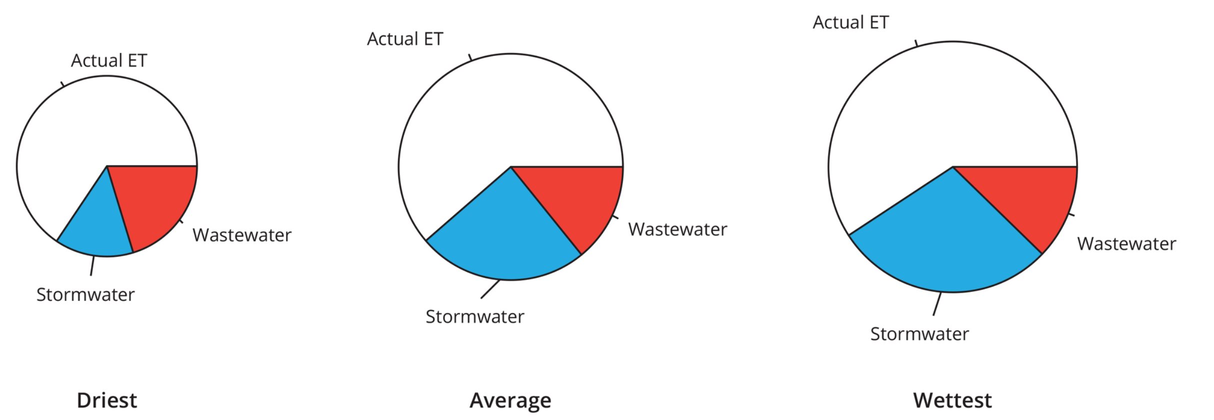

Figure 9.2.6. Total Water Output from Curtin in ACT

In summary, at the annual scale the urban water balance indicates the human impact on the hydrologic cycle. Water is imported into urban catchments and this exceeds the amount of wastewater exported, therefore there must be a net increase in outputs. Data from Curtin in the ACT shows that in dry years more than half of water inputs are via the mains supply system.

2.2.3. Implications of the Urban Water Balance: Stormwater as a Resource

Stormwater management throughout Australia was the subject of a recent Senate inquiry that recognised urban stormwater runoff as an under-utilised resource that creates significant environmental and flooding challenges (Commonwealth of Australia, 2015). Urban areas generate substantially greater stormwater runoff and pollutant loads compared to natural landscapes and are degrading our urban waterways and receiving waters. These additional flows substantially increase the discharges and overflows from sewer networks.

The volumes of stormwater runoff from urban areas exceed the water demand in many cities.

The water balance in cities include stormwater runoff, wastewater discharges and imported reticulated mains water. To illustrate this, Figure 9.2.7 presents the average annual water balance from the perspective of households in a range of Australian cities (Coombes, 2015).

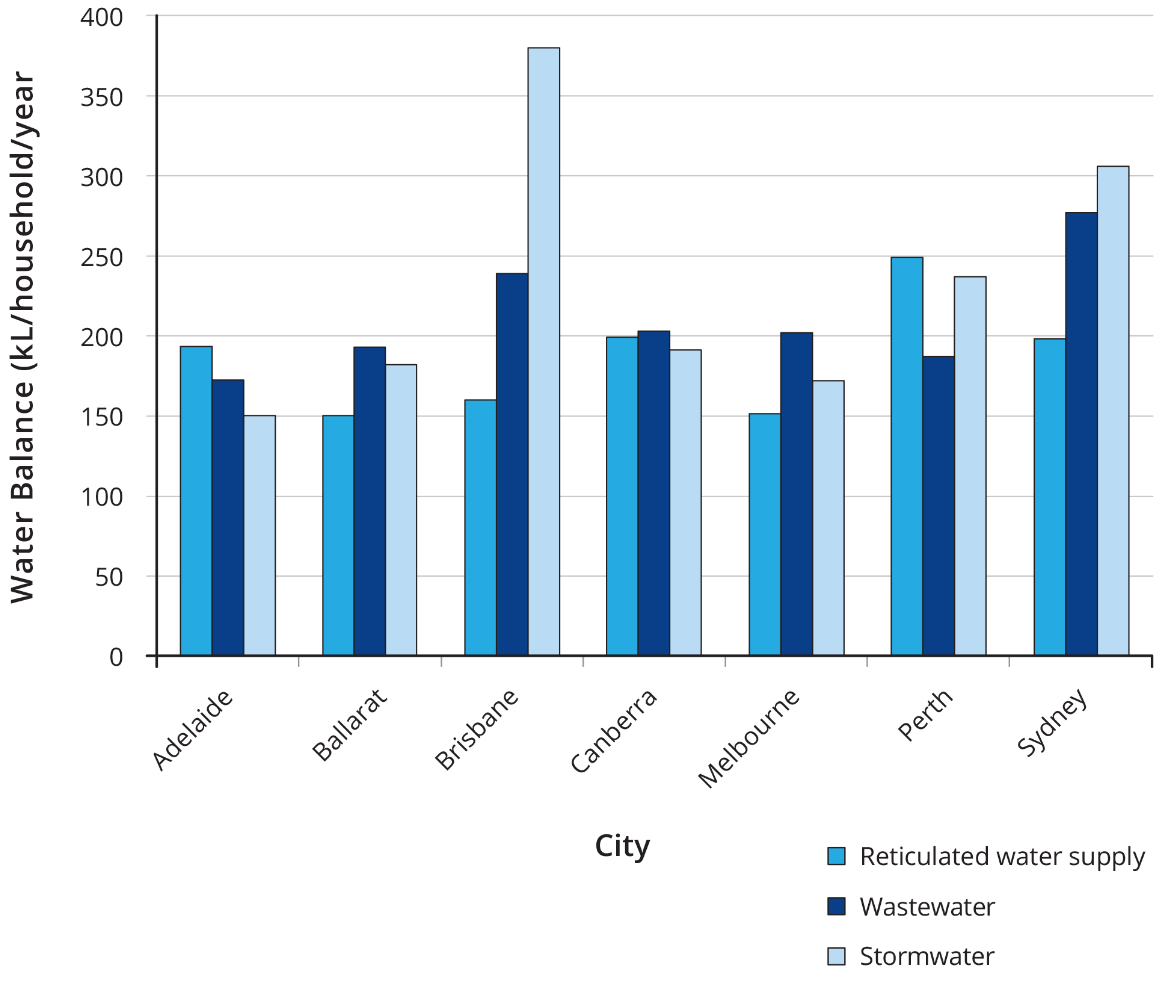

Figure 9.2.7. Average Annual Water Balances from Households in Adelaide, Ballarat, Brisbane, Canberra, Melbourne, Perth and Sydney

Figure 9.2.7 reveals how the combined volumes of stormwater runoff and wastewater discharging from households (and their properties) in each of the cities are greater than the volume of imported reticulated water supply at each location. Indeed, the average annual volumes of stormwater runoff from residential properties is similar to or greater than the average reticulated water demand from most of the properties. Improving stormwater management provides an opportunity to supplement urban water supplies as well as enhancing the amenity of urban areas and protecting the health of waterways in most cities.

The timing of water balances (rainfall, local and imported surface water supplies, groundwater, metered water use, sewage collected and stormwater runoff from urban surface) in the Ballarat Water District during recent drought is provided in Figure 9.2.8 as an example of water cycle processes (Coombes, 2015).

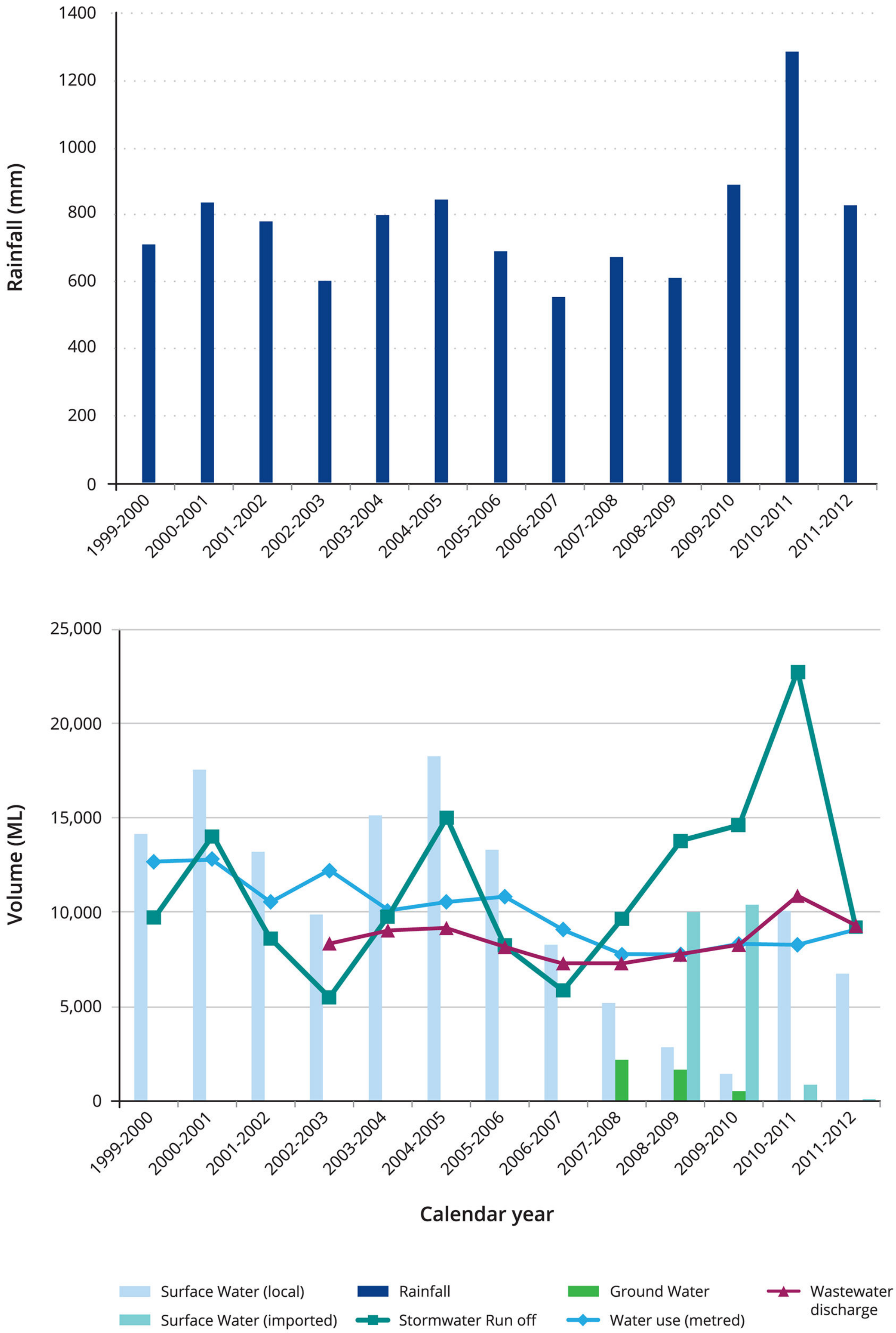

Figure 9.2.8. Water Cycle Processes in the Ballarat Water District from 1999 to 2012

Figure 9.2.8 indicates how the Ballarat Water District was dependent on surface water from nearby dams on local waterways [surface water (local)], until the worst of the drought in 2006. The reduced flows into local dams were supplemented using local ground water and the surface water imported from the Goulburn River (Murray-Darling Basin). Citizen's actions to reduce water use in response to water restrictions, installation of water efficient appliances and rainwater harvesting also halved the demands for utility water supply [water use (metred)] of the Ballarat Water District. The Council and the Water Authority also implemented stormwater harvesting and wastewater reuse solutions. In combination with the availability of ground water and imported surface water from the Goulburn River, these actions ensured that the City of Ballarat did not exhaust water supplies during drought. Despite rainwater and stormwater harvesting, there were still substantial stormwater runoff events suggesting that additional water was available albeit at additional cost.

The integrated action across the water cycle by the entire Ballarat community was a success from a water supply perspective that demonstrates the value of integrated solutions and understanding urban water balances. Nevertheless, this example also highlights the variable and temporal nature of urban water balances and connectivity with surrounding systems. Some of the key insights highlighted in Figure 9.2.8 are that substantial stormwater runoff events occur during drought, annual volumes of wastewater discharges were similar to water demands during water restrictions. Increases in stormwater runoff drive increases in wastewater discharges to be greater than water demands. The integrated solution for Ballarat was able to overcome the jurisdictional and institutional boundary conditions that limit opportunities for catchment based solutions in many cases.

2.2.4. Comparison of Rural and Urban Water Balances

A few studies that contrast water balances for urban and neighbouring natural catchments (Grimmond and Oke, 1986; Stephenson, 1994; Bhaskar and Welty, 2012). As expected, there is an increase in runoff, which we explore in the next section. The impact on evapotranspiration is less clear and depends on specific conditions as was apparent in the data for Curtain (Mitchell et al., 2003).

The partitioning of outflow between evaporation and stormwater runoff depends on water availability, conveyance infrastructure, storage in the catchment and the extent of irrigated parkland and gardens. There are a few examples, other than for Curtain, where this has been looked at in detail in an Australian context. In Melbourne, during a time of highly restricted water use for irrigation, Coutts et al. (2009) found that rapid stormwater runoff resulted in much reduced water availability and decreased evapotranspiration in urban areas compared to neighbouring rural sites. The result was a very dry urban landscape with energy partitioned into heating the atmosphere (which drove hot dry conditions) or into heat storage (which increased overnight temperature).

Bell (1972) suggests a similar decrease in evapotranspiration in Sydney (and consequent increase in runoff) as urbanisation increased. Recent investigations by Parker (2013) for Melbourne and by Argueso et al. (2014) for Sydney discuss a significant urban heat island effect, that is driven by increased heat storage capacity of urban structures and reduced evaporation from cities.

2.3. Aspects of Urban Stormwater Management Systems

2.3.1. Impervious Areas

An annual water balance illustrates the long term hydrologic changes caused by urbanisation. There are also substantial changes to flow events that are caused by:

- The expansion of impervious areas; and
- Efficient conveyance networks (Hollis, 1988; Schueler, 1994; Jacobson, 2011).

Urbanisation results in impervious surfaces replacing vegetated landscapes and this:

- Decreases the storage of water within soil profiles and on the ground surface and so increases the proportion of rain that runs off;
- Increases the velocity of overland flow; and
- Reduces the amount of rainfall that recharges groundwater.

Additionally, the natural stream network is augmented by conveyance networks (pipes and channels) that directly collects water from roofs and roads throughout the urban catchment. The expanded conveyance (drainage) network:

- Reduces the overland flow distance before water reaches a stream;
- Increases flow velocity because constructed drains are smoother and straighter than natural channels or overland flow paths;
- Reduces the storage of water in the channel system and on the catchment;
- Decreases the amount of water lost to evaporation because the water is quickly removed by the drainage network; and
- Means that almost all areas will contribute flow to a stream because the piped drainage network often extends to the furthest reaches the catchment.

As a result, although the exact effect of urbanisation on stream hydrology depends on the specific circumstances, there are some general comments that apply to many urban waterways in Australia. Urbanisation results in:

- Increased volumes of stormwater runoff;
- Increased frequency of high flow events;
- Increased magnitude of high flow events;
- Increased rates of change (both rising and falling limb of hydrographs);
- Increased catchment responsiveness to rainfall – more runoff events;
- Increased speed of catchment response;
- Reduced seasonality of high flows – high flow events occur year round rather than being mainly concentrated in a wet season;
- Greater variation in daily flows;
- Increased frequency of surface runoff to streams; and
- Reduced infiltration of rainfall.

Hydrologic changes caused by urbanisation occur at the same time as, and partly cause, changes to sediment loads, stream ecology and water quality (Walsh et al, 2005). Key hydrologic changes are considered in more detail in the following sections.

Increased Flow Volumes

More rainfall is converted to runoff in urban catchments from impervious surfaces and from pervious areas that are commonly compacted or irrigated by imported water (Harris and Rantz, 1964; Cordery, 1976; Hollis and Ovenden, 1988a; Hollis and Ovenden, 1988b; Hollis, 1988; Ferguson and Suckling, 1990; Boyd et al., 1994; Walsh et al., 2012; Askarizadeh et al., 2015).

Increased Flood Frequency and Magnitude

The increase in magnitude of flooding because of urbanisation has been recognised for many decades (Leopold, 1968). Urbanisation causes up to a 10-fold increase in peak flood flows in the range 4 EY to 1 EY with diminishing impacts on larger floods (Tholin and Keifer, 1959; ASCE, 1975; Espey and Winslow, 1974; Hollis, 1975; Cordery, 1976; Packman, 1981; Mein and Goyen, 1988; Ferguson and Suckling, 1990; Wong et al, 2000; Beighley and Moglen, 2002; Brath et al., 2006; Prosdocimi et al., 2015).

Increased flood magnitudes have been confirmed by analysis of paired catchment data in Australia as demonstrated by the comparison of urban Giralang and rural Gungahlin catchments in Canberra (Codner et al., 1988) as well as numerous modelling studies (Carroll, 1995). The impact of this increased flooding is substantial and makes up a large proportion of overall average annual flood damage estimates (Ronan, 2009).

Faster Flood Peaks – Flashiness

Runoff in urban streams responds more rapidly to rainfall in comparison to rural catchments and recedes more quickly. The quick response means there are more flow peaks in urban streams (Mein and Goyen, 1988; McMahon et al., 2003; Baker et al., 2004; Heejun, 2007; Walsh et al., 2012). Urbanisation was found to reduce the volume of channel storage by a factor of 30 in Canberra (Codner et al., 1988). This contributes to the rapid response of urban streams and increased flood flows.

The lag time – the time between the centre of mass of effective rainfall and the centre of mass of a flood hydrograph – decreases by 1.5 to 10 times in response to urbanisation (Packman, 1981; Bufill and Boyd, 1989).

Increased Runoff Frequency

Increased frequency of stormwater runoff is correlated with increased area of impervious surfaces. Small rainfall events of 1 to 2 mm will cause runoff from impervious surfaces (ASCE, 1975; Codner et al., 1988; Boyd et al., 1993; Walsh et al., 2012) but much more rainfall is usually required to produce runoff from grassland or forest (Hill et al., 1998; Hill et al., 2014). The frequency of stormwater runoff can increase by a factor of ten or more.

The increased responsiveness of urban landscapes to rainfall means that seasonality of flows in urban streams is different to rural streams. In many areas, rural catchments will only produce runoff after saturation of soil profiles following long periods where rainfall exceeds evapotranspiration. This result produces seasonal stream flows in many rural catchments with little runoff, when catchments are dry even when there is heavy rainfall (Western and Grayson, 2000). In urban streams, flows occur anytime there is rainfall. In temperate urban catchments, the largest urban runoff often occurs following intense thunderstorm rain during summer when, in equivalent rural catchment, there is little flow (Codner et al., 1988; Smith et al., 2013).

Changed Base Flows

The influence of urbanisation has complex impacts on groundwater and base flow in streams. Various features of urbanisation have confounding effects and their relative magnitude will determine the overall influence on base flow in streams. These features include:

- Reduced vegetation cover;
- Increases in impervious surfaces that limits infiltration and reduces evaporation of shallow groundwater;
- Infiltration from irrigation of gardens;
- Water leaking from pipes which contributes to ground water; and
- Drainage of groundwater into pipes or the gravel-filled trenches that surround pipes.

The most common response to urbanisation is that base flow in urban streams is decreased. More impervious areas means less opportunity for water to infiltrate so groundwater storage, for storage in soil profiles and discharges are reduced (Simmons and Reynolds, 1982; Lerner, 2002; Brandes et al., 2005). Less commonly, there may be increased base flow, particularly where stormwater is deliberately infiltrated (Ku et al., 1992; Al-Rashed and Sherif, 2001; Barron et al., 2013).

2.3.2. Conveyance

Urbanisation changes the processes of conveying water. The network of urban stormwater conveyance infrastructure is denser and more extensive than the natural stream system it replaces. This means that water is conveyed rapidly from both pervious and impervious surfaces throughout an urban catchment. Resistance to flows is lower in straight and smooth drainage paths of urban waterways, as compared to their natural counterparts.

The way water is conveyed from impervious areas can enhance or mitigate the influence of impervious areas. Modelling by Wong et al (2000) suggests that condition of the waterways also influences peak discharges that follow urbanisation. The largest impacts occur when urban streams are lined and made hydraulically efficient.

The importance of stormwater conveyance was confirmed in catchments with similar imperviousness but with and without conventional drainage infrastructure. This alteration of hydraulic behaviour was substantially reduced in suburbs with less efficient informal stormwater infrastructure that included roofs drained to gardens or rainwater tanks, and sealed roads which lacked curbs and drained to surrounding forest or earthen or vegetated swales (Hardy et al., 2004; Walsh et al, 2005).

Conveyance of Flood Flows

Understanding the conveyance of water in urban areas during times of overland flooding is a critical part of the analysis and design of urban stormwater management strategies. The major/minor principle requires that overland flow paths must be considered once the capacity of conveyance conduits is exceeded. This behaviour can be complex. Modelling of overland flow paths is used in many areas to guide zoning of land to control development and so reduce flood risk (Baker et al., 2005).

The catchment boundary for overland flows will often differ from boundaries of flows in conduits. This means that the behaviour of large floods may be substantially different from smaller events and has the potential to produce unexpected behaviours. An example is a suburb protected from riverine flooding by a levee. Stormwater is usually discharged under the levee into the river. If overland stormwater flooding cannot reach the river because of the levee it may, instead, back up and cause flooding. This type of unexpected and rapid flooding can be dangerous, as people are unlikely to be prepared for these types of events.

2.3.3. Receiving Environments

Many urban areas are adjacent to estuaries or bays that are the downstream boundary for water levels in streams. Coincident stormwater and estuarine flooding needs to be considered and is addressed in detail in Book 6, Chapter 5. Water authorities will often have mandated sea levels that must be used as part of the analysis flooding scenarios for planning (e.g; Melbourne Water (2012)).

Major rivers flowing through urban centres are also receivers of urban stormwater. These rivers will determine the base level to be used for modelling and additional analysis of the river system may be required to ensure flood risks are adequately considered.

The impact of urbanisation on major rivers can be contrasted with the effect on urban stormwater conveyance systems. Much of the water that is used in cities is harvested from the rivers that flow through them, for example, the Yarra River in Melbourne, the Hawkesbury-Nepean in Sydney and the Brisbane River. This results in lower flows and reduces flooding in main streams. There is a paradox here. The main rivers in urban areas have much reduced flow while in urban waterways flows are increased. For example, in Melbourne, there is about 125 km of streams and estuaries where flow has been substantially decreased by harvesting for urban water supply, and 1700 km of urban streams with substantially increased flow from urban catchments. From a citywide perspective, stormwater management needs to consider both of these impacts.

2.3.4. More Complex than Rural

Many aspects of urban flooding are more complex than similar issues in rural areas and require careful and thorough analysis. Key differences include:

- Very rapid response to rainfall;
- A greater proportion of rainfall converted to flood flow;
- Large numbers of people potentially affected by flooding;
- Development in one area adversely affecting flood risk in distant areas;
- Catchment areas than can change with the magnitude of flooding;
- Increased influence of the spatial pattern of rainfall because catchments respond to short rainfall events which are more spatial variable;
- Flooding from both riverine (fluvial) and stormwater (pluvial) overflows; and

- Floods can occur at any time of the year and may be most severe when triggered by summer thunderstorms - there is often no requirement for antecedent rainfall to wet the catchment to generate flooding.

In reviewing the components of average annual flood damage, Ronan (2009) suggested that, in general, risks from riverine flooding were reasonably well addressed but that stormwater flooding was a major issue that was yet to be adequately considered.

2.3.5. Combined and Separate Systems

The discussion in this section has generally assumed that suburbs have separate sanitary sewers and stormwater management systems. This is mostly true for Australian towns and cities. However, two areas have combined sewers – a single pipe that carries both wastewater and stormwater. These are the central area of Launceston, Tasmania and a small area in the CBD of Sydney. When the first sewers were built in Sydney, around 1857, there were five combined sewer systems: Woolloomooloo, Blackwattle Bay, Hay Street, Tank Stream and Bennelong. These discharged to Sydney Harbour. Most of these original sewers were converted to carry stormwater only following the construction of the Bondi Ocean Outfall Sewer in 1889 and wastewater was discharged in the ocean. Later developments in Sydney and elsewhere adopted separate stormwater drainage.

For an analysis of decision-making between separate and combined systems of sewerage, see Tarr (1979). For a history of urban drainage approaches, see Delleur (2003).

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Chapter 3. Philosophy of Urban Stormwater Management

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3.1. Introduction

Urban stormwater management is historically described as the hydraulic design of urban drainage networks that safely conveys stormwater runoff to receiving environments. The industry's approach to urban water management in Australia has changed significantly since the establishment of centralised and separate water supply, stormwater and wastewater paradigm in the 1800s.

Urban water management evolved over time to include waterways protection, mitigation of stormwater quality, use of Water Sensitive Urban Design (WSUD), Integrated Water Cycle Management (IWCM), Water Sensitive Cities (WSC), Integrated Water Management (IWM), and many other approaches. Although these approaches are relatively new, they have wide adoption and support in legislation and policies for water management throughout Australia. Similar changes in approach to urban stormwater management in other countries include Sustainable Urban Drainage Systems (SuDS) (Bozovic et al., 2017) and Low Impact Design (LID) (USEPA, 2008). Consequently, the approach to urban stormwater management includes water supply and is based on retention and conveyance of stormwater runoff to meet multi-purpose design objectives that enhance livability of urban areas, mitigate nuisance, and avoid damage to property and loss of life.

3.2. The Journey from 1987 to 2016

Australia has experienced considerable improvement in urban water management since the 1800s, supported and underpinned by publications such as ARR (PMSEIC, 2007). Stormwater drainage in Australia evolved from combined sewers that rapidly discharged the accumulated rubbish, sewage, sullage and stormwater from streets to waterways (Armstrong, 1967; Lloyd et al., 1992). The impact on waterways and amenity of urban settlements drove the separation of sewage and stormwater infrastructure. Filling of swamps and development of contributing catchments to accommodate population growth resulted in frequent flooding of early settlements. Drainage solutions emerged to avoid stagnant water, local flooding and health impacts in urban areas. Nation building works programs during economic depressions (for example in 1890 and 1920) and following wars provided large scale drainage infrastructure throughout Australia.

The ARR 1987 guideline focused on collection and conveyance of peak stormwater flows in drainage networks. The guideline's advice on hydrologic and hydraulic analysis was consistent with the emerging computer age and hand calculation while programmable calculator and computer methods were discussed. The increasing complexity of the different methods and an associated requirement for use of computers was highlighted.

Use of statistical design rainfall bursts was recommended to calculate inflows to drainage networks and the Rational Method was described as the best known method for estimation of urban stormwater runoff. The main objective of urban drainage was to convey stormwater from streets and adjoining properties without nuisance from minor rain events, and to avoid property flooding and associated damage from major rain events (the minor/major design approach).

In contrast to the introductory comments, urban drainage was presented as a prescriptive approach using pipes to convey minor flows, with streets, open space and trunk drains used to transport major flows. Trunk drainage was described to include designs for open channels, detention and retention basins to control peak discharge, and bridges. While urban stormwater management was presented and interpreted as a drainage approach, Chapter 14 in ARR 1987 highlighted how urban drainage solutions should also:

- limit pollutants entering receiving waters;
- consider water conservation;
- integrate overall planning schemes;
- be based on measured or observed real system behaviour;
- be viewed in relation to the total urban system; and
- maximise benefits to society.

Drainage solutions solely focused on developed catchment and were mostly designed by engineers. The simplicity of methods for estimating stormwater runoff implied accuracy and certainty of design performance for many users. Urban water management further evolved in the mid-1990s to cover protection of waterways, mitigation of urban stormwater quality, WSUD (Whelans and Maunsell, 1994), IWM and IWCM (Coombes and Kuczera, 2002) approaches. Nevertheless urban stormwater runoff creates complex impacts on urban stream ecosystems and receiving waterways (Walsh et al., 2005; Paul and Meyer, 2001). Increases in runoff volumes and rates from urban areas (flow regimes) contribute to degradation of riparian ecosystems and promotes geomorphic changes within urban streams (Walsh et al., 2012). Although these approaches are relatively new, they have subsequently gained widespread adoption and support throughout Australia. To support this evolution, Engineers Australia published 'Australian Runoff Quality – A Guide To Water Sensitive Urban Design' in 2006 (Engineers Australia, 2006).

The acceptance of WSUD, IWCM and related approaches is manifested in three significant ways:

- the development of benchmark projects (e.g; Lynbrook estate (Lloyd et al, 2002), Fig Tree Place (Coombes et al., 2000) and Little Stringy Bark Creek (Walsh et al., 2015)) that provided evidence that these new approaches were successful;
- the creation of local policies and plans for integrated water management; and
- the adoption of policies for sustainable water management by state and federal governments.

Recent droughts, such as the 'millennium drought' also triggered many other changes in the urban water sector, largely associated with water conservation, harvesting, recycling and reuse (Aishett and Steinhauser, 2011).

Urban areas are complex systems that are subject to dynamic interaction of economic, social, physical and environmental processes across time and space (Forrester, 1969;

Coombes and Kuczera, 2003; Beven and Alcock, 2012). Continuous intervention is required to renew urban economic, technical and social structures to maintain human welfare and protect ecosystem services (Forrester, 1969; Meadows, 1999). Understanding these processes into the future also encounters the uncertainty created by non-stationary data that describes past processes. Design and analysis processes should include distributed approaches to account for the time based dynamics of essential data. The integrated nature of contemporary water management approaches is different to the objectives and design solutions envisaged in 1987. Urban water management is now required to consider multiple objectives (e.g. resilience, livability, sustainability and affordability) and the perspective of many disciplines. Advances in computing power, more available data and associated research also allows the analysis of increasingly complex systems to understand the trade-offs between multiple objectives (Coombes and Barry, 2014). Design of urban water management seeks to integrate land and water planning. Use of more comprehensive datasets revealed a greater range of potential outcomes that needs understanding to develop integrated solutions.

According to Argue (2017), the urban designer aims at managing the impact of urban stormwater runoff 'at source' and at multiple scales by retaining stormwater in landscapes and soil profiles, rainwater harvesting and disconnecting impervious surfaces from drainage networks (Poelsma et al., 2013). Consistent with the philosophy of source control and systems analysis, stormwater runoff is now seen as an opportunity and is valued as a resource (Clarke, 1990; Mitchell et al., 2003; McAlister et al., 2004). Modern design criteria may include analysis of the volumes, timing and frequency of stormwater runoff to determine peak flow rates, water quality and requirements to mimic natural flow regimes to protect waterway health (Walsh, 2004).

3.3. Evolving Opportunities and Challenges

Urbanisation generates dramatic changes within the natural water cycle. Impervious surfaces and directly connected drainage infrastructure decreases evapotranspiration and infiltration to soil profiles. This increases the volume and frequency of stormwater runoff and reduces baseflows; which can create flooding and affect waterway health. Drainage strategies that are reliant on conveyance can transfer additional stormwater runoff and pollutant loads generated by urban areas to other locations. The different regional scale responses within a river basin and a linked urban catchment are presented in Figure 9.3.1.

The impervious surfaces and hydraulically efficient infrastructure associated with urban catchments increases the magnitude and frequency of stormwater runoff whilse reducing the infiltration to soil profiles and subsequent baseflows in waterways. The accumulation of stormwater flows within urban catchments is highlighted. The first response at A is the (undisturbed) ecosystem upstream from urban impacts, the second response at B includes the impact of water extraction to supply the urban area (changed flow regime in rivers created by water supply) and the third response at C includes water discharges from the urban catchment (changed flow and water quality regime from both stormwater runoff and wastewater discharges) into the river basin.

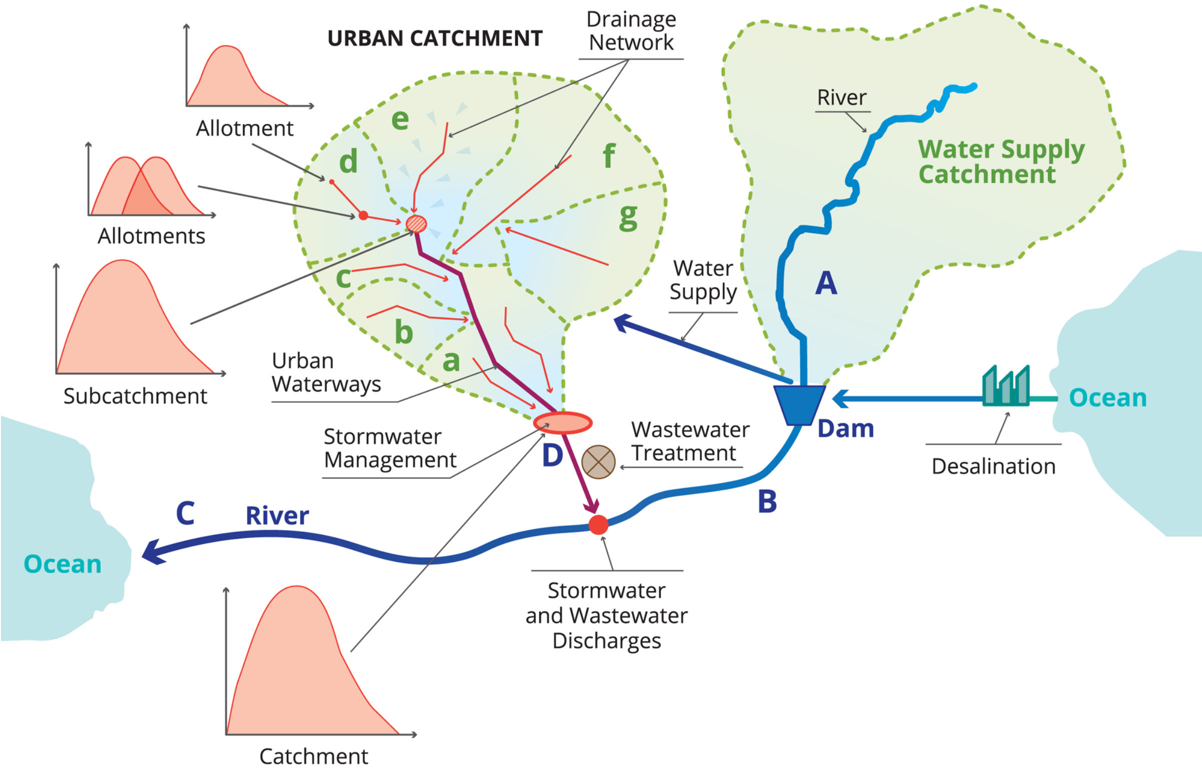

Figure 9.3.1. Schematic of Traditional Urban Catchments and Cumulative Stormwater Runoff Processes

Figure 9.3.1 demonstrates analysis and solutions at point D at the bottom of urban catchments; it can exclude understanding of impacts within the urban catchment (sub-catchments a-h) and external impacts to the river basin at B and C. Traditional analysis of urban catchments is from the perspective of rapid discharge and accumulation of stormwater via drainage networks (in sub-catchments a-h) with flow and water quality management at the bottom of the urban catchment (D) using retarding basins, constructed wetlands, and stormwater harvesting. However, the benefits for flood protection, improved stormwater quality, and protection of the health of waterways from this approach do not occur within the urban catchment upstream of point D.

Figure 9.3.1 also highlights how distributed land uses (allotments or properties) produce hydrographs of stormwater runoff into the street drainage system. This system accumulates stormwater runoff from multiple inputs, creating progressively larger volumes of stormwater runoff, which ultimately flows into urban waterways or adjoining catchments (Pezzaniti et al., 2002). This process results in significant changes in volume and timing of stormwater discharge to downstream environments.

There has been an emerging understanding that this issue can be solved by viewing urban stormwater as an opportunity to supplement urban water supplies and enhance the amenity of urban areas (Mitchell et al., 2003; Barry and Coombes, 2006; Wong, 2006). This includes development of green infrastructure and microclimates that reduce urban heat island effects. Urban catchments with impervious surfaces are substantially more efficient than conventional water supply catchments in translating rainfall into surface runoff. Rainwater and stormwater harvesting can extend supplies from regional reservoirs and the restoration of environmental flows in rivers subject to extractions for water supply (Coombes, 2007). These insights are consistent with earlier applied research by Goyen (1981) that both volumes and peak flows of stormwater runoff are required to design stormwater infrastructure, and the local property scale is the building block of cumulative rainfall runoff processes (Goyen, 2000). Reducing urban stormwater runoff volumes via harvesting and retention in upstream catchments can also decrease stormwater driven peak discharges and surcharges in wastewater infrastructure (Coombes and Barry, 2014).There has been an emerging understanding that this issue can be solved

Changes in land use, climate, increased density of urban areas and decline in hydraulic capacity of aging drainage networks can result in local flooding and damage to property. Climate change is expected to reduce annual rainfall and generate more intense rainfall events in a warming climate (PMSEIC, 2007; Wasko and Sharma, 2015). This will intensify the challenges of providing secure water supplies and mitigating urban stormwater runoff. There may also be the need to replace stormwater conveyance networks installed during post-war urban redevelopment that are nearing the end of useful life. In this situation, the capacity of an aging network or increased runoff from increasing development density can be supplemented by source control measures and integrated solutions (Barton et al., 2007). Integrated solutions and flexible approaches to design can avoid costly replacement of existing infrastructure.

Flood management issues for many urban areas are driven by runoff discharged towards waterways (overland flooding) rather than from flood flows originating at waterways (fluvial flooding). There is a need to consider more extensive range of stormwater runoff events, from frequent to rare or extreme and the associated impacts on urban environments (Weinmann, 2007). Management of these flood related impacts require integrated management of the full spectrum of flood events (Figure 9.3.2).

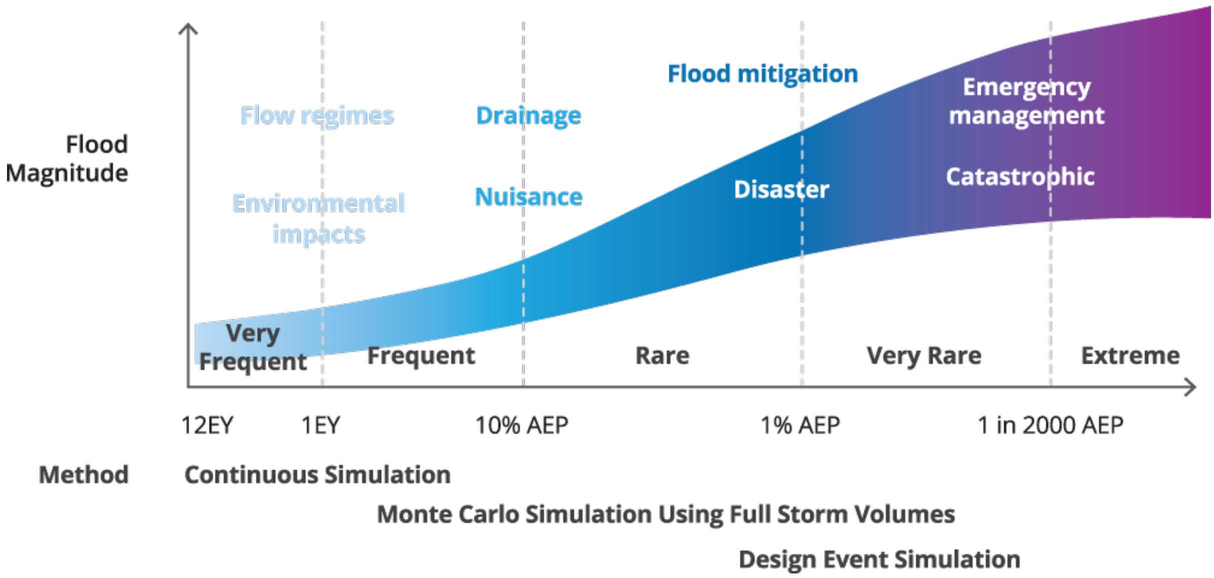

Figure 9.3.2. The Full Spectrum of Flood Events (Adapted from Weinmann (2007))

Figure 9.3.2 highlights the evolving methods of analysis, including continuous simulation and Monte Carlo simulation of full storm volumes that are likely to be required to account for the full spectrum of rainfall events as defined by Exceedance per Year (EY) or Annual Exceedance Probability (AEP). The definition of rain events is currently a mix of assumptions regarding frequency and magnitude that is clarified in this version of ARR to allow effective advice on design of stormwater management schemes. This includes development of green infrastructure and microclimates with reduction of urban heat island effects.

Strategic use of water efficiency, rainwater, stormwater and wastewater at multiple scales can supplement the performance of centralised water supply systems to provide more sustainable and affordable outcomes (Victorian Government, 2013). These integrated strategies diminish the requirement to transport water, stormwater and wastewater across regions with associated reductions in costs of extension, renewal and operation of infrastructure (Coombes and Barry, 2014). This leads to decreased requirement to augment regional water supplies and long run economic benefits. These strategies also focus on restoring more natural flow regimes in waterways and they will be beneficial in reducing remedial works in waterways and will provide reduction in size or footprint of quality treatment measures (Poelsma et al., 2013).

Current approaches to stormwater management include separate design processes and infrastructure for flooding, drainage and water quality. Jurisdictional and institutional boundary conditions are often imposed on analysis (Brown and Farrelly, 2007; Daniell et al., 2014). Integrated design includes solutions that meet multiple objectives, the catchment boundaries of each element and aims to avoid redundant infrastructure. Realisation of these benefits is dependent on integrated design approaches that account for changes in the timing and volumes of stormwater runoff, and respond to multiple objectives. Analysis of the economic benefits of integrated designs and drainage networks should be evaluated across an entire system from the perspective of whole of society. The methods and objectives for estimating urban stormwater runoff and the design of pipe drainage networks from 1987 do not include these additional considerations.

A challenge to integrated solutions is presented by engineering and economic methods of estimating performance that are reliant on average assumptions and judgements as inputs to empirical methods of estimating performance. Consequently, optimum design based on average assumptions and model approximations may not represent the actual integrated response of a project.

Educated empirical input assumptions and estimation processes can be reasonably approximated as generic processes for known historical and static problems (Kuczera et al., 2006; Weinmann, 2007). However, these processes may not replicate performance of multiple solutions within a system. For example, with respect to intersection of local water cycle solutions with town planning processes and regional infrastructure and, therefore, cannot understand or value a system that changes runoff behaviour from the smallest distributed scales (from the ‘bottom up’) (Argue, 2017; Coombes and Barry, 2014; Goyen, 2000). For example, cumulative actions at the smallest scale, such as retaining stormwater in the soil profile on each property can produce significant changes in responses throughout urban systems as shown in Figure 9.3.3.

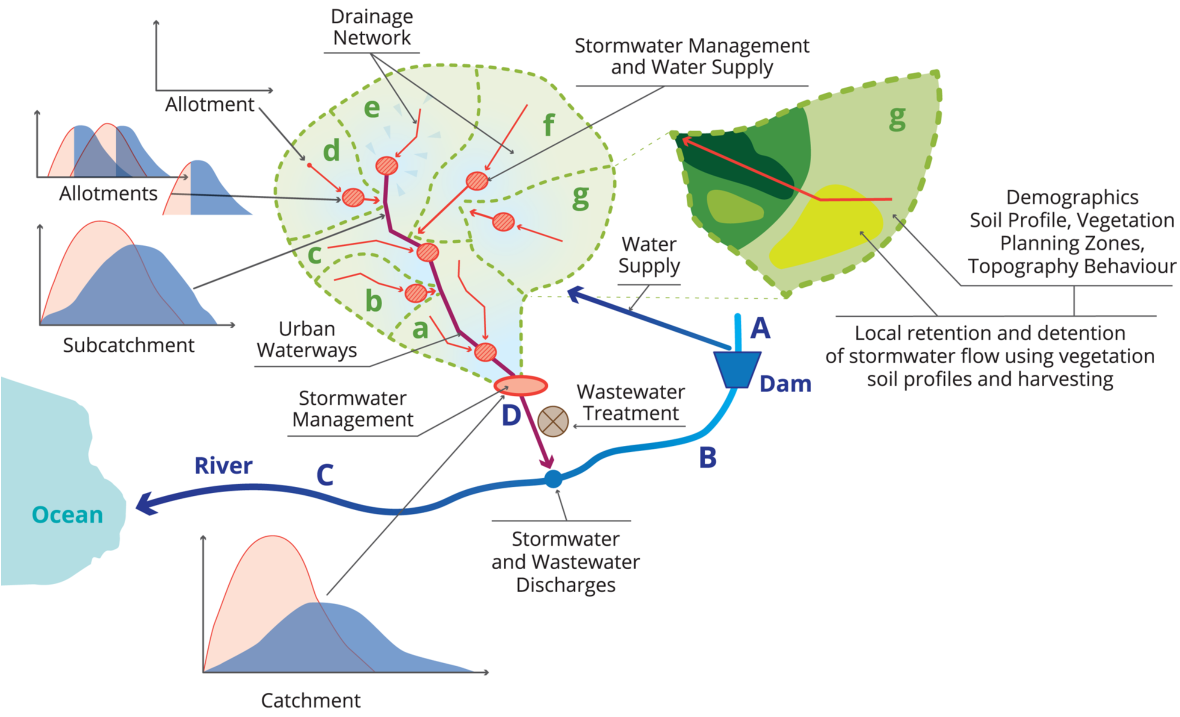

Figure 9.3.3. Cumulative Impacts of Distributed Management

It also follows that historical 'top down' design processes may not evaluate distributed processes because a small proportion of the available data may be simplified as whole of system average or fixed inputs (such as a runoff coefficient and average rainfall intensity). Thus, the signals of linked distributed performance (such as local volume management measures) in a system are smoothed or completely lost by partial use of data as averages and by the scale of analysis. Therefore, there is no direct mechanism to capture cascading changes in behaviours throughout a system. This can lead to competing objectives (For example: local versus regional), inappropriate solutions and information disparity such as provision of a wetland and retarding or detention basin downstream of an urban area when management is required within the urban area to protect urban amenity, stream health and avoid local flooding. This paradox can only be resolved through a broader analysis framework which recognises location based principles of proportionality and efficient intervention.

For example, consider the connectivity of contemporary water cycle networks presented in Figure 9.3.4.

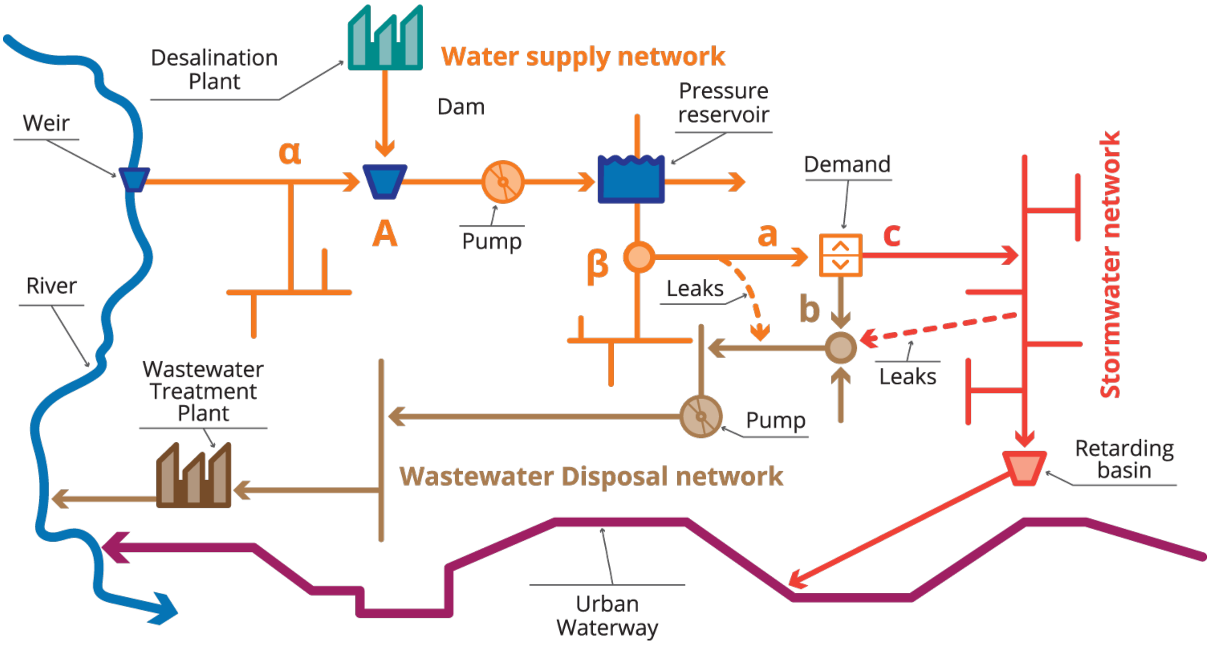

Figure 9.3.4. Schematic of the Connectivity of Urban Water Networks

Figure 9.3.4 shows that an input, or extraction at any point α or β, or an increase in water storage in a reservoir, at location A, will have some influence on flows and capacities at many other points in the system. These in turn, will translate into changes in performance and costs across the linked networks of infrastructure. Similarly, changes in behaviour (demand) at any point in the system will generate different linked impacts a, b and c on water, wastewater and stormwater networks respectively. Analysis and design of integrated solutions needs to account for the linked dynamic nature of the urban water cycle and demography. The inclusion of rainwater and stormwater harvesting, and wastewater reuse further increases the level of connectivity of urban water networks.

The historical practice for estimation of stormwater runoff rates and the design of drainage (conveyance) infrastructure is based on a methodology where all inputs, other than rainfall, are fixed variables. The fixed values of the input variables are selected to ensure that the exceedance probability of stormwater runoff is similar to that of regional rainfall statistics. However, catchments that contain cascading integrated solutions involving retention, slow drainage, harvesting of stormwater and disconnection of impervious surfaces require enhanced design methods (Kuczera et al., 2006; Wong et al, 2008; Coombes and Barry, 2008). These emerging methods for analysis and design of integrated solutions include the following considerations:

- Long sequences of rainfall that include full volumes of storm events are required to generate probabilistic designs of integrated solutions;
- Peak rainfall events may not generate peak stormwater runoff from projects with integrated solutions;
- The frequency of peak rainfall may not be equal to the frequency of peak stormwater runoff from integrated solutions;
- Stormwater runoff from urban catchments is influenced by land use planning, and the connectivity and sequencing of integrated solutions across scales;

- The probability distribution of the parameters that influence the performance of the integrated solutions (for example human behaviour, rainfall and soil processes) and the ultimate stormwater runoff behaviour are unknown for each project;

- Integrated solutions often meet multiple objectives (for example water supply, stormwater drainage, management of stormwater quality, provision of amenity and protection of waterways) and are dependent on linked interactions with surrounding infrastructure; and

- We should be mindful that the limitations of design processes are not always apparent and diligence is required to ensure that substantial problems are avoided.

In this situation, continuous simulation using historical or synthetic sequences of rainfall in a Monte Carlo framework may be required to understand the probability of stormwater runoff and the design of infrastructure (Kuczera et al., 2006; Weinmann, 2007). There are approximately 20,000 daily rainfall records with sufficient continuous rainfall records (more than 3,500) to allow continuous simulation using real or synthetic continuous rainfall records. Similarly, the designer can use ensembles of full volumes design storm event to test an integrated design solution. Assumptions and methods of analysis imposed by approval authorities in accordance with ARR 1987 can constrain the use of more appropriate analysis techniques required for better understanding the behaviour of integrated solutions. Similarly, a default requirement by approval authorities for drainage (conveyance) networks that are designed using peak storm bursts alone can limit the adoption of innovative and integrated solutions.

A combination of event based estimation techniques, directly or indirectly, may not reliably produce probabilistic design of drainage, water quality, and water or wastewater infrastructure within integrated strategies. While use of best available event based design approximations are an accepted default or deemed to comply approach for design of infrastructure, there is a need for more advanced methods for design of integrated solutions.

The absence of an integrated approach to design and planning in stormwater catchments may lead to missed opportunities and poor investment decisions, which ultimately results in higher costs with diminished social and environmental benefits (Coombes, 2005). Estimation of stormwater runoff and design of drainage (conveyance) networks for mitigation of urban flooding needs to be enhanced to provide integration with water cycle management within a systems framework.

The definition and purpose of minor or major drainage system is unclear in the context of modern approaches to water cycle management. Replacement of minor or major drainage descriptions with a definition of managing nuisance or disaster respectively, would provide a clearer focus on the relative importance of both concepts. To avoid nuisance, one may be too focused on a prescriptive drainage approach to the minor system. A well-designed major system to avoid disaster is likely to allow more opportunity for integrated solutions that will also mitigate nuisance. We also need to be cognisant that water supply and stormwater quality options can also assist in avoiding disaster and mitigating nuisance.

3.4. Urban Flooding

Urban flooding may include overland (pluvial) flooding and fluvial flooding (river and creek flows). This distinction can be important as the two types of flooding have different behaviours that may require particular analysis and management approaches.

3.4.1. Overland Flooding

Overland flooding is typically generated by short durations (minutes to hours) of intense rainfall on small catchments up to approximately 1 km^2 in area. This rainfall causes significant concentrations of surface runoff at low points and depressions throughout the urban topography. These concentrations of flows continue downslope and discharge into larger natural waterways with defined banks such as creeks, rivers or lakes where flows become fluvial in character.

Overland flooding can be responsible for significant damage. Adequate major flow paths must be provided or retained to manage these events. A stormwater management strategy is required that includes systematic identification of overland flow paths and design practices that recognise and respond to overland flood risks. Simple design practices such as slightly elevating property and floor levels above the surrounding terrain can effectively eliminate most overland flood risks.

Approaches to analysis have been developed in recent years to assist identification of overland flow paths that involve use of two dimensional hydrodynamic models where real and design rainfall events are applied throughout sub-catchments. These methods use digital terrain models of land profiles that are usually derived from LiDAR and aerial photogrammetry information. Hydrodynamic models can predict the accumulation of runoff across these surfaces and the generation of concentrated flows. Depth and velocity depth thresholds can be applied to model outputs and mapped spatially to allow identification of the most significant accumulation of flow. A map of a fluvial flow path prepared using a two dimensional hydraulic model is presented in Figure 9.3.5.

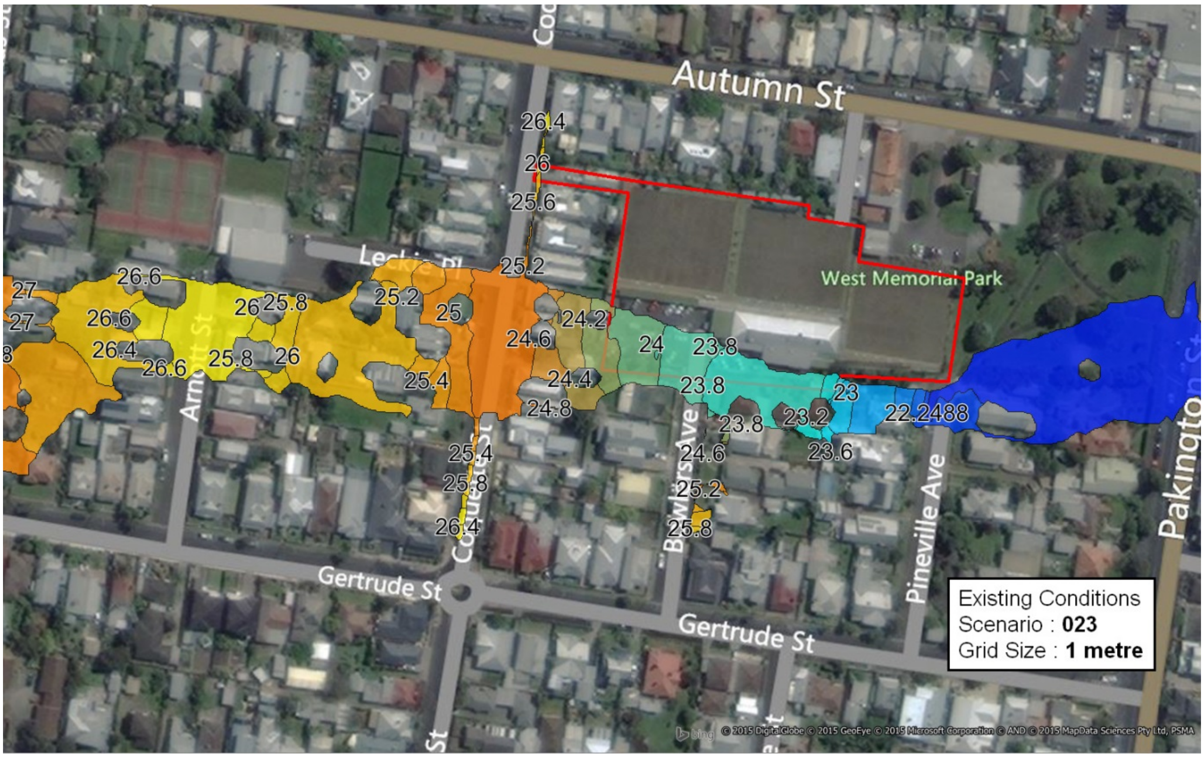

Figure 9.3.5. Example Overland Flow Path Map Generated Using a Two Dimensional Model.

This modelling approach is complex and is undertaken by a designer with suitable experience to ensure reliable outcomes. However, successful application of this method can be efficient and reveal a range of important stormwater management issues, including overland flow paths.

Approaches to analysis with less complexity may be more practical for smaller areas or simpler stormwater management strategies. This may involve the capture of detailed ground survey and inspection of the data by a suitably experienced designer to manually estimate the location of low points and likely flow paths. Simple hydrologic and hydraulic calculations (refer to Book 9, Chapter 5) could then be applied to estimate the depth and width of stormwater at regular intervals throughout overland flow paths.

Caution should always be employed when interpreting the mapping of results for stormwater flows and inundation as there may be significant uncertainties about the results caused by:

- obstructions to flow paths such as buildings and fences;
- rapidly changing flow conditions throughout a flow path;
- limitations in the accuracy of survey information; and
- limited opportunity for calibration.

The application of two dimensional modelling approach produces results that reveal hydrologic uncertainty due to use of the hydraulic model to simulate the natural physical processes of stormwater flows. These results may be in contrast to empirical or statistical relationships between rainfall and runoff that are used to estimate stormwater runoff in some traditional hydrologic modelling software.

Identification of overland flow paths allows development of stormwater management strategies. These may include:

- mapping of flooding to promote public awareness of flood risks;
- education about flood risks;
- investigation of potential upgrades to stormwater management networks; and
- building and development controls.

Flood warning emergency systems are usually inappropriate for overland flooding, as the potential warning times are too short. However, incorporation of overland flooding information with radar rainfall forecasts may assist in providing emergency management warnings.

Building and development controls should include provisions that prevent the erection of new buildings within overland flow paths or set minimum floor levels that are deemed safe. Other building controls may also require measures that minimise potential blockage and obstruction to flows within effected building envelopes. Application of these controls to particular sites may require detailed site-based flood investigations to more accurately estimate flood levels and behaviours.

A freeboard allowance above a calculated flood level is applied to determine the minimum level of infrastructure such as a habitable dwelling. Freeboard is required to account for the uncertainties that are inherent in the calculation of flooding. A typical minimum value of 0.3 m above a flood surface is suggested. However, this value can be varied to account for local factors such as the sensitivity of specific infrastructure to flood damage and expected uncertainty in estimates of flood level estimates for a site. Uncertainty about flood levels are variable and dependent on many factors including the nature of the catchment and the cross-sectional profile across the flow path.

Freeboard should not be used to protect against measurable uncertainties for example risk of blockage and climate change. If these risks are a concern for the site then they should be explicitly incorporated into the basic flood level estimates before freeboard is applied.

3.4.2. Fluvial Flooding (River and Creek Flooding)

Fluvial flooding is often referred to as river and creek flooding, and is generally caused by long durations (hours to days) of intense rainfall across large catchments. These catchments range in area from 1 km^2 to many thousands of km^2. Excess runoff from these catchments accumulates and is concentrated as flows in creeks, rivers and lakes that have natural features such as a main channel and defined banks. Stormwater escapes the main channel at locations where hydraulic capacity exceeded and caused inundation of surrounding land. This flooding can occur across vast areas of flat or low-lying terrain. The extent of flooding can be quite narrow and well defined at locations where the natural topography is incised. Fluvial flooding is generally easier to analyse than overland flooding because the channels are more readily identified and represented using computer models.

This type of flooding is natural. However, careful urban planning is required to avoid substantial damage to infrastructure and property. Fluvial flooding is recognised as one of the most significant natural hazards in Australia that is responsible for a significant proportion of economic losses and damage to property. Therefore, fluvial flooding has been the target of significant government programs for mapping of flood hazards and implementation of measures that mitigate potential economic losses and damage to property.

Fluvial flooding is a constraint to urban stormwater management that needs to be understood as it may heavily influence the type's solutions that are proposed. Numerical methods for the estimation of flood behaviour and identification of fluvial flood hazard are well established and tested. These methods are described in Book 6, Book 7 and Book 8.

The management of hazards created by fluvial flooding differs from overland flooding as the quantity of floodwaters can be much greater and therefore more difficult to control and contain using physical changes to the floodplain. It is often preferable and more cost effective to avoid these hazards using a process of careful urban planning. This is best achieved by the use of strategic plans and a suite of flood related building and development controls.

Public flood awareness mapping, flood education, flood mitigation and flood warning emergency systems become more important where development has already occurred within parts of the floodplain subject to fluvial flooding. Catchments that generate fluvial flooding are often large and the lag between rainfall and runoff can be sufficient which increases the feasibility of flood warning and emergency management strategies.

3.4.3. The Overland and Fluvial Interface

There is often an interface zone within catchments where both fluvial and overland flow paths may exist and differentiation between the two types of flowpaths becomes subjective. For example, a small gully drains through a town directly into a major creek as presented in Figure 9.3.6.

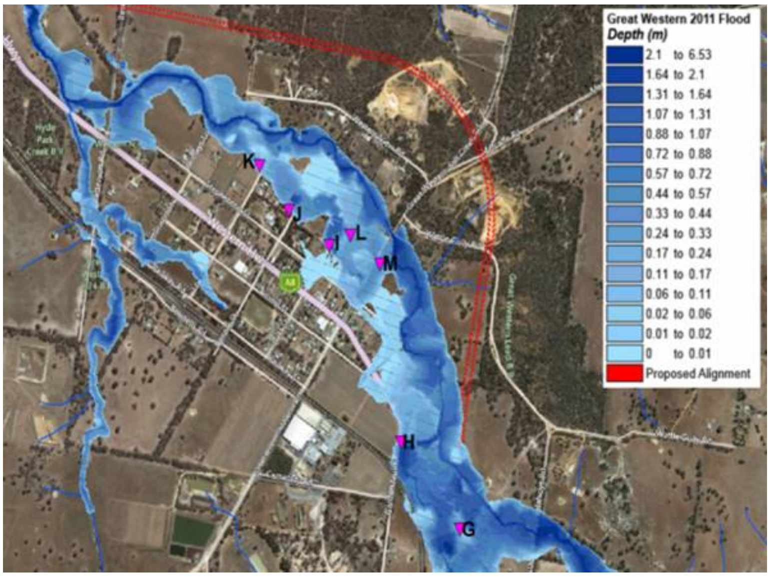

Figure 9.3.6. Example of Fluvial Flow Path with Interface with Overland Flow Path

Analysis of stormwater management strategies at the interface zone requires first principles assessment of management techniques from the perspective of both overland and fluvial flooding.

Both types of flooding can occur simultaneously. However, this is unlikely since the rainfall mechanisms that typically cause each type of flood are different. It is more likely that overland and fluvial flooding will occur at different times and possibly not during a single rainfall event. This complex behaviour can confuse attempts to communicate flood risks and implement management strategies. Confusion also arises when insurance claims are made for loss and damage because the decision to pay a claim sometimes relies upon whether the flooding was overland or fluvial in nature. In addition, the insurance industry has begun to offer fluvial flood insurance cover, which may reduce this problem in future. Nevertheless, it is important for practitioners to recognize the potential for both forms of flooding and carefully assess flood behaviour at each site and for each flood event from first principles.

3.5. Conveyance Systems

A typical stormwater (drainage) conveyance system must convey a wide range of flows within a confined corridor of land (refer to Book 9, Chapter 5). At the same time the system must meet appropriate standards of flood safety and be delivered for low life-cycle cost. This challenge is best addressed through application of a design approach referred to as a ‘major and minor stormwater management system’.

A Major and Minor Stormwater Management System Has Two Parts:

- The minor system manages nuisance. This runoff is conveyed in a manner that maintains safety, minimises nuisance and damage to property. The infrastructure is also provided to avoid potential maintenance problems for example ponding and saturation of designated areas. Importantly, the minor system also includes volume management measures that aim to hold water within urban landscapes and sub-catchments (refer to Book 9, Chapter 4) – these solutions may include ponding of stormwater within a defined area. The minor system must withstand the effects of regular stormwater inundation.
- A major system primarily intended to mitigate disaster. The major system typically includes overland flow paths on roads and through open space, and trunk conveyance

infrastructure. This system conveys additional stormwater runoff produced during larger less probable and rarer storm events with the intent of managing the potential for flood disaster. Overland conveyance of stormwater from large events is potentially hazardous due to the velocity and depth of flows, and must be safely contained within a defined corridor of major system flows.

3.5.1. Capacity to Manage Flooding

The overall combined capacity of the major and minor drainage system to manage flooding or inundation needs to be established for each design. This capacity is normally expressed in terms of the exceedance probability of design rainfall, creating a flood that must be contained within the conveyance or drainage system. It is common practice to set the capacity of the major system at a similar exceedance probability as the flood event used for regional flood planning (e.g. 1% AEP discharge).

However, there may be justification to deviate from this practice where a suitable risk assessment identifies the need. For example where the consequences of flooding at a particular location are high, it may be necessary to expand the overall system capacity to cater for more extreme events. This is not commonly required and this type of decision must have regard to the overall life-cycle cost and benefits that a larger capacity system may deliver.

The threshold at which the capacity of the minor system is exceeded and the major system begins to convey runoff is also a matter for consideration at the design stage or for policy makers at the time when preparing local design standards for stormwater management. The capacity of minor system is typically established to manage stormwater events ranging from 50% AEP to 5% AEP. Documentation of these standards can be found in drainage design guidelines prepared by local government and relevant state authorities. No single universally appropriate capacities of minor systems can be applied in practice.

Some factors that may influence the balance between the capacity of major and minor systems are described in Table 9.3.1. These factors may generate a number of different capacity standards for minor systems that account for different locations and jurisdictions.

Table 9.3.1. Factors Influencing the Balance between Capacities of Major and Minor Systems in Design

Factor	Description
Land availability	Sufficient land may be available for major systems to safely convey additional surface flows and reduce the proportion of flows conveyed by minor systems. The use of volume management and WSUD approaches can also change the proportion of flows assigned to minor and major systems.
Local rainfall patterns	In some areas, such as tropical northern Australia, runoff generated by frequent storms may be too large to cost effectively convey using minor systems. Major flow paths will need to be expanded accordingly to manage a proportion of these flows.
Likely level of exposure to the major flow path hazard	Major systems that are highly frequented by people or vehicles, for example in city streets or major motorways, involve greater exposure to floodwaters and corresponding risks. In these cases, it may be appropriate for a greater proportion of runoff to be conveyed in minor systems.

Factor	Description
Physical and downstream constraints	When new stormwater management systems are required for an existing urban area, it may be impractical or cost prohibitive to achieve an ideal capacity and compromise may be required.
Erosion	Natural or otherwise unlined minor systems may be subject to erosion when flow durations and or velocities are too high. If volume management options (as discussed in Book 9, Chapter 4) are not available, then lowering the capacity of the minor systems and forcing a greater proportion of flow into the major system may be one way to manage these effects.
Blockage potential	Where the capacity of minor systems is reduced by a likelihood of blockage with debris, resources should be directed towards safer and more durable surface flow paths within major systems.
Climate change	The expected future increases in short duration rainfall intensities may require appropriate design responses to increase the capacity of minor systems or change the relationship with major systems to maintain current levels of service.

3.5.2. Alignment and Configuration

The characteristics of urban form including the layout of roads, location of urban parkland and topography will influence the alignment and configuration of stormwater management networks. It is difficult to modify the stormwater management network after installation. A design process should aim for a long service life. Concept planning for major and minor stormwater management systems should therefore be undertaken carefully as an early task in the design of new urban developments.

The depth and velocity of flows along any proposed surface flow paths are considered when calculating the dimensions of stormwater conveyance corridors and must meet relevant standards for design, safety and maintenance. A design should also ensure that operation of a conveyance network during severe storms does not cause unexpected or catastrophic consequences (for example, an unintended diversion of flows into an adjoining catchment because of blockage or extreme events).

Wherever possible the width of the land corridor set aside for stormwater management should be generous to improve the constructability of the system and reduce the costs of any future renewal and maintenance activities. Opportunities for co-location of stormwater management within urban parklands should be considered. The alignments of stormwater conveyance networks typically follow natural low points to minimise earthworks. However, some re-alignment away from the natural low points may occur to account for urban form and limit conflicts with other urban infrastructure. However, the design of conveyance networks should also consider minimising damage to existing ephemeral waterways.

Alignments of major systems are often parallel to minor systems and should be continuous until intersection with a natural watercourse or receiving waters. The design should include adequate management to avoid nuisance or risks at crossings, for example roadways or footpaths.

Configuration of stormwater management strategies (including conveyance networks) will depend on the land use within and alongside the selected overland flow paths (refer to Book 9, Chapter 5). This configuration may also vary throughout a stormwater management solution. Some of the typical configurations deployed in Australian design practice are presented in Figure 9.3.7. The most common configuration (shown in Figure 9.3.7) comprises an underground conveyance (inlet structures and pipes) network (minor system) within surface flow paths on roads (major system).

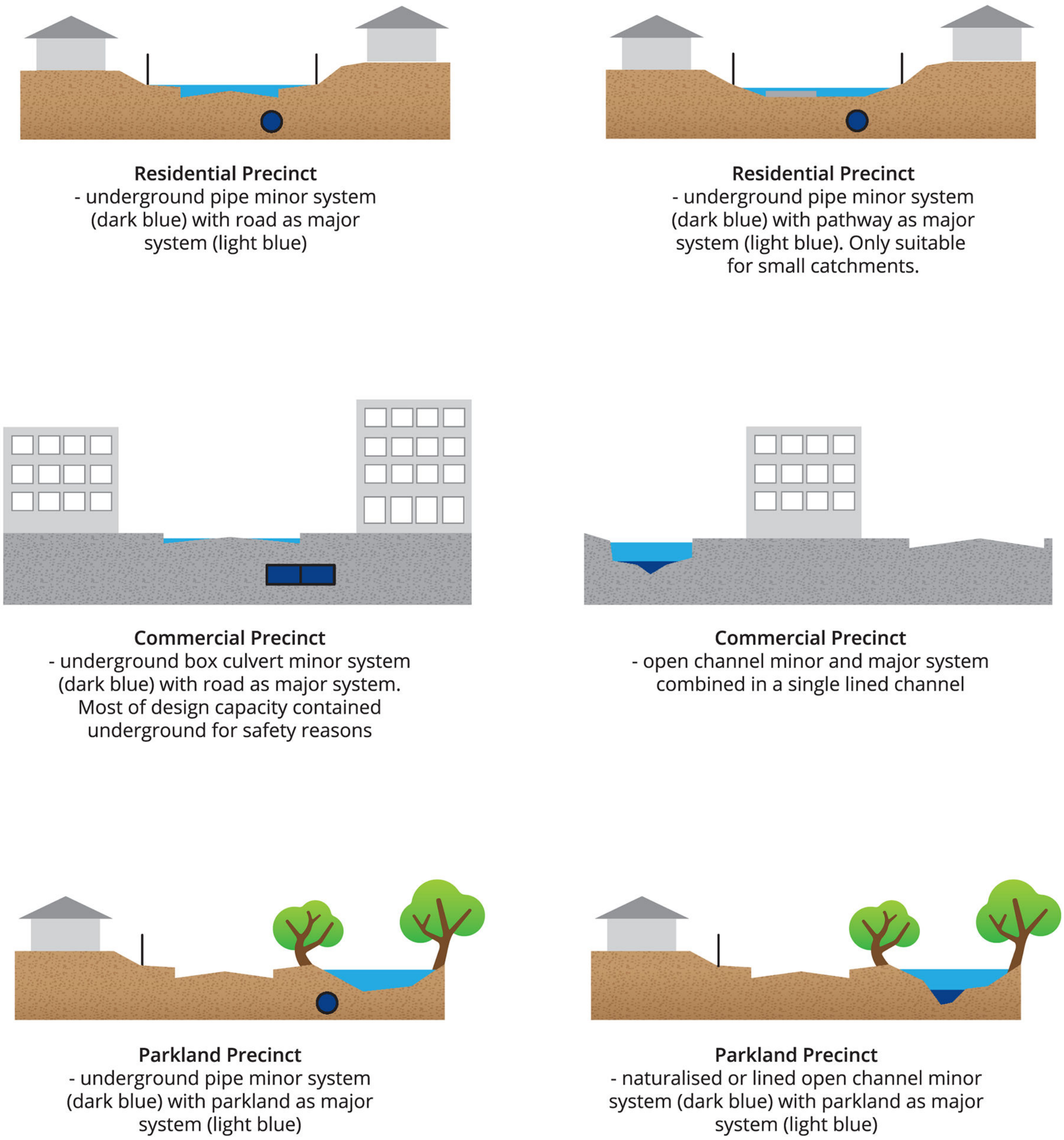

Figure 9.3.7. Typical Configurations of Major Minor Conveyance Systems Deployed in Australian Practice

The design of the major and minor systems should integrate smoothly with other urban infrastructure and manage impacts on natural environments. In particular, innovative design of urban parks can be used to achieve drainage objectives while also enhancing aesthetic and environmental outcomes.

Innovative approaches to stormwater management strategies can reduce construction costs and requirement for land area. This opportunity should be given early consideration in the concept design phase from perspective of multi-disciplinary teams.

3.5.3. Analysis

Suitable hydrologic and hydraulic calculation methods, described in Book 9, Chapter 4, Book 9, Chapter 5 and Book 9, Chapter 6, are used to estimate depths and velocities of stormwater flows with associated extents of flooding throughout major and minor systems, which facilitates the design of various components. The methods selected for analysis or design must be able to simulate the complexity of the stormwater management strategy. A design problem may include complex flow behaviours, for example parallel underground and surface flow paths, multiple inflows and the effects of storage and tail water conditions.

These methods must have the capacity to predict the hydraulic performance of the overall system and of each different component within the system for example inlet structures, pipes and channels. Hydraulic performance must be assessed using a range of storm events and configurations. Ideally, a design should be challenged by ensembles of full volume storm events to determine the critical storm duration and shape for each AEP.

The available software modelling tools can facilitate most of these complex calculations. However, emerging engineering practice and software tools aim to seamlessly handle the full range of linked hydrologic and hydraulic calculations required to account for surface flow behaviours throughout complex conveyance networks. These complex scenarios may require combinations of hydrologic models linked to hydraulic models with one dimensional conveyance network and two dimensional surface flows.

3.6. Stormwater Volume Management

3.6.1. Key Considerations

The historical practice of designing urban stormwater management has traditionally focused on peak flows and conveyance. Design standards have evolved to require comprehensive management of hydrologic changes created by urbanisation. It is now recognized that volume and regimes of stormwater runoff need to be managed (Beven and Alcock, 2012; Poelsma et al., 2013).

Typically, this is achieved through the design and installation of volume management facilities. Detailed aspects of these facilities are described in Book 9, Chapter 4, however at a philosophical level the questions that need consideration when developing a catchment-wide volume management strategy are:

What are the Volume Management Objectives for the Catchment?

Volume management objectives can include control of peak discharge, harvesting or infiltration of water and water quality treatment (refer to Book 9, Chapter 4, Section 2). These objectives are achieved using a volume management facility (either a single facility or a number of them) which can store and release runoff at different times, or even store runoff for later use.

The impact on downstream floodplains and receiving waters must be determined by assessing the catchment–wide consequences of compounding peak flow and volume discharges (increases in runoff volume and peak discharges) from different sub-catchments, as well as increased duration of flows in ephemeral aquatic ecosystems. This impact assessment will then help inform a decision about the volume management objectives to be pursued.

Phillips and Yu (2015) suggest whilst undertaking these assessments, catchment managers should also consider whether to use an ensemble of complete storms with a storm burst of around the critical duration or a storm burst only to determine the benchmark condition(s). The decision of what design to adopt can be informed through identifying the level of risk the community is willing to accept within the catchment.

Should the Objectives be Achieved in Combined Facilities?

It is preferable to provide infrastructure that meets multiple objectives. Where multiple volume objectives are sought for the catchment, it is possible to design separate volume management facilities that each target only a single objective.

For example, a facility might only manage peak discharge from a site for a single probability design flood event used for regional flood planning (e.g. 1% AEP). This might be achieved by storing a proportion of the hydrograph volume and releasing it later during the storm event through a constricted outlet. This is commonly called a detention basin or retarding basin.

Separate facilities might be required to also meet other stormwater volume objectives for example a rainwater tank for harvesting and a bio-retention basin for water quality improvement.

A more comprehensive facility might aim to achieve a peak discharge control objective alongside other volume objectives, by storing a proportion of the hydrograph volume and releasing it well after the storm event has finished, or even store it for later use (i.e. not released into the stormwater system at all). For example a constructed wetland (water quality) with an extended detention storage compartment above (peak discharge control), providing pre-treatment for a stormwater harvest facility (retention).

What is the Performance Level Sought?

For each facility and objective it is necessary to determine whether the facility must achieve a low or high level of performance.

For example, it may be sufficient to retain the hydrologic conditions equivalent to a pre-developed condition, which might be considered a low level of performance.

In some circumstances a higher level of performance might be required, for example, a return of hydrologic conditions back to a natural state.

The performance level sought will be related to the sensitivity of the downstream receiving waterway and whether the local community aspires to achieve a high performance solution.

Where should Volume Management be Achieved in the Catchment?

In some circumstances, there is opportunity to make broad strategic decisions about the distribution of these facilities across a catchment. Some typical volume management strategies that can be followed include:

- An ‘at source’ management strategy: this employs small facilities, widely distributed across the catchment, many of which will only service a small catchment or single property. Strategies of this type are most commonly part of a more comprehensive and integrated urban water strategy.
- A ‘neighbourhood scale’ management strategy: this strategy employs larger facilities that are less widely distributed than lot scale facilities but servicing larger catchments. These facilities are normally publicly managed and co-located alongside a watercourse or drainage reserve at the interface between underground and surface conveyance paths.

- A 'regional scale' management strategy: this strategy uses very large facilities that are located at the catchment outlet and service all properties in the watershed. These are normally publicly owned and co-located with major parkland. This is also referred to as an 'end of pipe' strategy.

How does Existing Urban Development Influence the Volume Strategy?

Some typical types of urbanising catchments and their associated volume strategy considerations are:

- **Future growth areas** where there is currently limited urban development (also commonly referred to as 'Greenfields' development). For these catchments the over-riding strategic objective commonly applied is to preserve the nature and amenity of their waterways in terms of hydrology (flow and channel geometry) and aquatic communities. This can be achieved using 'source control' measures applied throughout their contributing catchments. These measures include rainwater tanks, bio-retention facilities, 'rain gardens', infiltration trenches, 'soakaways' and access to aquifers where soil and geological conditions are favourable.

  Since there is often opportunity to forward plan in 'greenfields' catchments there may also be opportunities for comprehensive 'neighbourhood scale' and 'regional scale' placement strategies.

  Every effort should be made in these catchments to encourage 'informal' drainage, green spaces and to disconnect as much impervious surface as possible. The criterion for successful design of these systems is keeping the volume discharged from each site the same after development as before, for design flood events. Use of these practices, is referred to by Argue (2017) as a 'regime-in-balance' strategy. It is suggested that adoption of such a strategy can keep urban waterways operating as natural systems for many years before increased urbanisation might then require the introduction of rectification strategies such as increased channel lining.

- **Highly urbanised catchments** where the strategic objective is often to minimise the need for further modification or upgrades to conveyance networks as development and re-development continues. For these catchments land availability may constrain opportunities for wide adoption of 'neighbourhood scale' and 'regional scale' placement strategies. However volume management objectives can be achieved in a similar manner to a 'greenfields' catchment using 'source control' practices as re-development takes place. An additional opportunity, 'roof gardens', is provided by the presence of multi-storey and high-rise elements of this class of development.

  The objective for successful design of these systems is keeping the volume discharged from each site the same after development as before, for a design flood event. This objective is more difficult to achieve than in 'greenfields' catchments giving rise to the more common use of temporary on-site storages holding stormwater after flood peaks have passed. This problem can be solved by 'slow release', infiltration or harvesting to ensure storages are empty ahead of closely-spaced storm events. With such provisions in place, the supporting infrastructure can continue to operate successfully without enlargement.

  Prediction of Australia's urban growth to mid-21st Century suggests that development within catchments of this type will provide the majority of new urbanisation.

- **Over-developed catchments** are a particular case of highly urbanised catchments described above, and apply to many of our older, inner-city suburbs. These catchments are characterised by frequent episodes of flash flooding and resulting community disruption.

The criterion for successful design of these systems is not just to match pre-development conditions but to go further and minimise the volume discharged from each site after re-development. This is referred to by Argue (2017) as the 'yield-minimum' strategy. The nature of re-development in an already over-developed urban catchment is frequently large-scale, for example urban renewal projects. These lend themselves to complete re-organisation of local drainage infrastructure and, hence, opportunities for less discharge during the 'design' runoff events. Every component of re-development incorporated under the 'yield-minimum' strategy moves the catchment in the direction of a balance between runoff being generated and infrastructure capacity.

Are There Other Constraints that may Influence the Strategy?

Catchment managers will also need to take into account the local landscape and soil conditions, which may limit the application of certain volume and quantity management solutions. For example, heavy clay soils may limit the application of infiltration based solutions, whereas sandy soils may promote such solutions.

Other examples of constraints that may have strategic influence are:

- sensitive riparian vegetation communities
- land ownership and development patterns, and
- different choices may be required depending on the nature of the catchment and the asset policies of the local stormwater authority.

3.6.2. Selecting a Strategy

Once the above questions have been considered it might be appropriate to establish and document a catchment-wide strategy for stormwater volume management. Such a strategy should be used to assist with the design and assessment of individual volume management proposals.

Typical catchment management strategies (as designed using bottom up or top down methods or other analyses) can include a number of different approaches which reflect the local authority's commitment to WSUD principles, as well as commitment to restore overloaded systems to balance.

Three examples of management strategies for catchment-wide volume management are provided by Argue (2017). These are consistent with the risk management framework discussed in Book 1, Chapter 5 and are defined as follows:

Yield-maximum: maximise the quantity of storm runoff captured at the end of the catchment and ensure that the floodwaters are contained within a defined floodplain. This strategy is most suitable for local authorities with a desire to have large centrally controlled systems, rather than distributed local solutions.

Regime-in-balance: maintain the harmonious and synergistic relationship that exists between continuing urban development and 'acceptable' use of the floodplain for agricultural and amenity pursuits. This strategy is most suitable for catchment or sub-catchments where development has occurred or is likely to occur and will discharge to a nearly intact or sensitive receiving environment.

Yield-minimum: improve the performance of the urban flood control infrastructure through minimisation of stormwater discharge from each development site (including redevelopment sites). This strategy is most suitable for catchment or sub-catchments with already poorly controlled urban development with a history of flood damage and ecosystem deterioration.

Large catchments, where urbanisation is actively occurring, and over an extended period, may contain precincts where a mix of these strategies might be appropriate. Notably, all strategies will benefit from urban planning that promotes rainfall infiltration, harvesting and retains natural hydrologic function.

3.7. Stormwater Offsets

Tradeable permits or offset schemes are also known as market mechanisms and are established methods within the pollution control industry, in water markets and for management of nutrient or salinity loads in river basins. These processes commonly involve financial contributions paid by a landholder for provision of pollution control works at another location, construction of an alternative mitigation scheme instead of a conventional solution in the landholders development site, or the sale of a water licence from a landholder to another landholder at another location.

Tradeable permits for pollution control are attractive as they provide opportunities for economic efficiency, flexibility and incentives for innovation (Kraemer et al., 2004; Haensch et al., 2016). The international experience with water pollution emission trading is not extensive but does include some successful examples (Shortle, 2013). Trading of pollution abatement responsibilities can cause water quality to deteriorate at different times and rates in some parts of a catchment. Therefore designing a tradeable permit or offset scheme needs to take spatial, temporal and environmental equivalence effects into account.

At the time of writing, Melbourne Water (MWC, 2018) and Queensland Healthy Waterways (Water by Design, 2014) (for example) operate stormwater quality offset schemes. These schemes involve a financial contribution paid by developers for stormwater management works to be undertaken in another location to meet catchment wide objectives for managing stormwater and protecting waterway health. These schemes respond to the assumption that regional stormwater management is more cost and time effective than distributed smaller scale solutions. These off-set schemes can be useful for urban areas subject to infill development that may have limited space for infrastructure.

There are limited examples of trading or offset schemes for management of stormwater runoff volumes or peak flows. The District of Columbia Water and Sewer Authority (for example: DCWater (2018)) provide an impervious area charge incentive program for customers to reduce effective impervious surfaces and, therefore, stormwater runoff on their properties which avoids regional works. Properties that use best management practices such as rain gardens, rainwater harvesting, green spaces and pervious paving are considered to reduce effective impervious surfaces and results in a reduced stormwater charge. Similarly, the historical on-site detention (OSD) strategies by the Upper Parramatta River Catchment Trust (for example: UPRCT (2005)) offset the need for regional stormwater basins by use of detention storages (OSD) on properties.

Use of formal stormwater off-set schemes to transfer local management of stormwater volumes and peak flows to regional facilities is not common, but these types of approaches are embodied in most developer contribution schemes for regional infrastructure. Stormwater off-set schemes for management of runoff volumes and peak flows should include the following key principles:

- Transfer of stormwater management to another location should not negatively impact on surrounding local properties at any (legal) point of discharge

- The spatial, temporal and cumulative allocation of required treatment capacity must be defined using a catchment management strategy. It is unlikely that transfer of local stormwater management requirement to a downstream regional location will be a linear or average process

- A scheme must result in the desired and measurable changes in flow (and water quality) resulting from the infrastructure and stormwater strategies within the same catchment

- The funds obtained from stormwater off-sets must be tied to measurable deliverables in the catchment

- The scheme must provide for regional infrastructure in a reasonable time period that is consistent with the timing of upstream development.

- The relative financial contributions from upstream developers must be proportional to their flow and pollutant loads that will be managed by the regional scheme

- The scheme must have the same life cycle or equivalent life cycle as the life cycle of the upstream development (e.g. short-term mitigation strategy, such as flow and erosion management, cannot be used for a long-term offset to a developed area stormwater management)

- If water quality is part of the scheme, consideration should be given to the bio-availability of pollutants removed through the different upstream and catchment wide management methods

- Clear ownership and rules about the off-set scheme should be established and risk should be mitigated through the adoption of appropriate ratios, and

- The ongoing maintenance and renewal costs associated with the regional infrastructure must be allocated to ensure the performance of the scheme does not deteriorate over time.

Stormwater off-set schemes that transfer management of stormwater volumes and peak flow to other locations have the potential for ecological impacts in local waterways or downstream receiving waters. The ultimate objectives of an off-set scheme should include performance targets that also consider secondary effects (such as impacts on local waterways) and monitoring strategies should be implemented to measure effects of strategies.

Chee (2015) highlights that there is limited evidence of success of stormwater off-set schemes and formal monitoring strategies would provide an opportunity to more critically consider the evidence of how well schemes that have been implemented and their operation. It is also emphasized that achieving equivalence in stream biodiversity and ecological function is extremely difficult.

Coker et al. (2018) argue that stormwater off-sets should not result in avoided management of stormwater runoff. They emphasize the substantial challenge of adequately considering spatial, temporal and environmental influences of off-sets, and the importance of quantifying the spatial extent of stormwater impacts from the development in question. It is highlighted that unmitigated stormwater runoff from relatively small proportions of urban areas may propagate severe impacts a long way downstream which can render the practice of offsetting within a single catchment a difficult undertaking.

3.8. Acknowledgements

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Chapter 4. Stormwater Volume Management

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With contributions from John Argue, Brett Phillips and Urban Book Editor Peter Coombes

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4.1. Introduction

Progressing from the urban stormwater philosophy discussed in Book 9, Chapter 3, this chapter provides introductory guidance on the design of 'volume management facilities'. These are discrete infrastructure measures in various forms and configurations, each of which are designed to store and release runoff volumes to manage the changes caused by urbanisation. They are linked by conveyance infrastructure (refer to Book 9, Chapter 5) to form an urban stormwater network.

This chapter focusses on the concept design phase of a volume management facility and outlines the detailed design process. Before applying the content in this chapter it is assumed that the general position of the facility within the catchment is already largely understood, and preferably informed by a catchment strategy, as discussed in Book 9, Chapter 3 and Book 9, Chapter 5.

Stormwater storages receive runoff volumes from the catchment via upstream conveyance infrastructure. The manner in which these runoff volumes are managed depends on the practice that is adopted. The storage and release of runoff changes the characteristics of the runoff hydrograph and is a fundamentally important feature of all volume management facilities.

There is considerable legacy terminology used to describe these facilities including detention (or retarding), retention, extended detention or slow release. These terms are a derivative of outlet structures and different operational strategies that change the behaviour of stormwater storages.

Stormwater storages designed in accordance with 'detention' practices include those where runoff is temporarily stored and simultaneously released via an outlet structure (Figure 9.4.1). This process typically lowers peak discharge and attenuates the hydrograph so that the average time of release is delayed. The storage volume and capacity of the outlet must be determined by catchment wide modelling to achieve target outflow peak discharges at the catchment outlet.

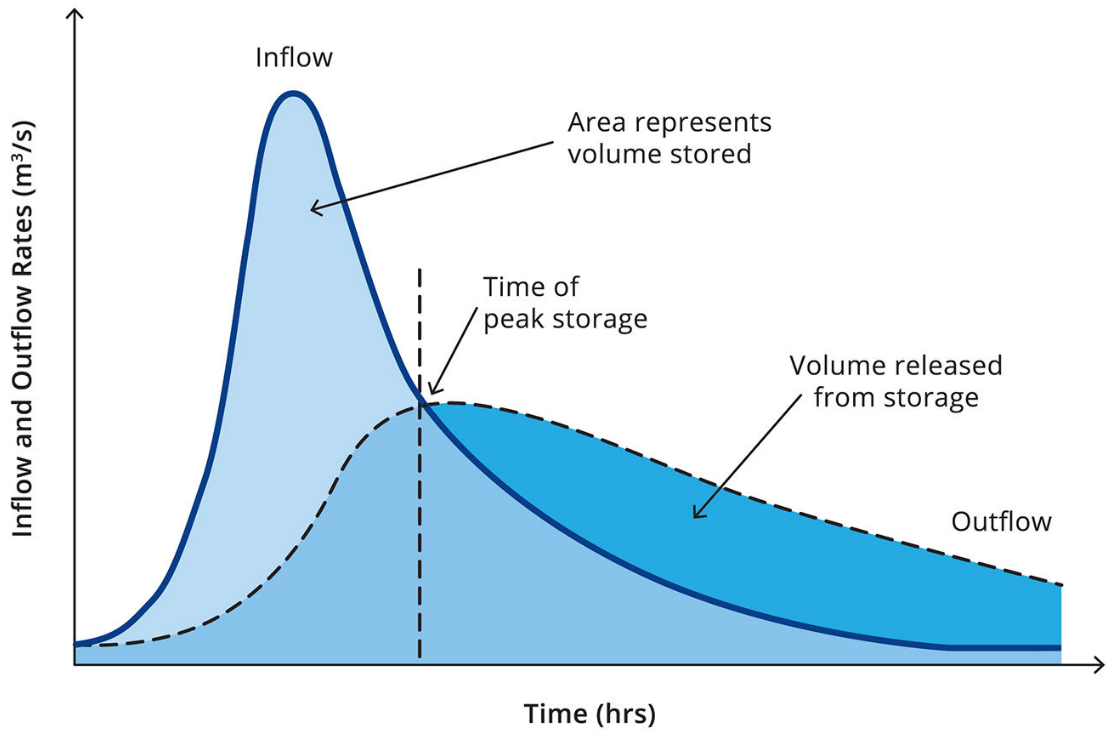

Figure 9.4.1. Typical Hydrograph Change Generated by a Temporary Storage (Without Harvesting)

Assuming the stormwater storage is empty at the beginning of a storm, the potential hydrograph change that can occur depends on:

- the outlet's discharge capacity relative to the peak discharge of the storm;
- the size of the storage basin volume relative to the total runoff volume from the storm; and
- the volume of water harvested from the storage.

As a general rule, if the storage volume is large relative to the total runoff volume, the greater the potential hydrograph attenuation that can occur. This performance also depends on the outlet capacity. A small outlet capacity relative to peak inflows will tend to favour attenuation of small storms and large storms it will overflow early, whereas a large outlet capacity will tend to favour attenuation of large storms and small storms will pass through the facility without attenuation in storage. While the storage and outlet structure are separate physical components of a volume management facility, they must be designed in an integrated manner since the capacity of the storage will effect the performance and sizing of the outlet structure and vice versa. This is a critical aspect of the design of a volume management facility with detention characteristics that requires an iterative approach to sizing.

Stormwater storages designed in accordance with 'retention' practices provide sufficient storage in the volume management facility to contain additional runoff from urban development. The volume of stored stormwater is then drawn down by infiltration, harvesting or slow release. Typical hydrographs of flows from a rural catchment and subsequent urban development of the catchment are presented in Figure 9.4.2. Inflow and outflow hydrographs which apply to a volume management facility used in a typical retention strategy, are shown in Figure 9.4.3.

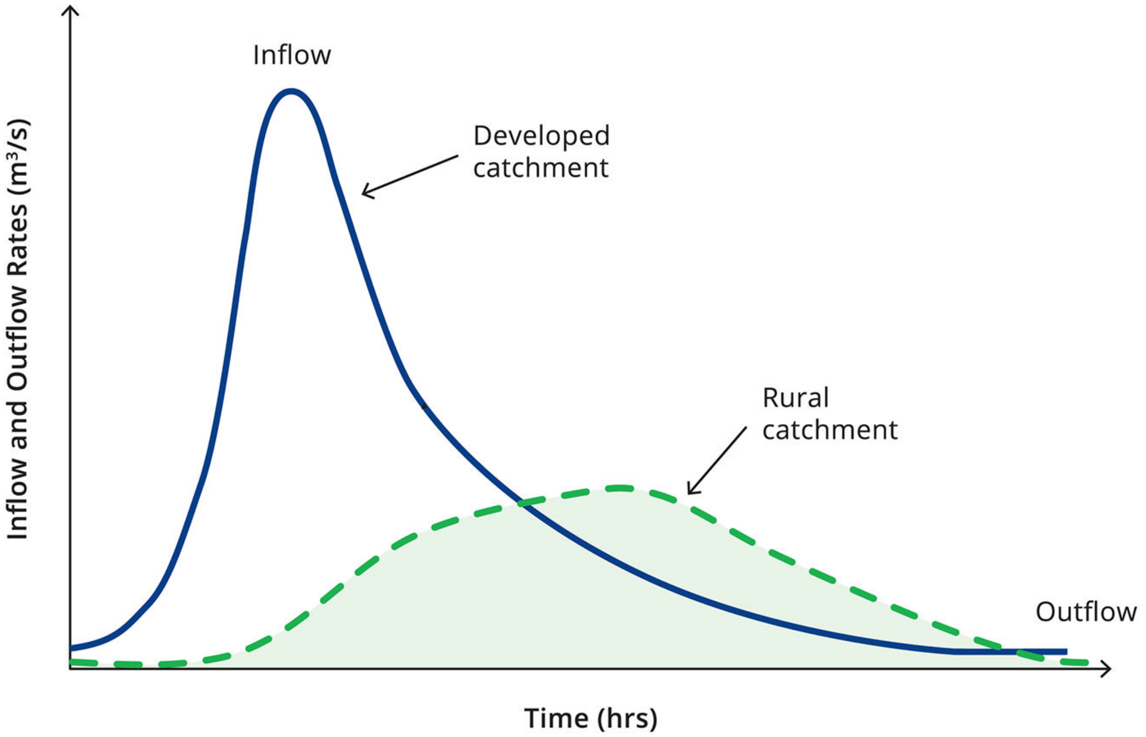

Figure 9.4.2. Rural and Developed Catchment Hydrographs

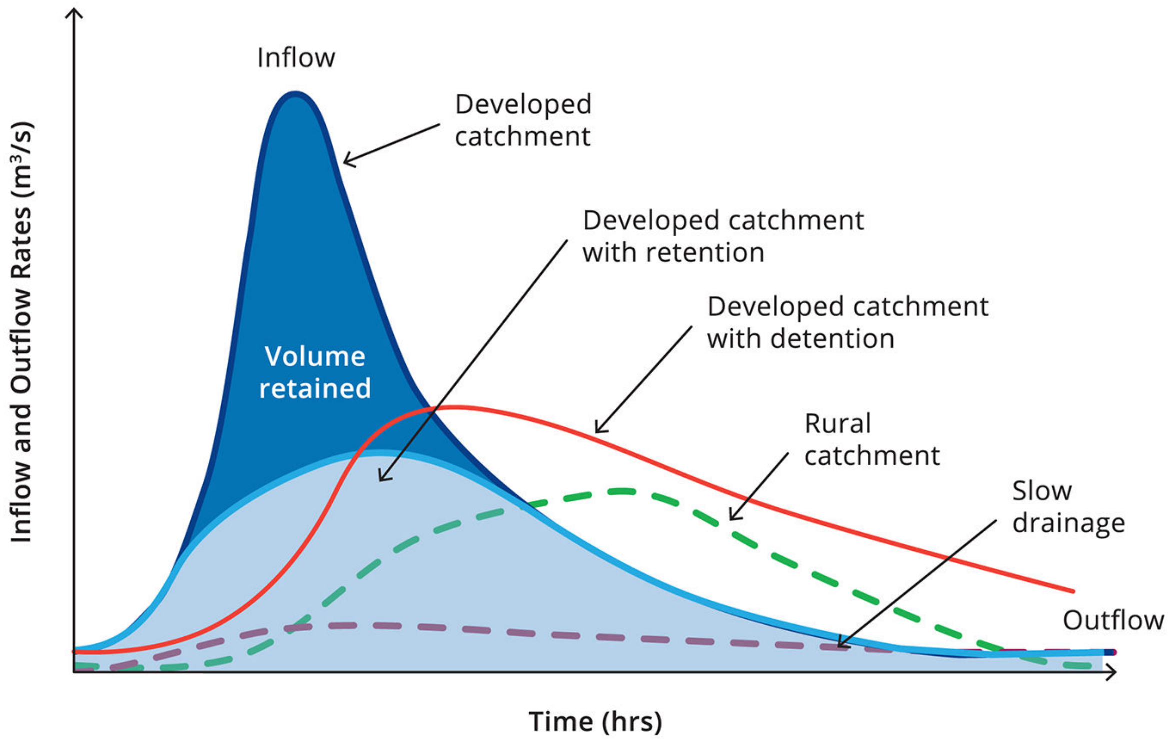

Figure 9.4.3. Developed Catchment with Retention as Compared to Detention and Slow Drainage Strategies

The hydrographs in Figure 9.4.2 represent runoff from a rural catchment and from the urban landscape developed on it. Ideal retention performance of the storage is reproduction of the rural hydrograph followed by outflow of the remaining stored runoff via slow release over a longer duration (typically greater than 24 hours). Argue (2017) outlines that it is difficult to achieve this outcome and recommends a storage volume equal to the total additional runoff expected from the development and the emptying time of volume management facility is a function of outlet infrastructure.

The outflow hydrograph resulting from this approach should be similar to that shown in Figure 9.4.3 (developed catchment with retention). A first approximation solution is likely to produce a different outflow hydrograph from the required result. Continuous simulation of the volume management facility is recommended with the aim of adjusting the design i.e. storage and outflow configuration, to produce the desired outflow hydrograph.

The concept design phase of volume management facilities commences with a thorough understanding of the volume management objectives intended for the facility (refer to Book 9, Chapter 4, Section 2). Once these objectives are defined, consideration can be given to the configuration of the facility and how its components might be sized and positioned to best meet the objectives and local site conditions (refer Book 9, Chapter 4, Section 3 and Book 9, Chapter 4, Section 4). Detailed design then follows to comprehensively define the facility to permit construction (refer to Book 9, Chapter 4, Section 5).

4.2. Volume Management Objectives

The design of a volume management facility must include objectives which are relevant to the site, the surrounding catchment and receiving waterways. A summary of the most commonly encountered volume management design objectives in Australian practice is provided in Table 9.4.1. Each objective has 'associated benefits' that are also listed to help distinguish the relevance of each objective to a particular site and design.

An adequate number of facilities are required within catchments to ensure that the controls will significantly affect peak discharges, volume targets and water quality targets at catchment outlets. A key aspect of the design of storage based measures is to ensure that the storages are empty or nearly empty at the commencement of a flood producing rain event. It is essential to determine the spectrum of design flood events that these facilities will manage (refer to Book 9, Chapter 3).

Table 9.4.1. Summary of Volume Management Design Objectives

Objective	Potential Associated Benefits
Control Peak Discharges This objective seeks to limit the peak flood flows and volumes discharging from a catchment to a pre-determined and acceptable level. Commonly the acceptable level is set at the natural or 'pre-development' condition. In some cases the acceptable level may be set below the natural condition in order to achieve a net benefit or offset an impact elsewhere. In highly developed catchments (infill development), the acceptable level may correspond to flows from the original development. These objectives may seek to change the total volume of stormwater leaving a site (retention), or delay the volume for a	• Reduced property flood damage • Reduced personal safety risks due to flooding • Reduced infrastructure damage • Reduced conveyance infrastructure

Objective	Potential Associated Benefits
short period of time (hours) (detention or retarding) which may reduce the peak of the flood hydrograph discharging from a catchment. Careful consideration of the spectrum of design flood events needs to be given and its impact on downstream receiving systems (for example stream forming flows and flood flows), which can result in 'slow release' systems. Emerging stormwater management practices seek to reduce the volume and timing of stormwater discharges from catchments. This combined approach is particularly relevant for managing stormwater runoff from increasing urban density (refer Book 9, Chapter 3). This objective is a very commonly sought outcome. It is of most relevance to urban catchments where there is a constrained floodplain downstream or sensitive ecosystem that cannot accommodate increase in peak flood discharges or volumes.	requirements (downstream)
Harvest or Infiltrate Rainwater or Stormwater This objective seeks to extract a proportion of the runoff volume from a catchment and either use this water for a consumptive purpose (i.e. consistent use to ensure draw down of storages), or infiltrate the runoff directly into local soils or subterranean aquifers (possibly for later extraction). These integrated design approaches can require interaction with soil properties, capacity of aquifers, urban form and demands of water (refer Book 9, Chapter 3). The designer should account for the elements in the design of a catchment wide strategy to ensure that adequate storage space is available in storages to achieve the objectives of the strategy. Analysis of these measures must include continuous simulation and the use of full volume storms to understand the required storage capacity for a given set of rainfall events.	• Maintain waterway stability and reduce scour • Maintain groundwater behaviour • Maintain hydrologic behaviour including natural runoff regimes • Increase volume of water stored in an aquifer • Increased availability of water for harvesting and use
Improve Water Quality This objective seeks to reduce concentrations and loads of contaminants within urban runoff to pre-determined and acceptable levels. This is achieved by: delaying some of the runoff volume for a period of time (hours to days) (detention), or storing part of the stormwater on-site (retention) and passing the retained water through treatment processes where physical, chemical and biological processes reduce contaminants in the water column. Storage of stormwater can also provide some limited water quality treatment through settlement, even where this objective is not necessarily sought.	• Maintain aquatic health • Maintain visual amenity • Improved water quality prior to discharge or prior to harvesting activities

An early design task should examine the relevance of the objectives from Table 9.4.1 for a design in the context of prior studies, investigations, catchment strategies and receiving waterbody conditions. This process allows the designer to establish a preliminary understanding of the behaviour of the site, the catchment and receiving system. Another important task is to check local stormwater authority and state government policy requirements and standards. In the absence of background studies and local authority guidance, the designer should critically assess the relevance of the above-listed objectives from first principles. The ‘associated benefits’ listed in Table 9.4.1 may assist.

Volume management initially emerged as a design consideration to control of peak discharges in catchments. This was driven by a need to manage flood impacts associated with development and an emerging understanding that the stormwater runoff behaviour of urban catchment is volume dependent. Nevertheless, the design process was driven by peak rainfall bursts rather than the full volumes of storm events. Progressively, as our understanding of urban impacts on waterways has broadened, standards have changed to the point where it is now quite common for the other volume management objectives listed in Table 9.4.1 to also be considered. Facilities that target these multiple objectives have a stronger business case and are therefore more commonly sought after in modern practice and use the full spectrum of storms to protect the downstream receiving systems.

If there are indeed multiple objectives sought for a design, it may be advantageous to design a single facility that will meet all the desired objectives. However, current stormwater management practice incorporates multiple solutions across scales to better manage risk profiles (refer Book 9, Chapter 3). Figure 9.4.4 shows how more than one design objective can be relevant to a site or a catchment, or an entire stormwater management strategy. For example, design objectives for a facility or a strategy may include:

- control peak discharges and harvest (or infiltrate) stormwater;
- control peak discharge and improve water quality;
- improve water quality and harvest (or infiltrate) stormwater; and
- control peak discharge, improve water quality and harvest (or infiltrate) stormwater.

Where possible the design process should pursue performance characteristics that target all the desired objectives. This goal is most likely to be achieved when a particular management strategy is selected as the primary objective, for example peak discharge reduction or water quality improvement, and the subsidiary objectives are incorporated by exploiting opportunities made available by the primary objective.

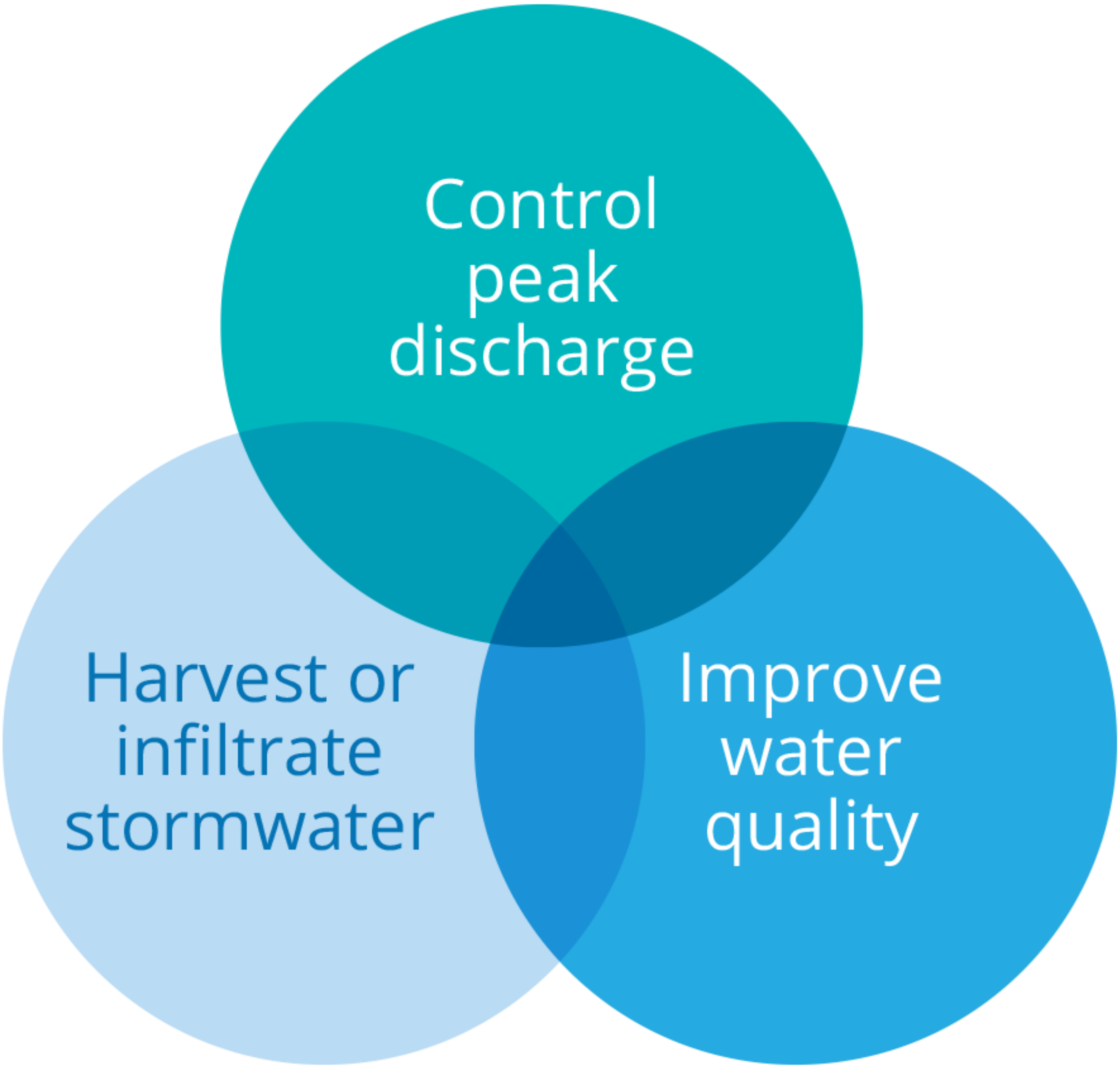

Figure 9.4.4. Potential Overlapping Volume Management Design Objectives

4.3. Components of a Volume Management Facility

4.3.1. Overview

There are up to four generic infrastructure components that are common to majority of volume management facilities; an inlet structure, storage, an outlet structure, and treatment media. These are described in Table 9.4.2.

Table 9.4.2. Volume Management Facility Components

Component	Purpose	Examples
Inlet Structure A conduit or flow path that controls the inflow into the facility and connects the	To transition flows from the upstream conveyance system into the storage device in a controlled manner (refer Book 9, Chapter 5 for	• Headwall outlet structure with riprap • Level spreader

Component	Purpose	Examples
upstream conveyance network to the storage.	more details on conveyance system outlets).	• Energy dissipator
Storage An area of land or a storage structure that contains water after rainfall occurs.	To receive and store a pre-determined volume of water for a pre-determined period of time. Partial discharge from the site and partial retention in on-site storage facilities.	• Small storages such as a On-Site Detention (OSD) tank • Large storages such as basins • 'Nested' basins • Roof gardens, rainwater tanks, bio-retention facilities, raingardens, infiltration trenches, soakaways, access to aquifers
Outlet structure A conduit or flowpath that connects the storage basin to downstream conveyance infrastructure	To control water release from the storage at a pre-determined rate and direct it to the appropriate location downstream. To control water release from the storage to a pre-determined slow release rate. Control outflow from the storage to satisfy a required emptying time criterion.	• Pipe or box culvert through an embankment (with headwall or pit entry) • Discharge control pit • Rainwater distribution system • Spillway across the top of an embankment • High overflow discharge pipe • Aquifer infiltration zone • Combinations of the above
Treatment processes A physical installation located, in-line or off-line, usually within a storage, upstream of a site, neighbourhood or regional discharge point. A material or process that removes water-borne contaminants from runoff as it passes through the storage basin.	To reduce or remove concentrations of contaminants from runoff as it passes through the device towards the outlet.	• Sediment forebay • Gross pollutant trap • Aquatic plants • Vegetated soil media • Sand, gravel or other filtration media • Storage processes including settlement, bio-reaction and natural flocculation

Each of these components can be configured and combined with the other components in different ways to meet different design objectives. The size, shape and material of each component can also be selected to respond to performance criteria and site constraints. Some components can be omitted depending on the design objectives. For example treatment processes are only required where the design seeks to improve water quality or the impacts of the storage on improving water quality need to be enhanced.

4.3.2. Common Configurations in Australian Practice

There are a large number of potential sizing and configuration options available to the designer. Changes to the relative sizes of each component (from Table 9.4.2), along with combinations of different materials and different hydraulic designs can adjust the way in which an overall facility or strategy will perform against the volume management objectives and respond to different site constraints.

The volume management facility configurations that are in common use in Australia are listed and described in Table 9.4.3. Further guidance on selecting a specific design configuration is also provided in Book 9, Chapter 4, Section 4 and Book 9, Chapter 4, Section 5

Table 9.4.3. Common Volume Management Facility Configurations in Australian Practice

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
Detention Basin (Retarding Basin)	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. The size of storage to be determined from catchment-wide analysis focused on the target peak flow at the catchment outlet. Normally dry.	A concrete pipe or box culvert passing through the embankment at the base level of the storage. A spillway at the top level of the storage to pass flow in excess of the culvert capacity.	Nil	Neighbourhood Precinct
On-Site Detention (OSD)	A small underground tank or surface depression. Normally dry.	A small pipe at the base level of the storage with an orifice to reduce outlet flow rates. A small weir at the top of the storage to pass flow in excess of orifice capacity (Figure 9.4.7 and Figure 9.4.8).	Nil	Lot Site
Rainwater Harvesting	Surface or underground storages capturing runoff from roof surfaces and consumed for indoor and outdoor purposes. The storage has a permanent storage volume and may have an air space above	Constant water usage (for example indoor demands) draws down storage volumes prior to rainfall events. A small pipe may link to the downstream stormwater network at the	Volume reduction processes reduce erosion of streams and reduces transport of urban pollutants.	Lot Site Neighbourhood Precinct

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
	the permanent storage for stormwater detention.	top level of the permanent storage. A second pipe at the top level of the air space caters for high level overflows (Figure 9.4.9).		
Bioretention Basin	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. Shallow storage over filter media. Experiences a cycle of wetting and drying.	A network of sub-soil drainage at the bottom of the filter media. Outlet pit and pipe culvert for flows that exceed the permeability of the filter. A spillway at the top level of the storage to pass flow in excess of the culvert capacity (Figure 9.4.10).	Sandy loam filter media with high permeability and suitable vegetation.	Site Neighbourhood
Constructed Wetland	A storage basin excavated into the ground surface and partially formed by embankment on downslope side. Normally wet with bathymetry designed to support healthy range of aquatic plants. Ephemeral wetlands are subject to a cycle of wetting and drying that replicate natural processes.	Outlet pit and pipe culvert. High flow bypass (directs high flows away from wetland area). A spillway at the top level of the storage to pass flow in excess of the culvert capacity (Figure 9.4.12).	Aquatic plants growing in a suitable soil substrate.	Precinct
Managed Aquifer Recharge	An infiltration zone, in the floor of a basin, with good permeable connectivity to the groundwater system or a gravel filled soakaway with aquifer access via a bore pipe.	A permeable soil layer in the floor of the basin with connectivity to an aquifer A spillway at the top level of the basin to pass flow in excess of the permeable layer (Figure 9.4.13).	Removal of stormwater volumes decreases erosion of streams and reduces transport of urban pollutants. Normally requires pre-treatment.	Neighbourhood Precinct
Infiltration System	An infiltration zone, in the floor of a drainage pit, swale, basin, trench or	A porous floor in the base of the structure with	Removed contaminants and volumes of	Lot Site

Common Description	Storage Basin	Outlet Structure	Treatment Processes[a]	Typical Catchment Scale[b]
	pavement with good permeable connectivity to the groundwater system. Overflow from a rain water tank passed into bio-retention, raingarden, gravel filled trench or soakaway, normally dry, or directly to a local aquifer	connectivity to deeper sub-soils. A spillway, pipe or channel at the top level of the structure to pass flow in excess of the permeable layer (Figure 9.4.14 and Figure 9.4.15).	stormwater from flows. This further reduces transport of pollutants.	Neighbourhood
Stormwater Harvest Pond	A large storage pond formed by excavation into the ground surface and possibly formed by embankment on downslope side.	A pump system to extract water for use. A spillway at the top level of the pond to pass flow in excess of demand.	Reduce runoff volumes diminishes erosion of streams and reduces transport of urban pollutants. Normally requires pre-treatment.	Neighbourhood Precinct

[a]Note those devices without treatment processes may still provide water treatment benefits due to the effects of temporary storage and/or harvesting of runoff.

[b]Scale definitions taken from Book 9, Chapter 6

4.4. Concept Design

4.4.1. Overview

Concept design is an important phase in the overall infrastructure delivery process. It provides early insight into the likely physical characteristics of a facility, and allows design integration with other nearby infrastructure including, for example, stormwater conveyance infrastructure, open space, roads and buildings. If an approval is required, then the concept design will form part of the evidence needed for a submission. A concept design will also be needed to establish a financial budget.

The following sub-sections outlines the concept design phase of a typical volume management facility. Four concept design tasks are described:

- Choosing the best location for the facility (Book 9, Chapter 4, Section 4)
- Choosing the best design solution, having regard to the design objectives and site variables (Book 9, Chapter 4, Section 4)
- Preliminary sizing and configuration (Book 9, Chapter 4, Section 4)
- Collaboration and integration with other relevant professional disciplines (Book 9, Chapter 4, Section 4)

While these tasks are presented in this sequence, the tasks should not necessarily be completed in this sequence nor in a linear fashion. There is often a need for iteration and concurrent completion of design tasks. For example, collaboration and a preliminary sizing may be required to inform the selection of a preferred location. Once the preferred location is determined, the preliminary sizing must be updated.

Concept design can only commence once an overall catchment strategy has been established (refer to Book 9, Chapter 3) and design objectives determined (refer to Book 9, Chapter 4, Section 2). These foundational design aspects are assumed to have been resolved prior to implementing the following guidance. In particular a decision must be made as to the general position of the facility or strategy within the catchment. For example, it should be decided prior to commencing concept design whether the facility will be constructed to service a catchment comprising a single lot, a neighbourhood, or an urban precinct that is large in scale. With this overall constraint in mind, the following concept design tasks should be considered.

4.4.2. Choose a Location

The site chosen for a volume management facility is important to the success of the design. The site will have associated site variables, such as topography, soil types, catchment characteristics and groundwater characteristics. In some circumstances, the design may need to trade off some capabilities or require special features to completely respond to these site variables, and avoid constructability and long-term performance issues.

Where there is flexibility, it is best to choose a site that presents the smallest design challenge and meets the objectives for the project. The following discussion is intended to assist in this regard.

Topography

Volume management facilities may be located on or adjacent to the lowest point in the catchment to be serviced. This maximises the catchment area to be managed. Similarly the location may also need to capture flows from upstream conveyance infrastructure. If the site cannot easily service the relevant upstream sub-catchment then performance against the design objectives may be compromised.

While catchment hydrology (refer to Book 9, Chapter 6) is an integral part of the design process, even before such calculations are undertaken, the concept design should be informed by a general appreciation of the catchment draining to the proposed facility. As a minimum, the size of the catchment area draining to the facility needs to be determined so that preliminary sizing can be undertaken.

The location chosen may need to be adequately elevated (or able to be raised using an embankment), so that hydraulic performance of the outlet structure is not adversely influenced by backwater. This is a particular consideration for facilities that have treatment processes and vegetation or where the storage is intended to be well drained.

Areas in low-lying coastal districts must also consider the effects of high-tide and possible future changes to the tide level due to sea-level rise (refer to Book 4, Chapter 4 and Book 6, Chapter 5). Frequent backwater flooding from regional flood events should also be avoided, unless its impact can be assessed and proven acceptable.

The average ground slope in the location chosen should ideally be no steeper than 5%. Steeper sites are not precluded, however they will require more careful consideration of the type and shape of storage to avoid excessive earthworks. It may also introduce the need for vertical retaining wall elements which may be undesirable if they hamper access, introduce safety risks, increase maintenance and increase longer-term facility replacement and renewal costs.

Soils

Ideally the soils in the chosen location will be suitable for construction and sufficiently deep to avoid excavation into rock.

Where an embankment is to be formed, the soil properties should allow tight compaction in layers to form a cohesive matrix and stable slope within the range of 1 in 2 to 1 in 10.

The soils used to construct any embankments or spillways should also have a very low permeability, particularly where significant volumes of water are to be stored or where long-term water storage is intended. If the soil type is not suitable then other soil materials will need to be imported for blending or replacement, or other materials considered such as clay liners.

Sites with dispersive and acid sulphate soils will require a careful selection of storage solution. If unavoidable, then the design must include appropriate management measures.

Groundwater Characteristics

Where stormwater infiltration is one of the overall design objectives, the site selected must be underlain by geologic strata that allow this infiltration to occur. Long-term groundwater behaviour in the vicinity should also be profiled, and a site selected where the elevation of the infiltration zone is not substantially below normal groundwater levels.

If infiltration is not required or desired, then a site should be chosen where the groundwater profile is unlikely to intersect the storage profile. This will simplify construction and ensure the storage can be more easily drained.

The stream baseflow, flow regimes and runoff water quality characteristics will also be relevant where water quality improvement or stormwater harvesting or infiltration objectives are targeted.

The quality of the groundwater store should also be investigated and water quality criteria for infiltration will need to be observed in accordance with local guidelines and Australian and New Zealand Environment and Conservation Council (ANZECC, 2000).

Vegetation

The selected site should not require the damage or removal of valuable trees or large stands of native vegetation. If it is determined that this cannot be avoided then special approvals may be required and a flora and fauna specialist should be engaged to assist to provide advise the design team. An environmental offset planting may be necessary.

If the facility is intended to be vegetated then an appropriate depth and quality of surface soil is required to support healthy plant growth.

4.4.3. Choosing a Design Solution

A design solution should be selected that best targets the established objectives and provides an optimum response to the constraints and variables of the site. A listing of common design solutions is provided in Table 9.4.4.

This is a basic guide aimed to provide an indicative starting point for the inexperienced urban stormwater designer and should not be interpreted as a barrier to innovative strategies or a replacement for first principles analysis. Those with experience will recognise opportunities for hybrid solutions that have broader application. For example, a hybrid facility involving a detention (retarding) basin with managed aquifer recharge (retention) and stormwater harvesting (retention) may provide a more comprehensive design solution to a volume management problem and for protection of urban waterways.

It is noted that in Table 9.4.4 there are several solution and objective combinations that are flagged as “suitable with limitations”. This means that the solution may not always perform well with respect to the relevant objective, however it can in some circumstances. For example, a particular managed aquifer recharge facility may not normally provide control of peak discharge in large floods when the water levels in the aquifer are high. However, it may still afford some benefits in small floods and greater benefits if aquifer levels are low. Some further information about these possible limitations is provided in Book 9, Chapter 4, Section 5.

Table 9.4.4. Indicative Suitability of Common Volume Management Design Solutions

Solution	Control Peak Discharge	Improve Water Quality	Harvest or Infiltrate Stormwater
Detention (Retarding) Basin (refer Book 9, Chapter 4, Section 5)	Suitable	Not suitable	Not suitable
On-Site Detention (OSD) (refer Book 9, Chapter 4, Section 5)	Suitable	Not suitable	Not suitable
Rainwater Harvesting (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable
Bioretention Basin (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable with limitations
Constructed Wetland (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable	Suitable with limitations
Managed Aquifer Recharge (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable
Infiltration System (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable
Stormwater Harvest Pond (refer Book 9, Chapter 4, Section 5)	Suitable with limitations	Suitable with limitations	Suitable

4.4.4. Preliminary Sizing and Configuration

The approximate physical footprint of the structure must be understood to confirm the availability of sufficient space at the site. Where the surrounding infrastructure has yet to be planned, space requirements can be communicated early to other members of the design team.

The size of the structure is the first aspect to investigate. Ultimately the size of the structure is determined by detailed calculation and modelling, however in the very early stages of planning it may be possible to use simple hand calculations and 'rules of thumb'.

Preliminary sizing will depend on local rainfall conditions, climate patterns and performance criteria. A value is often selected based on prior experience with the design of other nearby facilities. For example, in the case of an infiltration measure, the estimated surface area can then be combined with length and width limitations to estimate the total requirement for land area at a preliminary level of accuracy.

The shape of the facility must then be considered. The shape of the facility will be largely governed by a combination of factors including:

- Minimising and balancing earthworks – to suit the site topography and drainage and minimise the volume of earthworks relative to the volume of runoff stored. At the same time have regard to the design of adjoining infrastructure such as stormwater conveyance, roads and buildings.

- Visual and landscape objectives – there may be visual and landscape objectives sought for the facility that might influence overall shape of the facility.

- Maintenance and safety objectives – Suitable allowance should be made for maintenance access and safe batter slopes.

- Achieving suitable length to width ratios – where the facility targets water quality improvement the length to width ratio must sit within a suitable range, typically between 3:1 and 10:1.

While determining the preliminary shape of the structure, consideration should also be given to the need for any vertical wall elements, the location of outlet structures and the position and alignment of any embankments.

4.4.5. Collaboration and Integration

The best integrated outcomes for an urban design project involving stormwater are only achieved when stormwater professionals are consulted at the very beginning.

The design of a volume management facility is a task best undertaken in close collaboration with the client representative, relevant stakeholders and the overall urban design team including:

- Urban Designers;

- Local authorities including Councils and government departments;

- Civil Engineers;

- Landscape Architects;
- Environmental Engineers, Geomorphologists and Ecologists; and
- Geotechnical Engineers.

This collaboration should occur early in the design process to minimise re-work and maximise the potential for integrated outcomes. For example good opportunities exist for co-location of volume management facilities within areas that also perform recreation, landscape and environmental functions.

Since the position of volume management facilities is often tightly controlled by site topography and hydraulic constraints, it is also important that the design is undertaken in conjunction with the overall bulk earthworks and stormwater conveyance solution to yield an overall efficient and low cost design.

4.4.6. Emergence of Volume Management Research

The use of volume management measures distributed throughout urban areas to assist in the management of peak discharges at the outlets of catchments has been the topic of emerging research and practical investigations since the 1990s by an increasing number of authors and practitioners (for example Joliffe (1997), Argue and Pezzaniti (2007), Argue and Pezzaniti (2009), Argue and Pezzaniti (2010), Argue and Pezzaniti (2012), Andoh and Declerck (1999), Coombes et al. (2000), Coombes et al. (2001), Coombes et al. (2002a), Coombes et al. (2015), van der Sterren et al. (2013), van der Sterren et al. (2014)).

More recently, investigations have also focused on understanding the performance of entire linked systems of water cycle management within urban catchments that can reveal the cumulative impacts of integrated or combined strategies that better represent real systems (Coombes et al., 2002b; Coombes, 2005; Walsh et al., 2012; Coombes and Barry, 2015). These issues are discussed in Book 9, Chapter 3. This body of research and practice has evolved since the previous version of ARR 1987 (Pilgrim, 1987) and represents significant new thinking in the stormwater industry.

Many authors have established that the use of volume management at a distributed scale may not be required to provide significant reductions in peak discharges at the property scale because reducing runoff volumes at the top of catchment provide substantial reductions in peak flows throughout catchments (for example: Herrmann and Schmida (1999), Andoh and Declerck (1999), Argue and Scott (2000), Vaes and Berlamont (2001)). Argue and Scott (2000) used a large catchment scale model to conclude that distributed peak discharge control (on-site detention) and volume management (rainwater harvesting) systems produce similar hydrographs at the catchment outlet. It was acknowledged that the peak discharges on a lot scale may be larger for volume management than for flow management. However, it was found for medium to large catchments that the cumulative effect of volume reductions obliterates the effect of peak discharges at individual sites. This indicates that the cumulative effects of distributed reductions in stormwater runoff volumes can be significant at a catchment scale due to the reduction in overall volume discharged to the catchment outlet (refer to Book 9, Chapter 3). These results are consistent with the basic elements of peak flows which are volume and time. Reducing either element must reduce peak flows within the catchment.

Coombes et al. (2001), Coombes et al. (2003) also found that at the lot scale the flow management (detention) systems reduced the peak discharge at the lot scale and volume management (rainwater harvesting) provided smaller changes in peak discharges at lot scale but significantly reduced the volumes of stormwater runoff which reduced peak discharges at the street and catchment scale. It was argued that flooding is a volume driven process and peak discharges at the lot scale had little or no bearing on the floods at a catchment scale. Use of first principles processes such as continuous simulation and detailed systems analysis rather than empirical assumptions (for example antecedent conditions associated with event based analysis) has also revealed that the shape of catchment hydrographs may be significantly altered by distributed and integrated solutions within catchments (for example; Coombes and Barry (2009), Coombes (2015)). van der Sterren (2012), Burns et al. (2013) and Coombes (2015) highlight the benefits of replacing the common design requirements with treatment trains on properties and throughout urban areas to manage peak discharges and flow regimes throughout and at the outlet of urban catchments.

4.4.7. Use of Computer Models

A coupled analysis of storage basin volume and outlet capacity is necessary in order to determine the most appropriate configuration for a facility. This analysis is usually iterative. Firstly, dimensions of the storage basin and outlet are estimated and tested by numerical calculation and then progressively adjusted to achieve the design objectives. This is normally undertaken using computer models that have been developed to assist with these calculations.

The design and analysis of these facilities must include the interactions with other stormwater management facilities and urban form in the catchment and catchment behaviours. The adopted modelling approach should also use rainfall time series and resolve full hydrographs of a total duration that is relevant to the objective being analysed. For peak discharge control, this may only be minutes or hours. For water quality improvement and stormwater harvesting applications, this may be years or decades. The model must have sufficient catchment resolution and detail to adequately represent the linked hydrologic processes in the catchment. Lumped models that simplify catchment representation and behaviours should be used with caution.

The modelling approach should allow different storm scenarios to be tested since the performance of a volume management facility may be highly sensitive to the selected storm characteristics and volumes. For example, volume management facilities will have a greater impact on peak discharges under conditions where the storm burst occurs in front of a storm, rather than under conditions when the storm burst occurs towards the back of a storm, when the detention storage is already partially full.

A designer may therefore need to consider using an ensemble of complete storms with a storm burst of around the critical duration or a storm burst only to determine the benchmark condition(s) (Phillips and Yu (2015); Book 9, Chapter 6). If a design approach adopts a storm burst only approach, then for a given Annual Exceedance Probability (AEP) the peak flows are assessed for a range of storm burst durations and the storm burst duration that gives the highest peak flow is adopted as the critical storm.

If a design approach adopts an ensemble of complete storms of a given AEP, then the designer will need to determine if the benchmark condition is to be based on the 50th percentile peak flow or on a different percentile of peak flow. Preliminary testing indicates that adopting the 50th percentile is a very good indicator of the results from more complex Monte-Carlo approaches in most circumstances. Ultimately, the decision of what percentile of peak flow to adopt can be informed through identifying the level of risk the community is willing to accept within the catchment.

Once a base model is established, which includes the proposed facility, the model should be capable of iterative changes to the dimensions of the storage and the outlet structure. Using a judgement driven and iterative approach, the model is used to determine an optimised configuration that results in the required hydrologic performance for the selected range of storms.

For more detailed guidance regarding the use of computer modelling in urban stormwater design refer to Book 9, Chapter 5 and Book 9, Chapter 6.

4.5. Detailed Design Considerations

This section provides introductory level detailed design guidance for each of the most common volume management facility types, as listed in Table 9.4.3. Furthermore comprehensive design guidance reflecting local design standards should be sought from the relevant local stormwater authority. References to some useful guidelines are provided in each of the following sections.

4.5.1. Detention Basins

Detention basins, also sometimes called retarding basins, are measures which temporarily store stormwater to reduce peak discharge. Outflows are typically controlled by a low-level pipe or culvert and a high-level overflow spillway as shown in Figure 9.4.5.

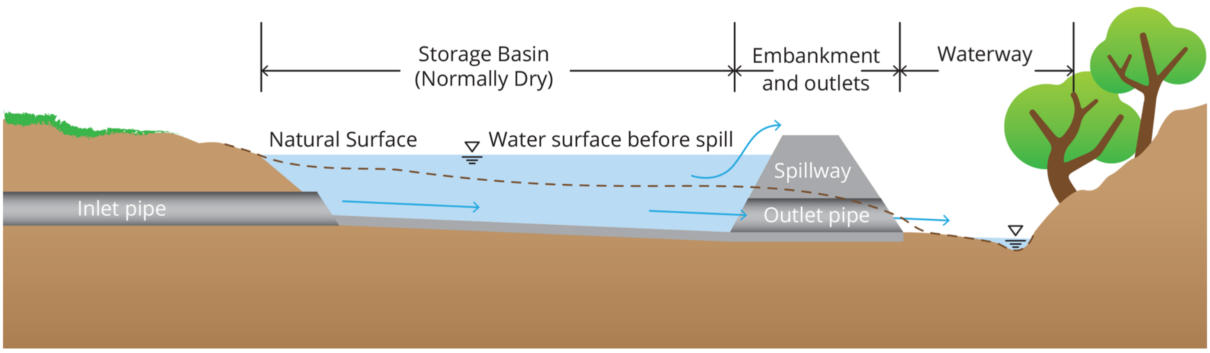

Figure 9.4.5. Detention Basin Typical Section

Detention basins can be designed to suit a range of catchment sizes. Community and regional scale basins may have considerable community benefits as areas for recreation and may be built around specific sizes and shapes of fields for sports such as football, netball and cricket. The sides of basins are usually sloping earth embankments, suitable for occasional spectator use. Basins used for passive recreation may include stands of trees (within the basin but not on any fill embankment), lawns and other vegetation.

Basins may be placed directly across a watercourse, or located off-stream, with flows in excess of a certain flow rate being diverted into them. They can be arranged in a widened section of drainage easement zoned both for recreation and drainage purposes.

Detention basins themselves are not suited to the improvement of water quality or harvesting and infiltration of stormwater. However other types of volume management facilities can be nested inside. For example a constructed wetland can be located in the floor of a large detention basin storage to also target water quality improvements.

Available Guidelines

There are many guidelines on community and regional detention including ACT Department of Urban Services (1998), Hobart City Council (2006), Department of Water, Western Australia (2007), Melbourne Water (2010), Queensland Department of Energy and Water Supply (2013). These guidelines can be readily used for designing and modelling detention systems, using the modelling and storm patterns as described in Book 9, Chapter 6.

Detailed Design Considerations

Flood Capacity

The final sizing of any 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 a partially full pipe flow or tailwater submergence occurs (Queensland Department of Energy and Water Supply, 2013). When located in-stream, the hydraulic modelling should also represent the stream conditions and the stream flows discharging through the basin in addition to the urban areas directed to it.

Large community and regional basins can be considered dams, as they can store significant volumes of stormwater, and therefore may pose a potential threat to communities residing downstream of a basin. As a result, the design must have regard to the ANCOLD (Australian National Committee on Large Dams, 2000) guidelines. A detailed risk assessment of a storm exceeding the Dam Crest Flood should be considered in the design of a detention basin within an urban area due to the potential severe consequences of the sudden failure of a basin on any urban development located on the floodplain downstream.

Detention basins should be designed with a flood capacity to convey appropriate extreme storms safely through the basin in accordance with the Hazard Category of the basin as defined by ANCOLD, as is the case for conventional dams.

An ‘Initial Assessment’, as defined by ANCOLD’s guidelines within the ANCOLD (2000a) should be undertaken for any proposed detention basin to determine the hazard category of the structure.

Depending on the findings of the ‘Initial Assessment’ a more detailed assessment (ANCOLD, 2000b) including a Dam Break analysis for both ‘flood failure’ and ‘sunny day’ scenarios may be required.

With increasing urbanisation there are now many catchments which contain a series of detention basins. Each basin within a catchment should be investigated not only individually but also collectively within the catchment, including all basins modelled as a whole (Melbourne Water, 2010).

In addition, two further issues should be considered:

- The consequences of one basin failure cascading downstream into lower basins should be evaluated; and
- The effect of long period releases from upper basins superimposing on flows through lower basins may require a revision of the basins’ operation throughout the catchment.

Embankment Design

The embankments of detention basins should be designed using appropriate stability analysis and geotechnical design practices. Particularly, appropriate foundation treatment should be specified. For earthen embankments suitable compaction levels, vegetation cover and stabilisation should be specified and protection provided to cater for cracking or dispersive soils. Impervious zones of an earthen embankment should take the form of a centrally located ‘core’ rather than an upstream face zone to reduce the effects of drying which may lead to cracking.

If the earth fill for any embankment is taken from borrow areas, these areas should be kept as far away from the embankment(s) as practicable. Should the borrow area penetrate any alluvial sand layers or lenses, the embankment’s cut-offs should be taken to at least one metre below the estimated depth of such sand layers/lenses at the detention basin floor.

Chimney intercept filters and filter/drainage blankets should be used for all high and extreme hazard category detention basins. Such filters may also be required for lower hazard category detention basins. All earthen embankments constructed from dispersion soils must have a chimney filter and downstream filter/drain (Melbourne Water, 2010).

Suggested basin freeboard requirements for a variety of basins are provided in Table 9.4.5.

Table 9.4.5. Detention Basin Freeboard Requirements (Adapted from Queensland Department of Energy and Water Supply (2013))

Situation	AEP	Maximum Depth or Level
Basin Formed by Road Embankment		
(a)	5%	Bottom of pavement box
(b)	2%	0.3 m below edge of shoulder
Basin Formed by Railway Embankment	2%	Underside of ballast
Large Basins with Separate High Level Spillway	1%	Embankment crest with freeboard ≥1% AEP storage depth and with minimum freeboard = 0.3 m [1]

External earthen embankment slopes and their protection should take into account long term maintenance of the structure. The side slopes of a grassed earthen embankment and basin storage area should not be steeper than 1(V):4(H) to prevent bank erosion and to facilitate maintenance and mowing.

The surfaces of an earthen embankment and overflow spillway must be protected against damage by scour. The degree of protection required is subject to the calculated flow velocity.

The following treatments are recommended as a guide (NSW Government, 2004):

- $V \leq 2$ m/s a dense well-knit turf cover using for example kikuyu;
- 2 m/s $< V <$ 7 m/s a dense well-knit turf cover incorporating a turf reinforcement system; and
- $V \geq 7$ m/s hard surfacing with concrete, riprap or similar.

Practical maintenance access should be provided to the full length of the embankment and any hydraulic structures passing through it.

Basin Floor

The floor of basin shall be designed with a suitable grade that provides positive drainage to the basin outlet and to prevent water logging. Detention basins may require underdrains to positively drain the bottom of the detention facility for ease of maintenance. If there are frequent trickle flows entering the basin then a low flow channel or pipe passing through the basin should be considered.

Primary Outlets

The key function of primary outlets is to release flows from a detention basin at the designed discharge rate. Some typical primary outlets are shown in Figure 9.4.6. Book 6 details how these outlets can be hydraulically designed.

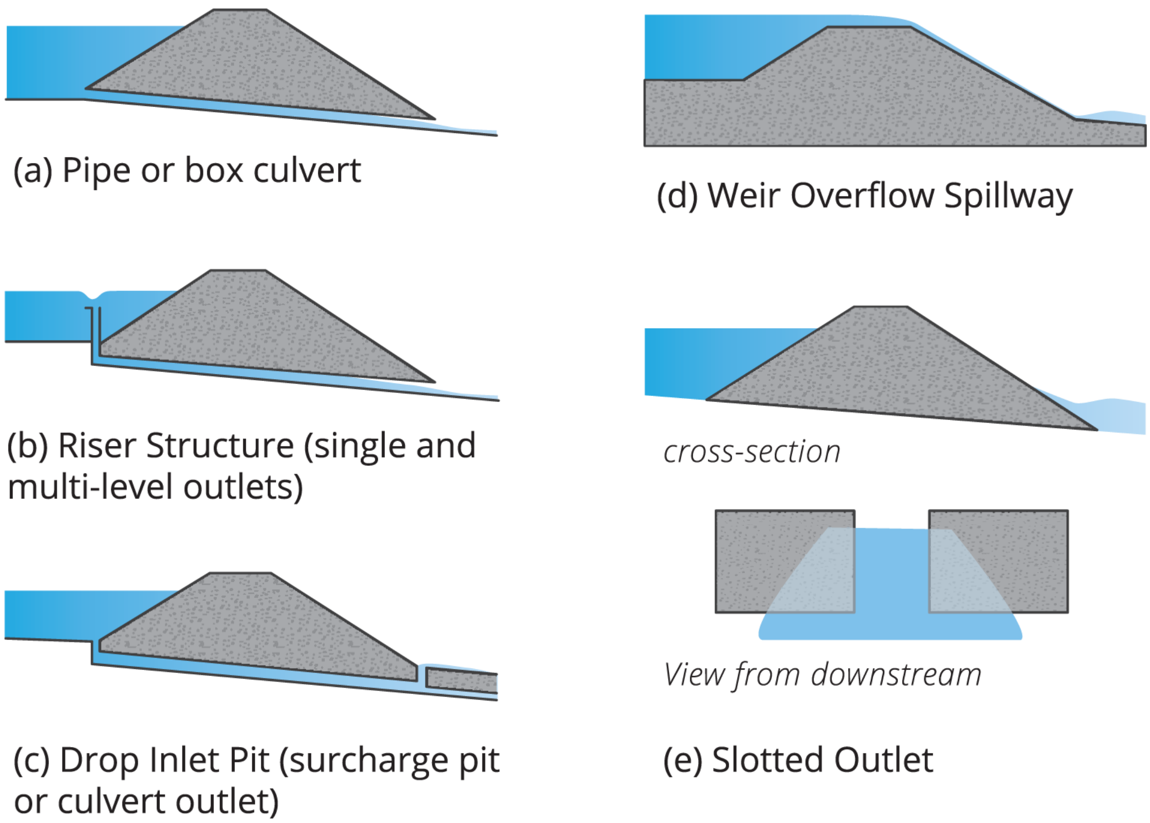

Figure 9.4.6. Typical Detention Basin Primary Outlets

Pipe or box culverts are often used as outlet structures for detention basin facilities. The design of these outlets can be for either single or multi-stage discharges. A single stage discharge system typically consists of a single culvert entrance system, which is not designed to carry emergency overflows (for example, when pipes are blocked). A multi-stage inlet typically involves the placement of a control structure at the inlet to the culvert. In particular, details on the hydraulics of rectangular weirs are given in Book 6, Chapter 3 and Book 9, Chapter 5.

Secondary Outlets

In general, the capacity of secondary outlets (typically spillways) should be based on the hazard rating of the structure as defined by the ANCOLD seven level rating system. The hazard rating defines the required 'Fall back' Design Flood. In some cases where the required 'Fall back' Design Flood is considered to be impractical, a full risk assessment of the basin may allow a lesser capacity spillway in line with ALARP (As Low As Reasonably Practicable) principles(Melbourne Water, 2010).

The design capacity of spillways should account for the possible reduced capacity of primary outlets which have the potential to become blocked during a major storm. The assessment of the possible blockage should be undertaken in accordance with the guidance provided in Book 6.

Recommendations for the design of outlet structures are provided by (ASCE, 1985) while the Design of Small Dams US Bureau of Reclamation (1987) provides procedures for the sizing and design of free overfall, ogee crest, side channel, labyrinth, chute, conduit, drop inlet (morning glory), baffled chute and culvert spillways.

Details on the hydraulics of rectangular weirs, sharp-crested rectangular weirs, broad-crested rectangular weirs, trapezoidal weirs, circular-crested weirs and compound weirs are provided in Book 6, Chapter 3.

4.5.2. On-site Detention

In many urban areas detention has been implemented, and in particular since 1975 the use of detention basins has been widespread in NSW (Institution of Engineers, Australia, 1985). However in urbanised areas the available sites for large detention basins (as described previously in Book 9, Chapter 4, Section 5) are limited or are fully utilised over time.

To avoid exacerbating what can be already substantial flooding problems in an urbanised catchment, planning and development controls are sometimes implemented at the lot scale to mitigate the impact of increased impervious surfaces. These are commonly described as On-Site Detention (OSD) as shown in Figure 9.4.7.

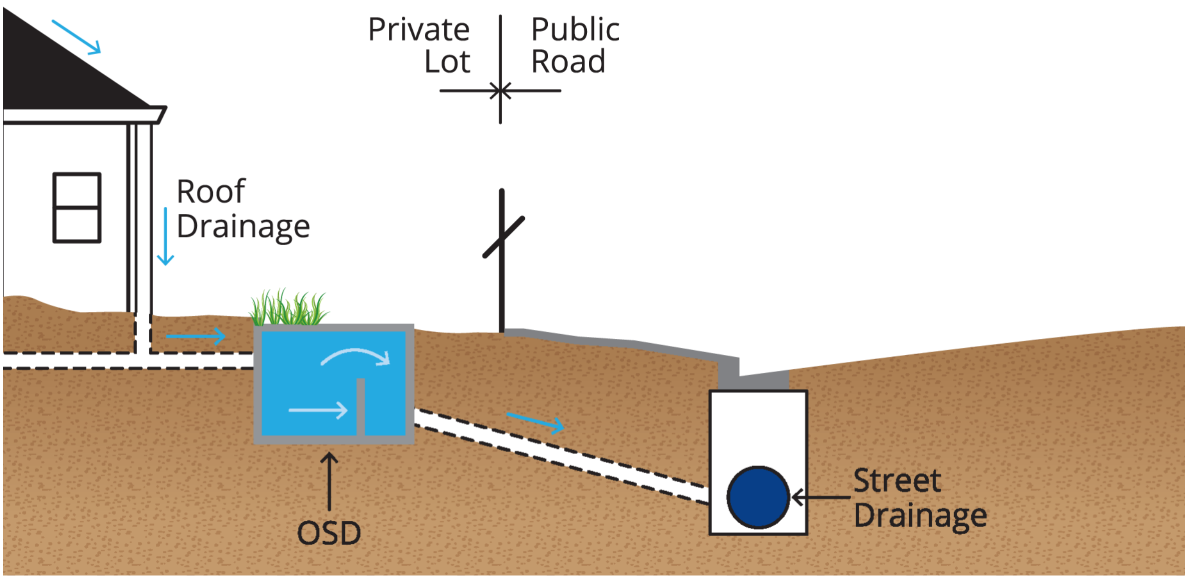

Figure 9.4.7. Typical Section Through a Below Ground On-Site Detention

In New South Wales, OSD was developed and first implemented by Ku-ring-gai Council, closely followed by Wollongong City Council (O'Loughlin et al., 1995). Since then many councils in Greater Sydney and elsewhere have implemented OSD systems. Other Councils outside of NSW have also adopted On-Site Detention, such as Hobart City Council (TAS), City of Casey (VIC), Manningham City Council (VIC), Melton Shire Council (VIC) and the City of Tea Tree (SA).

It is important to note that the imposition of OSD requirements at the lot scale is often done on the assumption that there are broader flood benefits at a catchment scale. However, in some cases there may be little or no catchment wide benefit from OSD, as the overall volume of runoff is not reduced, merely detained for a period of time. This effect is not always sufficient to influence catchment scale floods. OSD performance is also sensitive to the temporal pattern of rainfall.

Establishment of OSD policy therefore needs careful assessment at the outset using a catchment wide strategy to ensure the overall catchments to which the policy is intended to be applied are indeed suitable.

Available Guidelines

There are many guidelines on the sizing or design of OSD, for example Department of Irrigation and Drainage (2000), Upper Parramatta River Trust (2005), Hobart City Council (2006) and Derwent Estuary Program (2012). These guidelines can be readily used for designing OSD systems, using the modelling approaches outlined in Book 9, Chapter 6.

These documents can assist in the design of OSD systems, however, designers are encouraged to determine if the method identified in the guidelines are consistent and make suitable for using the contemporary flood estimation techniques identified in Book 9, Chapter 6 and the issues identified in Book 9, Chapter 3.

Detailed Design Considerations

Flood Capacity

Historically, the primary objective of OSD controls was to manage flooding in a 1% AEP event only. Further implementation and development on OSD has resulted in many authorities now requiring OSD systems to reduce the post-development flows to adopted benchmark peak discharges over a range of AEPs up to and including the 1% AEP event.

OSD discharge control requirements should be based on a catchment wide assessment. A catchment wide assessment has been typically downscaled to site control requirements, such as:

- Permissible Site Discharge (PSD) or Site Reference Discharge (SRD), which are defined as the maximum allowable discharge leaving the site (determined using catchment-based assessment of lot-based measures) with PSD giving a single discharge rates and SRD giving multiple discharge rates for different rainfall frequencies; and
- Site Storage Requirement (SSR), which is defined as the volume required for overall storage.

It should be noted that if the objective of OSD control is to manage flooding in a 1% AEP event only then typically only a single set of PSD and SSR values are defined. However, where authorities require OSD systems to perform over a range of AEPs a nest of frequency staged storages and outlets is required with multiple PSD and SSR values. An example of an OSD design is provided in Figure 9.4.8.

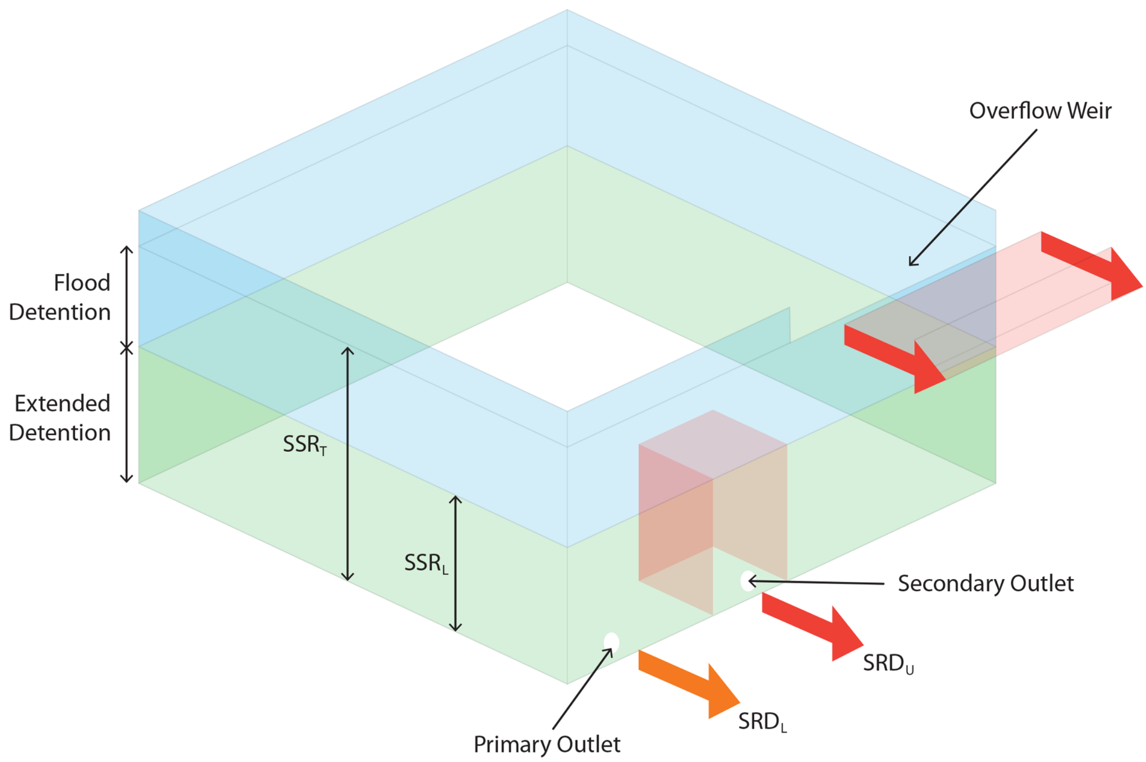

Figure 9.4.8. Frequency Staged Below Ground On-Site Detention System (adapted from Upper Parramatta River Trust (2005))

In the event that catchment wide assessments have not been conducted, one of the following site controls can be applied to enable the design of OSD systems:

1. The post-development flows of the subject site should be controlled to meet the pre-development flows for the site for a range of complete storms; or

2. Determine the capacity of the drainage system and divide by the area of lots that drain to the system. This gives an indicative estimate of the amount of the unit runoff (i.e. the PSD).

Either of these approaches are not as effective as designs based on a holistic catchment assessment, but may assist in the short term in managing nuisance flows in existing systems immediately downstream from sites.

On-Site Detention Types

OSD systems may comprise above-ground storage or underground storage or a combination of both. Above ground storage has advantages in terms of flexible configuration of site levels to achieve the required storage volume, capacity to incorporate retention through infiltration and pollutant removal landscaping features, reduced construction cost and easier maintenance. The advantages of underground storage are typically a reduced footprint in comparison to above ground storages and limitation of ponding on runoff on the surface. It is critical to select an appropriate storage type by considering the site layout, costs and effectiveness of OSD.

Above Ground On-Site Detention

OSD systems may comprise above ground storage or underground storage or a combination of both. It is critical to select an appropriate storage type by considering the site layout, costs and effectiveness of OSD.

Above ground storage has advantages in terms of flexible configuration of site levels to achieve the required storage volume, capacity to incorporate water quality treatment through infiltration and treatment media, low construction cost and potentially low maintenance.

The main types of above ground storages include landscaped storages, parking and paved storages, and rain water tanks with dedicated airspace for detention.

Where storage is not provided by a rain water tank the typical requirements listed in Table 9.4.6 should be considered.

Table 9.4.6. Above Ground On-Site Detention Storage Design Considerations

Design Aspect	Typical Considerations
Structural Adequacy	Design of surrounding embankments or retaining walls should consider structural and geotechnical aspects such as the need for reinforcement, compaction requirements and stable slopes. This includes when the storage is both full and empty.
Storage Configuration	Ponding depths shall not exceed the maximum storage depth requirements required by local standards. As an initial guide a maximum of 0.6 m is suggested for landscape areas with low pedestrian use (Department of Irrigation and Drainage, 2012). A Council may approve deeper ponding in individual cases where it is demonstrated that safety issues have been adequately addressed. For example, warning signs and or fencing should be installed where the depth exceeds0.6 m or adjacent to pedestrian traffic areas. Where ponding occurs in areas for recreational purposes (e.g. a playground) suitable velocity and depth should be selected to ensure the safety of children and the elderly. The storage volume should be increased by 20% to compensate for the potential loss of storage due to construction inaccuracies and the build-up of vegetation growth over time.
Floor Slope	The minimum ground surface slope should be 1.0%, while the desirable minimum surface slope is 1.5%.
Vegetation and Soils	Subsoil drainage around the outlet should be designed to prevent the ground becoming saturated during prolonged wet weather. Appropriate plant species for the vegetated areas should be selected that can withstand prolonged inundation and frequent wetting and drying. Any direct inflow point into a vegetated system (e.g. roof drainage or driveway runoff) should include a small energy dissipation device to reduce velocity and prevent erosion of the basin floor. Mulch utilised in the above ground storages should not be able to float and plants should be capable of withstanding frequent inundation as per the design depth and frequency.

Design Aspect	Typical Considerations
Overflow	An overflow should direct the flows to the legal point of discharge in a controlled and safe manner.
Freeboard	There should be freeboard above the stored flood level and adjacent habitable floor levels in accordance with local standards.
Safety and Access	Balustrades (fences) must comply with the Building Code of Australia (refer to Section D2.16 of the Code), while safety fences should comply with any legislated requirements for swimming pool fencing. Surface storages should be constructed so as to be easily accessible, with gentle side slopes permitting walking in or out. A maximum gradient of 1(V):4(H) (i.e. 1 vertical to 4 horizontal) should be required on at least one side to permit safe egress in an emergency. Where steep or vertical sides are unavoidable, due consideration should be given to safety aspects, such as the need for fencing or steps or a ladder, both when the storage is full and empty.
Frequency of Inundation	Frequent ponding can create maintenance problems or personal inconvenience to property owners. The initial 10%-20% of the storage should be provided in an area able to tolerate frequent inundation, e.g. a paved outdoor entertainment area, a permanent water feature, or a rock garden. Alternatively, a frequency staged storage approach should be adopted.

Below Ground Storages

Below ground storage tanks may be considered under the following conditions:

- Infeasible to construct above ground storages due to site constraints or topography; and
- Frequent inundation areas causing maintenance problems and inconvenience to the property owners or community members.

Below ground OSD storage tanks are usually made of reinforced concrete and can be pre-cast or cast in-situ to meet individual site requirements. When designing below ground tanks then typical design considerations include those listed below in Table 9.4.7.

Table 9.4.7. Below Ground On-Site Detention Storage Design Considerations (Department of Irrigation and Drainage, 2000), (Department of Irrigation and Drainage, 2012)

Design Aspect	Typical Considerations
Structural Adequacy	Storages must be structurally sound and be constructed from durable materials that are not subject to deterioration by corrosion or aggressive soil conditions. Tanks must be designed to withstand the expected live and dead loads on the structure, including external and internal hydrostatic loadings. Buoyancy should also be checked, especially for lightweight tanks, to ensure that the tank will not lift under high groundwater conditions. The soils and their impacts on concrete structure should be assessed to ensure that the correct structural specification is made.
Storage Configuration	Site geometry will dictate how the OSD system configured in plan. While a rectangular planform is typical and offers certain cost and maintenance

Design Aspect	Typical Considerations
	advantages site constraints will sometimes dictate a variation from a rectangular planform.
Floor Slope	To permit easy access to all parts of the storage for maintenance, the floor slope of the tank should be in the range 1% to 10%.
Ventilation	An important consideration for below ground storage systems is ventilation to minimise odour problems. Ventilation may be provided through the storage access opening(s) or by separate ventilation pipe risers and should be designed to prevent air from being trapped between the roof of the storage and the water surface.
Overflow	An overflow system must be provided to allow the storage to surcharge in a controlled manner if the capacity of the tank is exceeded due to a blockage of the outlet pipe or in the event of a storm with a magnitude greater than the design storm.
Freeboard	There should be freeboard above the stored flood level and adjacent habitable floor levels in accordance with local standards.
Safety and Access	A suitable amount of access hatches should be provided to enable contractors to readily adopt working in confined spaces techniques and equipment. Below-ground storage tanks should be provided with openings to allow access for maintenance. An access opening should be located directly above the outlet for cleaning when the storage tank is full and the outlet is clogged. A permanently installed ladder or step iron arrangement should be provided below each access opening if the storage is deeper than 1200 mm.
Frequency of Inundation	There should be no constraints on the frequency of inundation of the storage basin.

Below ground storage could be provided by modular system which could include one or more parallel rows of pipes connected by a common inlet and outlet chamber. The size of a modular unit is determined by the storage volume requirements, site constraints and the number of conduits or modular units which can be installed. When designing modular storage systems typical design considerations are similar to the design considerations for below ground storages as outlined above. Further guidance on conduit storage systems is provided by Department of Irrigation and Drainage (2000), Department of Irrigation and Drainage (2012).

Combined Above and Below Ground Storage

The designer of an OSD system faces a challenging task to achieve a balance between creating sufficient storages that are attractive and complementary to the architectural design, minimising personal inconvenience for property owners/residents and limiting costs.

These demands can be balanced by providing storage with a frequency staged storage approach. Under this approach, the design of OSD adopts combined storages multiple outlet approach, which can consist of an above ground storage and below ground storage. Underground storage is designed to store runoff for more frequent storm events, whilst the remainder of the required storage, up to the design storm event, is provided as above-ground storage.

This approach is likely to limit the depth of inundation and extent of area inundated in the above ground storage so that the greatest inconvenience to property owners or occupiers occurs very infrequently. It recognises that people are generally prepared to accept flooding which causes inconvenience as long as it does not cause a significant damage or does not happen too often. Conversely, the less the personal inconvenience the more frequently the inundation can be tolerated.

Outlet Structures

The outflows from OSD systems are typically controlled by orifices. Details on the hydraulics of orifices are discussed in Steward (1908); Medaugh and Johnson (1940); Lea (1942); Brater et al0. (1996); Bryant et al. (2008) and USBR (2001).

The orifice outlets should have a minimum internal diameter of at least 25 mm and need to be protected by a mesh screen to reduce the likelihood of the primary or secondary outlets being blocked by debris.

Upstream Drainage

The stormwater drainage system (including surface gradings, gutters, pipes, surface drains and overland flowpaths) for the property must:

- be able to collectively convey all runoff to the OSD system in a 1% AEP event with a duration equal to the time of concentration of the site; and
- ensure that the OSD storage is by-passed by all runoff from neighbouring properties and any part of the site not being directed to the OSD storage, for events up to and including the 1% AEP event.

Maintenance

While Councils are ultimately responsible for ensuring these systems are maintained through field inspections and enforcing the terms of any positive covenant covering OSD systems, the designer's task is to minimise the frequency of maintenance and make the job as simple as possible (Upper Parramatta River Trust, 2005).

4.5.3. Rain Water Harvesting

Rain water harvesting at the property or lot scale has been historically used for water supply throughout Australia and for the management of stormwater runoff in cities since the 1990s. Rain water harvesting systems that provide water supply for more frequent indoor uses can reduce catchment runoff (peak flow and volumes), improve urban stormwater quality and provide a supplementary water source. The effectiveness of distributed rain water harvesting solutions for management of stormwater within and at the outlet of catchments is dependent on the number of facilities, integration with other strategies in the catchment, density of development, climate regimes, and magnitude and frequency of demand for rain water supply.

Available Guidelines

Further guidance on rain water harvesting can be found in the following documents:

- Guidance on Use of Rain water Tanks (enHealth, 2012)
- Rain water Tank Design and Installation Handbook HB 230 refer to http://www.rainwaterharvesting.org.au

- Interim Rain water Harvesting System Guidelines (NSW Department of Planning and Environment, 2015)
- Design and Operation of Rain Water Harvesting Systems refer to http://urbanwatercyclesolutions.com

Detailed Design Considerations

Modelling

Rain water harvesting systems were historically designed and considered as a stand-alone facility. This process results in assumptions that rain water harvesting systems do not contribute to the control of quantity or quality of stormwater discharges from a site or throughout a catchment. It was often argued that rain water harvesting does not provide these benefits due to the uncertainty associated with antecedent conditions of storm events (how full is the storage prior to the design storm?). Methods to determine the antecedent conditions in rain water storages prior to storm events and for design rain water harvesting systems were developed and demonstrated by Coombes et al. (2001), Coombes et al. (2002b), Hardy et al. (2004), Coombes (2005), Coombes and Barry (2007), Coombes and Barry (2009) and Coombes (2009). This applied research and monitoring has provided a design process for rain water harvesting systems that requires continuous simulation at sub-daily intervals – preferably six minute time steps to determine the dynamic airspace (drawn down of storages by water demands). This process can also determine any detention airspace requirements of rain water storages prior to given storm events for allow integration with surrounding stormwater management strategies and use in catchment models reliant on design storms. These concepts have been applied by Phillips et al. (2005) for example and can be used to address concerns about antecedent conditions in linked stormwater designs. The design process for rain water harvesting systems has been enhanced by many authors including Burns et al. (2013) and van der Sterren (2012) (for example) to also account for flow regimes to protect urban waterways.

The rain water or stormwater harvesting system should be designed using continuous simulation (as identified in Coombes and Barry (2007)) and should consider the following:

- Rainfall at the site;
- Potential magnitude and frequency of rain water use and any rate of leakage from a leaky tank;
- Roof or catchment area draining to the tank;
- Size of inlet configuration, overflow and use (e.g. can the rate of flow be discharge into the tank and out of the storage without surcharging); and
- When underground – the backflow potential from downstream systems.

Upstream Drainage

The design of rain water harvesting systems can include gutter guards, leaf diverters, first flush devices and filter socks can limit the transfer of sediment and debris into rain water storages. Mesh screens on inlets, outlets and overflow devices will exclude animals and mosquitoes and other insects from entering storages therefore minimising the risk of harmful microorganisms and disease-carrying mosquitoes entering the tanks.

Runoff that is not collected in the storage and overflows from the storage should be diverted away from storage foundations, buildings or other structures (enHealth, 2012).

Storage Location

The location of the storage infrastructure will be dependent on aesthetic and space requirements for the chosen device. The tank must also be located where sufficient roof area can be drained by gravity to the top of the tank.

If the storage system is below-ground, site soil characteristics and surface flows will need to be considered. Surface flows should be prevented from entering the tank and soil conditions are particularly important if there are salinity or acid sulphate soil concerns which would affect the integrity of the structure (Department of Water, Western Australia, 2007).

Pumps and Connections

The tanks should be connected to internal domestic demands, typically toilet flushing. Appropriate flow rates need to be maintained for the occupant and therefore the majority of rain water supply systems will require a pump to distribute water to internal and external plumbing fixtures. A pump should be sized to balance the required flow and pressure for the intended uses of the rain water from the storage while minimising energy use. Generally flows of less than 30 L/min are suitable for most residential applications (NSW Department of Planning and Environment, 2015).

Local government or State Government policy requirements may exist in regards to pumps and connections.

Outlet

Runoff that is not collected in the storage and overflows should be diverted away from storage foundations, buildings or other structures (enHealth, 2012). This water should be directed into gardens, infiltration systems or the public stormwater management network. The overflow water should not be allowed to cause nuisance to neighbouring properties or to areas of public access.

Tanks with Dedicated Airspace

The increased uptake of rain water harvesting also creates an opportunity to adopt an integrated approach to lot scale stormwater management by designing the facility to control of peak discharge and harvest runoff volume. This approach may result in rain water tanks with three outlets, one for use of rain water (e.g. connected to selected indoor plumbing or garden irrigation) down the bottom of the tank, one for orifice discharge (i.e. the OSD outlet) half way up the tank, and the third outlet is an overflow at the top of the tank (as per Figure 9.4.9) as originally proposed by Coombes et al. (2001). The dedicated airspace above the minimum level outlet provides for additional attenuation of peak discharges.

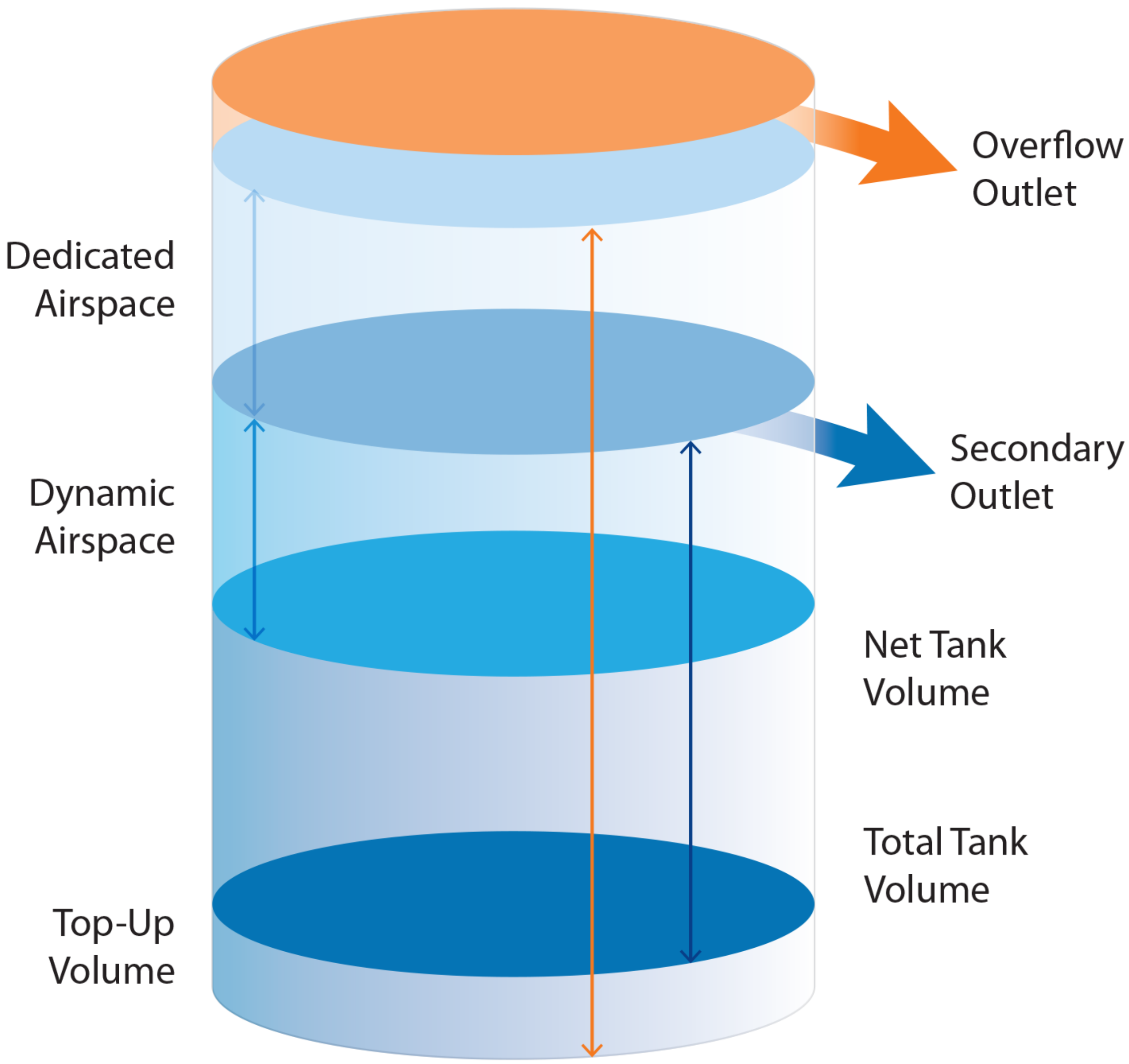

Figure 9.4.9. Rain Water Tank with Dedicated Air Space (adapted from Coombes et al. (2001))

4.5.4. Bioretention Basins

A bioretention basin is a shallow depression with a network of under-drainage and a soil-based filter media (refer to Figure 9.4.10). The filter media is vegetated with plants that tolerate periodic inundation. Stormwater is directed into the basin and percolates vertically through the soil and plant root zone providing water treatment. These facilities are sometimes also referred to as 'rain gardens'.

Bioretention basins primarily target water quality treatment objectives for small to medium catchments. In some circumstances it may also contribute to peak discharge control. Certain design types can also be used to promote the infiltration of stormwater into the groundwater system.

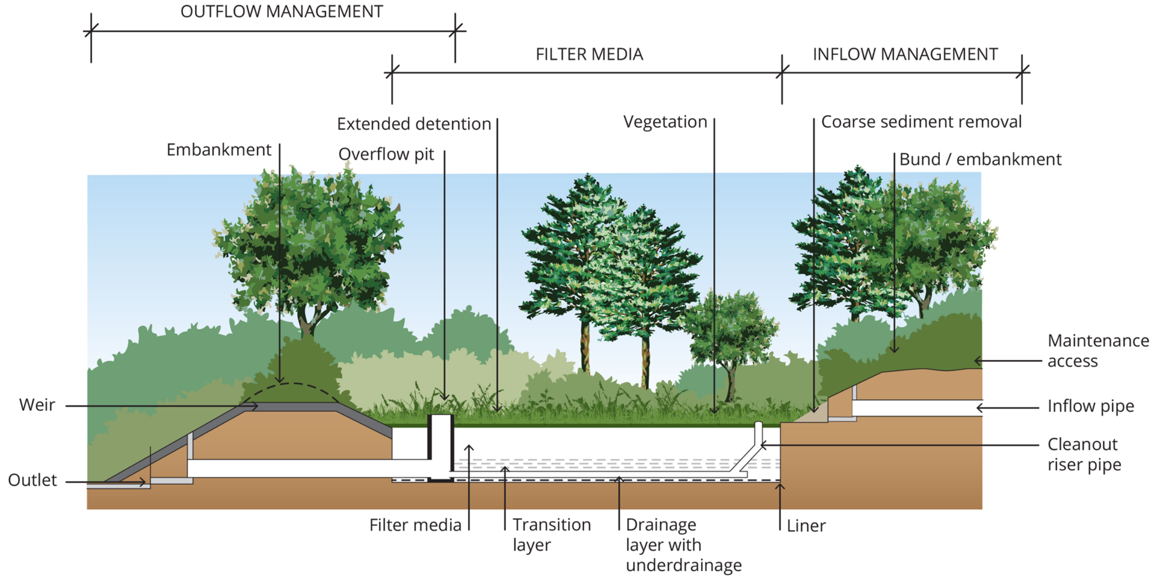

Figure 9.4.10. Components of a Bioretention Basin (Healthy Waterways by Design, 2014)

Available Guidelines

- Healthy Waterways by Design (2014) “Bioretention Technical Design Guidelines”
- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.
- Facility for Advanced Water Biofiltration (2009) “Guidelines for Filter Media in Biofiltration Systems”

Design Considerations

Basin Layout and Sizing

The core elements of a bioretention facility including a basin with filter media, an inlet structure and an outlet structure.

In practice a typical basin filter area requirement is between 1% and 2% of the catchment it serves. However the overall size of the basin will vary depending on its catchment and the treatment performance sought.

The shape of the basin is flexible but needs to facilitate even distribution of inflows across the filter media’s surface. The shape factor should therefore ideally approach a length to width ratio of 1 (i.e. square), though rectangular layouts are acceptable and common.

An indicative maximum catchment area constraint of about 10 hectares applies since areas greater than this normally produce trickle flows which can compromise the performance of the vegetation and filter media. It also becomes more difficult to evenly distribute inflows across a large filter area and manage scour velocities. This catchment area constraint will vary depending on local climate and soils.

Designs can be scaled down to lot scale and street scale sub-catchments. These facilities are sometimes referred to as bio-pods, rain gardens and tree pits.

The basin is designed to be frequently inundated for a short period of time, however the volume temporarily stored and the release rate are not normally effective at controlling peak discharge in large floods. Hybrid design opportunities exist where the bioretention basin is nested inside a larger detention basin facility to target peak flood discharge as well as water quality.

Filter Media and Layers

The floor of the basin comprises of a carefully blended soil filter media, minimum 400 mm depth, with a prescribed hydraulic conductivity of between 100 mm and 300 mm/hr. Over time the conductivity changes as the media settles and plants establish. The plant root zone enhances the water quality treatment performance of the filter and also helps to maintain an equilibrium level of hydraulic conductivity in the media.

Beneath the filter media are a sand transition layer and then a gravel drainage layer. The sand transition layer limits progressive migration of the filter media into the drainage layer. The drainage layer includes a network of slotted pipes that collect treated stormwater for discharge. This drainage layer can be designed as a saturated sump to sustain plant growth during extended dry seasons.

Bioretention basins are normally lined with low permeability clay or a plastic membrane. It is possible to design the system without a liner to encourage infiltration into the local groundwater table, however success with this approach will heavily depend on plant choice and climate.

Inlet Structures

The inlet structure receives flow from the upstream conveyance network. Typically the inlet comprises a small headwall pipe outlet, roadside kerb and gutter or an open channel swale. For large catchments a high-flow bypass is required to limit velocities within the basin and avoid scour of plants and filter media. For large catchments a coarse sediment capture zone (sometimes referred to as a ‘sediment forebay’) is also required to capture sediment and prevent smothering of the filter media. Regular clean-out of the coarse sediment capture zone is required. Maintenance access is therefore important.

Outlet Structures

The primary outlet is the filter media underdrainage system described previously. This is collected into an outlet pit before discharge into the downstream conveyance system. The secondary outlet normally comprises of an overflow pit or weir that is engaged once the hydraulic conductivity of the filter media is exceeded. The level of the weir is normally between 0.1 m and 0.3 m above the filter surface level. For larger systems a small armoured spillway or weir may also be provided to augment outlet capacity during a large storm.

The outlet discharge level should be sufficiently elevated above local backwater and tide levels to ensure the overall facility is free-draining. Emptying time for these systems can be critical and should be checked.

Vegetation and Landscape Integration

Bioretention basins should be thickly vegetated to encourage water treatment, enhance the long-term performance of the filter media and suppress weed growth. A wide range of plant species may be suitable, but those that tolerate dry conditions, can be periodically inundated and have fibrous root systems are preferred. Native sedges, rushes, grasses, tea tree, paper bark and swamp oak have all been found to perform well. The planting scheme that is chosen should blend with the surrounding landscape and habitat.

4.5.5. Constructed Wetlands

A constructed wetland is a system of water bodies that store water and sustain a range of aquatic macrophytes and semi-aquatic plants (Figure 9.4.11 and Figure 9.4.12). Stormwater is directed into the wetland and detained for a period of approximately 48 hours. During this time, physical, chemical and biological processes result in removal of water-borne pollutants.

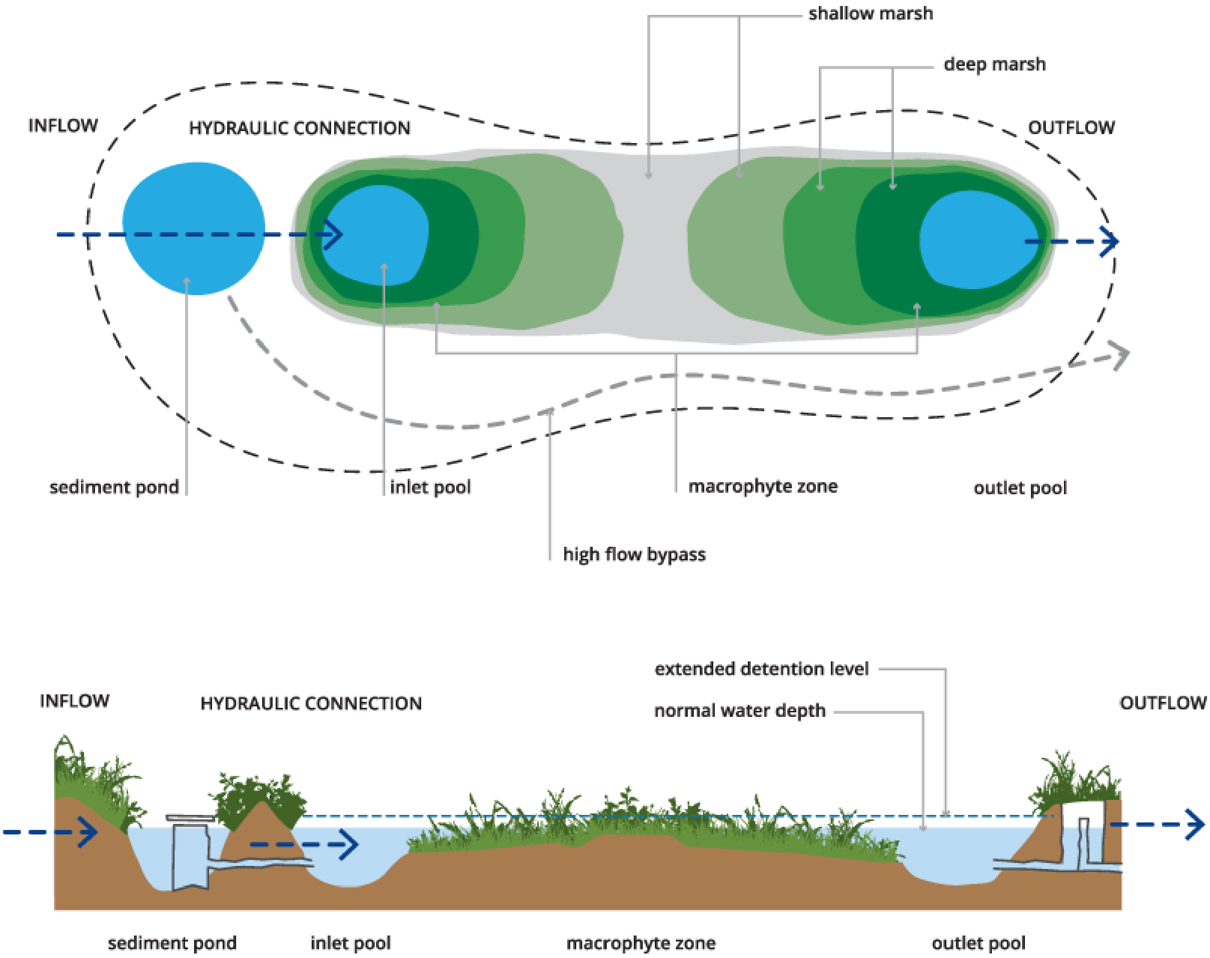

Figure 9.4.11. Schematic Layout of a Typical Constructed Wetland

Figure 9.4.12. Photo of a Typical Constructed Wetland (Source: Steve Roso)

A constructed wetland is most suitable for water quality improvement on catchments larger than approximately 10 hectares (indicative only and subject to local climate and design features). Subject to design and location it may also provide some peak discharge control. It is not directly suitable for harvesting or infiltration of stormwater as this can compromise the sustainability of vegetation. If this objective is sought a separate downstream pond facility should be provided.

Available Guidelines

- Water by Design (2017) "Wetland Technical Design Guidelines", Brisbane Queensland
- Melbourne Water (2016) Design, construction and establishment of constructed wetlands: design manual (Final Draft), Melbourne, Victoria
- Laurenson and Kuczera (1998) "The Constructed wetlands manual" Sydney, New South Wales

Design Considerations

Inlet Pond and High Flow Bypass

The inlet pond receives stormwater inflow from the upstream conveyance system. The depth and size of the pond should be sufficient to lower flow velocities and promote settling of course sediment particles. Regular clean-out of sediment from this area is required. Reliable maintenance access for machinery should therefore be considered in the design.

The inlet zone contains drainage structures that direct low flows out of the inlet pond and into a downstream wetland area.

During a storm the wetland area fills to a depth of about 0.5 m above the normal operating level. Once this threshold is reached, high flows are directed around the wetland area via a high flow bypass. This flow split is necessary to avoid re-suspension of sediment and plant damage in the wetland area.

Wetland Area

The wetland area is designed with a range of different ponding depths up to 1.5 m, perpendicular to the flowpath. These different depth zones promote a diversity of macrophytes and semi-aquatic plants and enhance the wetlands treatment capacity. The majority of the wetland area should comprise emergent macrophytes however deeper zones are important for diversity and to sustain the ecosystem during drier periods. The overall shape of the wetland should rest within a length to width ratio of between 3 and 10. Typically the total wetland area represents about 5% of the catchment area treated however this varies depending on the climate and treatment performance that is sought.

Outlet Structure

The stormwater that is temporarily held in the wetland after rain is progressively released via a restricted outlet. A typical residence time of 48 hours is sought, however this can vary depending on the site constraints and plant selection.

A secondary outlet is also required to limit the depth of submergence over the wetland.

Vegetation and Landscape Integration

Plant selection requires specialist input to design a planting scheme suited to the hydrologic regime and climate and therefore likely to establish and maintain a thick vegetation cover. The majority of the wetland footprint should be designed to support emergent macrophytes, however deeper zones are important for diversity and to sustain the ecosystem during drier periods. Regional biodiversity guidelines should be consulted for selection of appropriate plant species.

Opportunities should also be sought to integrate the wetland into passive open space recreation and/or local natural habitat.

The wetland should be well sealed with low permeability material to ensure water retention during dry periods. The bed of the wetland should also be lined with topsoil as a growth medium for the selected plants.

The initial establishment period is critical, careful maintenance is required including weeding and replacement of losses. Progressive flooding of the wetland is also needed to avoid drowning of small plants. Predation by birds is also sometimes a challenge that needs to be managed, particularly during the establishment phase.

4.5.6. Managed Aquifer Recharge

Managed Aquifer Recharge (MAR), also known as artificial recharge, is the infiltration or injection of water into an aquifer (Environmental Protection Authority, 2005) (refer to Figure 9.4.13). The water can be withdrawn at a later date, left in the aquifer for environmental benefits, such as maintaining water levels in wetlands, or used as a barrier to prevent saltwater or other contaminants from entering the aquifer (Department of Water, Western Australia, 2007).

MAR may be used as a means of managing water from a number of sources including stormwater. The MAR schemes can range in complexity and scale from the precinct scale, through local authority infiltration systems for road runoff and public open space irrigation bores, through to the regional scale, which involves infiltration or well injection of stormwater and provision of third pipe non-potable water supply for domestic use.

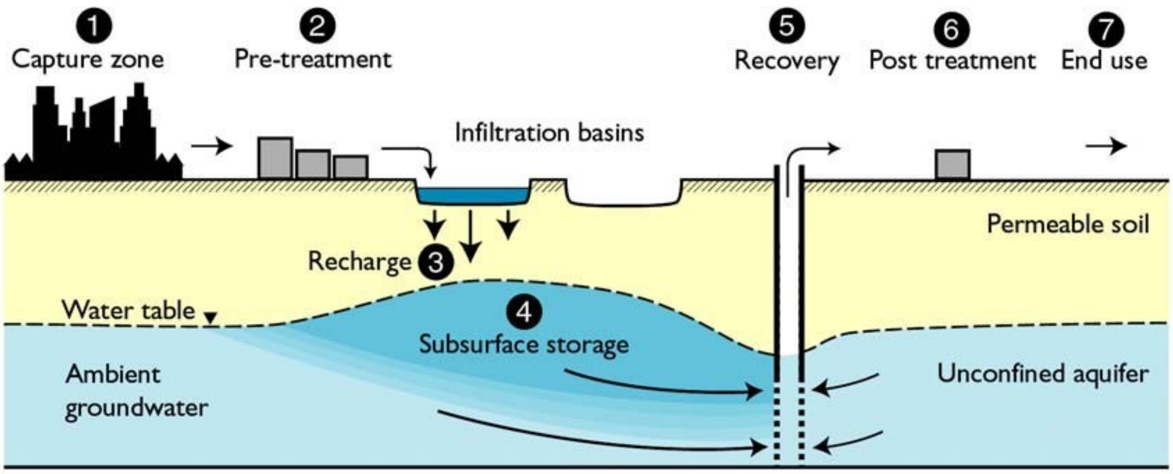

Figure 9.4.13. Example of a Managed Aquifer Recharge Scheme in an Unconfined Aquifer (Adapted from (Natural Resource Management Ministerial Council et al., 2009))

Available Guidelines

Further guidance on MAR can be found in the following documents:

- Melbourne Water (2005): WSUD: Engineering Procedures – Stormwater. Victorian Stormwater Committee, published CSIRO, Melbourne.

- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.

- Natural Resource Management Ministerial Council et al. (2009) “Australian Guidelines for Water Recycling - Managed Aquifer Recharge - National Water Quality Management Strategy - Document No 24, Canberra.

Design Considerations

System Components

As an example, a MAR scheme for infiltration of treated stormwater into a shallow aquifer could contain the following structural elements (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- soakwells, swales or infiltration basins used to detain runoff and preferentially recharge the superficial aquifer with harvested stormwater;

- an abstraction bore to recover water from the superficial aquifer for reuse;

- a reticulation system (in the case of irrigation reuse) (will require physical separation from potable water supply);
- a water quality treatment system for recovered water depending on its intended use (e.g. removal of iron staining minerals);
- systems to monitor groundwater levels and abstraction volumes; and
- systems to monitor the quality of groundwater and recovered water.

An MAR system may also incorporate the following additional elements (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- a diversion structure from a drain;
- a control unit to stop diversions when flows are outside an acceptable range of flows or quality;
- some form of treatment for stormwater prior to injection;
- a constructed wetland, detention pond, dam or tank, part or all of which acts as a temporary storage measure (and which may also be used as a buffer storage during recovery and reuse);
- a spill or overflow structure incorporated in the constructed wetland or detention storage;
- well(s) into which the water is injected (may require extraction equipment for periodic purging);
- an equipped well to recover water from the aquifer (injection and recovery may occur in the same well);
- a treatment system for recovered water (depending on its intended use);
- sampling ports on injection and recovery lines; and
- a control system to shut down recharge in the event of unfavourable conditions.

Site Suitability

Factors to consider in evaluating the suitability of an aquifer for a MAR scheme include (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- environmental values of the aquifer including ecosystem maintenance of caves, wetlands, phyreatophytic vegetation, surface water systems and human uses (irrigation, drinking water supply);
- adverse impacts on the environment and other aquifer users (e.g. reduced pumping pressure for nearby irrigators);
- an existing and/or future drinking water source area;
- sufficient permeability and storage within the receiving aquifer;
- depth of abstraction from the aquifer;

- existing allocation of the aquifer and groundwater resource;
- existing ambient groundwater quality and contaminant concentrations;
- loss of aquifer permeability and/or infiltration due to precipitation of minerals or clogging;
- possible damage to confining layers due to pressure increases;
- higher recovery efficiencies of porous media aquifers;
- aquifer mineral dissolution, if any; and
- potential for local aquitard collapse or distortion.

4.5.7. Infiltration Systems

Infiltration systems can come in a number of different forms, each having different size and geometry but all with a common purpose to promote infiltration of stormwater. They comprise of two main components; a storage basin and an infiltration zone. They are best suited to locations where natural soils have high permeability.

These facilities assist to manage stormwater volume through infiltration of stormwater that enters the groundwater system. They may also contribute to peak discharge control where rainfall intensities are low relative to the permeability of the infiltration zone. They are not intended to provide standalone water quality treatment and should ideally be accompanied by a treatment facility to prevent groundwater contamination.

Available Guidelines

Further guidance on infiltration systems can be found in the following documents:

- Healthy Waterways by Design (2006) “Water Sensitive Urban Design Technical Design Guidelines for South-east Queensland”
- Department of Water, Western Australia (2007) “Stormwater management manual for Western Australia”. Department of Water, WA, Perth, August.
- Argue and Pezzaniti (2012) “WSUD: basic procedures for ‘source control of stormwater – a Handbook for Australian practice”

Design Considerations

System Types

There are several different types of infiltration systems that are available to the designer, each of which suit different sites and applications. These are:

- Infiltration Trenches;
- Infiltration Basins;
- Soakage Well;
- Permeable Pavement; and
- Infiltration Swales.

Each of these is described further below.

Infiltration Trenches

An infiltration trench is a trench filled with gravel or other aggregate (e.g. blue metal), lined with geotextile and covered with topsoil. Often a perforated pipe runs across the media to ensure effective distribution of the stormwater along the system. Recharge storages can also be formed using modular plastic open crates or cells which can be laid in a trench or in rectangular formation. Such systems are typically 0.5 m to 1.5 m deep, surrounded by geotextile and covered with topsoil. Stormwater discharged into these systems is often pre-treated to reduce ongoing maintenance of such systems. Systems usually have an overflow pipe for larger storm events. There are a range of products which have various weight-bearing capacities so that the surface of the system can be used for parkland or vehicle parking areas. These systems can be combined to treat a large area (Department of Water, Western Australia, 2007).

Infiltration Basins (also Known as Retention Basins)

Community and regional infiltration basins are typically installed within public open space parklands. They can consist of a natural or constructed depression designed to capture and store the stormwater runoff on the surface prior to infiltrating into the soils. Basins are best suited to sandy soils and can be planted out with a range of vegetation to blend into the local landscape. The vegetation provides some water quality treatment and the root network assists in preventing the basin floor from clogging. Pre-treatment of inflows may be required in catchments with high sediment flows (Department of Water, Western Australia, 2007).

Soakage Wells

One method for infiltration of urban runoff into suitable soils is using soakage wells (for soils with hydraulic conductivity values $> 1 \times 10^{-6}$ m/s). These systems are used widely in Western Australia as an at-source stormwater management control, typically in small scale residential and commercial applications, or as road side entry pits at the beginning of a stormwater system. Soakage wells can be applied in retrofitting scenarios and existing road side entry pits/gullies can be retrofitted to perform an infiltration function (Department of Water, Western Australia, 2007).

Soakage wells consist of a vertical perforated liner with stormwater entering the system via an inlet pipe at the top of the device (refer Figure 9.4.14). The base of the soakwell is open or perforated and usually covered with a geotextile. Alternatively, pervious material, such as gravel or porous pavement, can be used to form the base of the soakwell. Where source water may have a high sediment load, there should be pre-treatment, such as filtering, as soakage wells are susceptible to clogging.

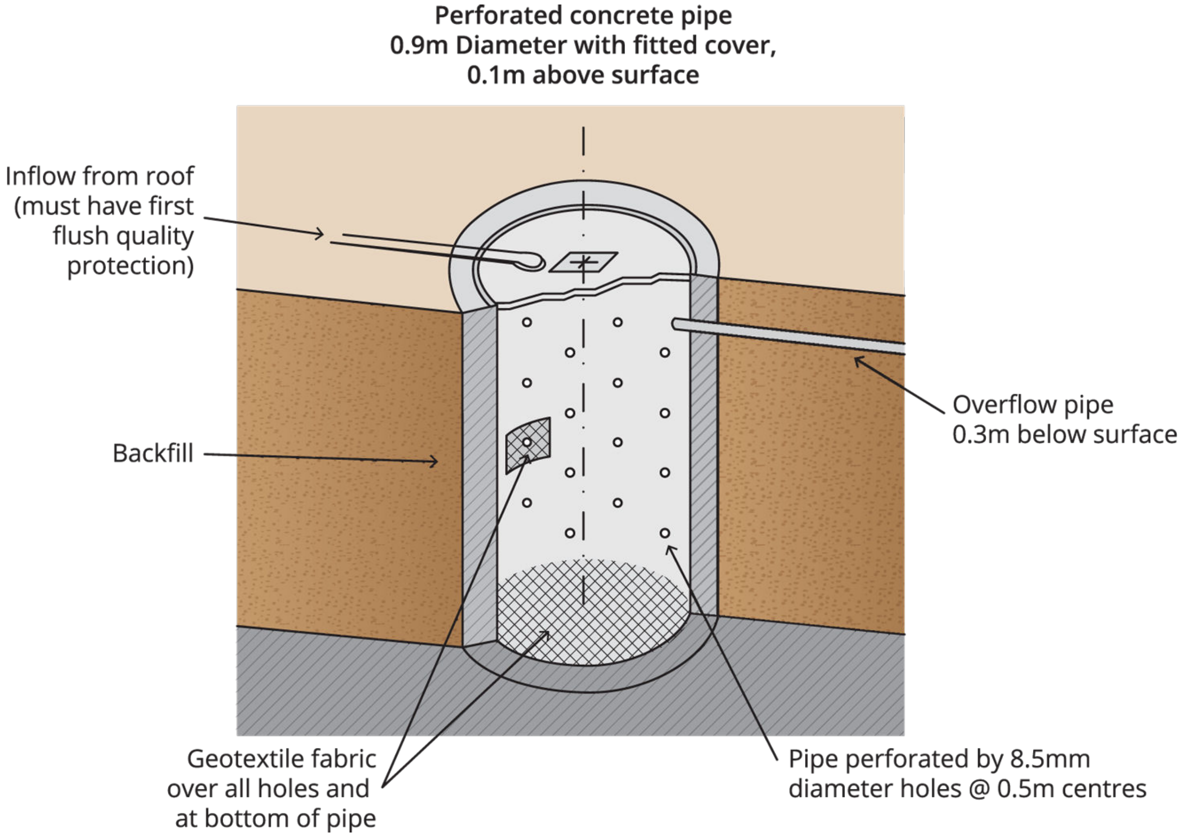

Figure 9.4.14. Leaky Well Infiltration System (adapted from Argue (2017))

Permeable Pavement

There are two types of pervious pavements that are effective in intercepting and diverting surface runoff into the host soil body:

- Permeable paving: concrete blocks incorporating slots or gravel-filled tubes providing (vertical) paths for surface flow to access gravel-filled (“leaky”) storages; and
- Porous paving: grassed surface integrated with a sandy-loam and plastic ring-matrix layer laid above a substructure of sand/gravel mix placed under optimum moisture content conditions.

The abstraction capabilities of permeable paving system slots and gravel-filled tubes can be as high as 4,000 mm/h when new – a performance which can show little deterioration over time where surface sediment loads are “light” or where the supply is pre-treated. Pre-treatment in a typical urban street context would require the insertion of a simple sediment trap (2.0 m^2 capacity) immediately upstream of the paving (requiring annual clean-out). The alternative to pre-treatment is regular (five-year intervals) cleaning of the paved surface.

Grassed surface paving shows infiltration capacity of, typically, at least 100 mm/h when new and, like permeable paving, shows little deterioration over time where supply sediment loads are relatively “light”. Porous paving is unsuited to the urban street context where permeable paving is used but can be relied upon for many decades of low maintenance service receiving runoff from, for example, a (conventional) paved carpark surface. “Low maintenance” in this context involves little more than regular mowing. The continued impressive performance of a porous paved surface is accounted for by the dynamic nature of the interaction – maintaining infiltration capacity - which takes place between the grass roots and the host soil.

Infiltration Swales

Infiltration swales are shallow grassed channels – typically 0.3 m to 0.5 m (maximum) deep, 5 m to 6 m wide in residential streets – with longitudinal slopes, preferably, less than 3%. They have wide application in stormwater retention systems for three main reasons:

- They can retain runoff through bed infiltration;
- They can be effective in retaining pollutants conveyed in stormwater (Breen et al., 1997; Lee et al., 2008) and
- They can fulfil a role in stormwater harvesting through soil moisture enhancement and, possibly, aquifer recharge and recovery.

The configuration of a typical infiltration swale in relation to a residential street carriageway is shown on Figure 9.4.15. This configuration includes a filter strip between the carriageway and the swale invert to provide pre-treatment and additional infiltration.

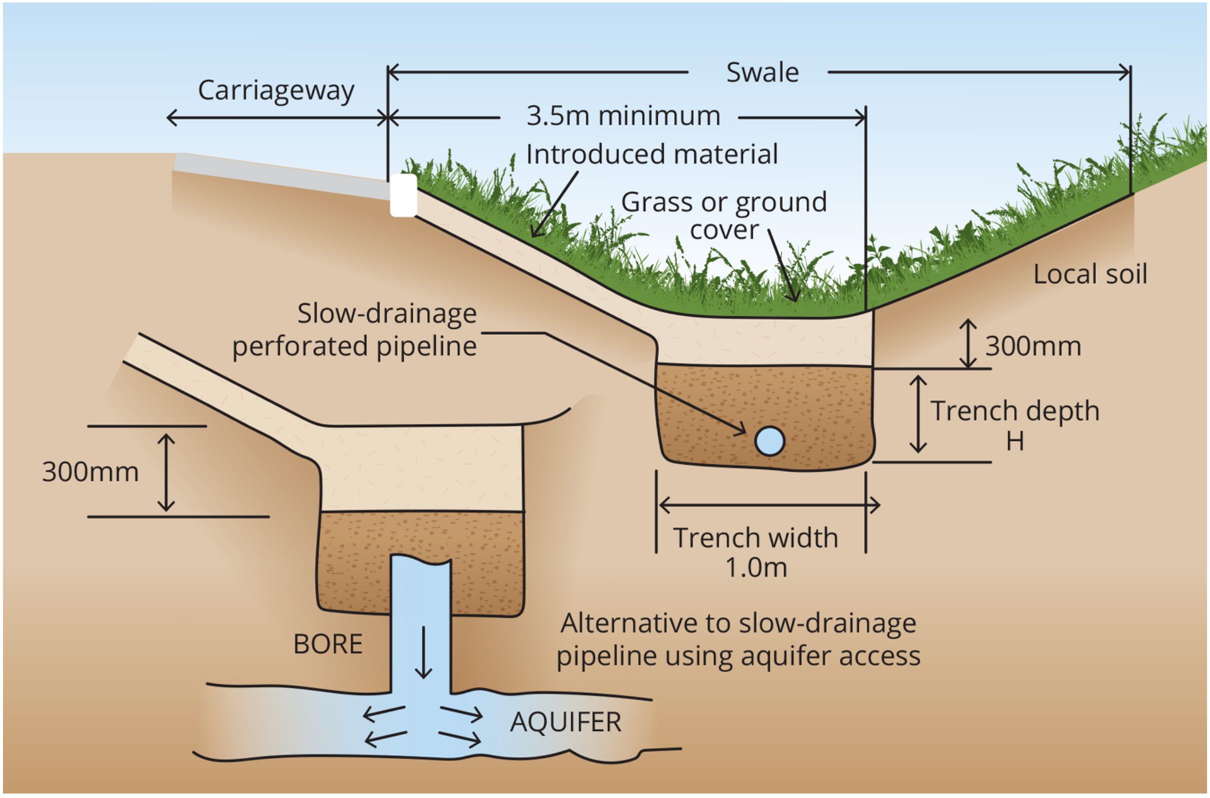

Figure 9.4.15. Main Components of an Infiltration Swale (with Filter Strip) (adapted (Argue, 2017),(Argue, 2013))

Swales only abstract flows up to a limit set by the infiltration capacity of the near-carriageway "filter strip" and channel bed. All exceedances above this capacity pass as open channel flow conveyed downstream within the boundaries of the swale. Another practice is to terminate such a swale in a "dry pond" perhaps in the vicinity of a major road intersection.

The process of abstraction is achieved through infiltration alone or by infiltration combined with sub-structure retention (gravel-filled trench or similar illustrated in Figure 9.4.7) with hydraulic disposal to aquifers (if available) or local waterways (slow-drainage) if necessary.

Site Selection

Due to their flexibility in shape, infiltration systems can be located in a relatively unusable portion of the site. However, design will need to consider clearance distances from adjacent building footings or boundaries to protect against cracking of walls and footings.

Identification of suitable sites for infiltration systems should also avoid steep terrain and areas of shallow soils overlying largely impervious rock (non-sedimentary rock and some sedimentary rock such as shale). An understanding of the seasonal and inter-annual variation of the groundwater table is also an essential element in the design of infiltration systems.

Soils

Soil types, surface geological conditions and groundwater levels determine the suitability of infiltration systems. Infiltration techniques can be implemented in a range of soil types, and are typically used in soils ranging from sands to clayey sands. While well-compacted sands are suitable these measures should not be installed in loose aeolian wind-blown sands.

Soils with lower hydraulic conductivities do not necessarily preclude the use of infiltration systems, but the size of the required system may typically become prohibitively large, or a more complex design approach may be required, such as including a slow drainage outlet system. Care should also be taken at sites with shallow soil overlying impervious bedrock, as the water stored on the bedrock will provide a stream of flow along the soil/rock interface (Department of Water, Western Australia, 2007).

Groundwater

The presence of a high groundwater table limits the potential use of infiltration systems in some areas, but does not preclude them. There are many instances of the successful application of infiltration basins on the Swan Coastal Plain where the basin base is located within 0.5 m of the average annual maximum groundwater level. The seasonal nature of local rainfall and variability in groundwater level should also be considered. Infiltration in areas with rising groundwater tables should be avoided where infiltration may accelerate the development of problems such as waterlogging and rising salinity (Department of Water, Western Australia, 2007).

Pre-treatment

In general, stormwater runoff should not be conveyed directly into an infiltration system, but the requirement for pre-treatment will depend on the catchment e.g. residential or industrial. Pre-treatment measures include the provision of leaf and roof litter guards along roof gutters, vegetated strips or swales, litter and sediment traps, sand filters and bioretention systems. To prevent infiltration systems from being clogged with sediment/litter during road and housing/building construction, temporary bunding or sediment controls need to be installed. It may also be necessary to achieve a prescribed water quality standard before stormwater can be discharged into groundwater (Department of Water, Western Australia, 2007).

Emptying Time

Emptying time is defined as the time taken to completely empty a storage associated with an infiltration system following the cessation of rainfall. This is an important design consideration as the computation procedures typically assume that the storage is empty prior to the commencement of the design storm event.

Ideally emptying time criteria should be ascertained by undertaking 'continuous simulation' modelling of a catchment (Argue, 2017) and should be conducted in accordance with Book 7 and combined with partial series analysis to determine the volume, frequency and rate of discharge from the site. In the absence of such assessments the emptying times for infiltration systems given in Table 9.4.8 are recommended in the interim.

Table 9.4.8. Interim Relationship between Annual Exceedance Proability and 'Emptying Time (Argue, 2017)

	EY			AEP (%)			
	1	0.5	0.2	10	5	2	1
Emptying Time (days)	0.5	1.0	1.5	2.0	2.5	3.0	3.5

4.5.8. Stormwater Harvest Ponds

A stormwater harvest pond comprises of a storage area to collect surface runoff for later extraction and use, often for irrigation. Ancillary infrastructure is also required for the pre and post treatment and distribution of this water.

A stormwater harvest pond is best suited to applications that target the harvesting and re-use of larger quantities of stormwater for non-potable use. Below a certain size threshold a pond may not be an economic way of storing water, in which case an alternative may be an underground tank.

A stormwater harvest pond does not directly target the improvement of water quality, however it can provide a minor contribution to this outcome in some circumstances where there is suitable irrigation demand. Similarly a stormwater harvest pond does not directly target the control of peak discharge but it may contribute to minor reductions in peak flow downstream for smaller storms.

Available Guidelines

- Healthy Waterways by Design (2009) "Stormwater Harvesting Guidelines (Draft)", Brisbane, Queensland

Design Considerations

Embankment Design

The guidance on embankment design given in Book 9, Chapter 4, Section 5 is also applicable to any dry ponds which are formed by embankments.

Configuration and Sizing

The configuration of a stormwater harvest pond is mostly influenced by physical site constraints and geotechnical limitations on batter slope. The shape of the pond does not directly affect its performance as a storage, but may affect the cost of civil construction. The most efficient shape in this regard approaches a circle or square.

The size of the pond relative to the estimated catchment yield is a balance between capital cost of construction and the reliability of supply. This sizing must be undertaken using a water balance of the site with realistic estimates of rainfall, runoff and demand.

Liners

A stormwater harvest pond requires a low permeability liner with freeboard above the normal maximum operating level. This can comprise of a non-dispersive compacted clay liner, or a synthetic membrane.

A well utilised stormwater harvest pond is not normally full and will experience significant fluctuations in water level. The liner may therefore need underdrainage to prevent excessive groundwater pressures developing on the outer wall of the membrane.

Treatment

Depending on the anticipated end-use of the stored water, the water extracted from the facility may require treatment to improve water quality to the required standard. This may involve filtration using a graded sand filter or similar.

Drainage Structures

A stormwater harvest pond requires a suitable inlet structure armoured against erosion and designed to accommodate potential inflows when the pond is fully drawn down.

The outlet structure typically comprises of an enclosed conduit or spillway, with invert set at the maximum operating level. The capacity of this spillway should be designed with the same level of consideration given to a detention basin spillway, with a capacity and freeboard matched to the level of accepted risk.

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Chapter 5. Stormwater Conveyance

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With contributions from the Book 9 editors (Peter Coombes and Steve Roso)

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5.1. Introduction

Stormwater conveyance combines hydrological and hydraulic methods to safely convey stormwater generated by rain falling on urban surfaces to an outlet. Analysis of conveyance infrastructure typically includes the hydrology of sub-catchments that transfer rainfall runoff to inlet structures feeding a network of other conveyance infrastructure including pipes, open channels, roadways and open space.

Conveyance infrastructure is one of the many tools available to the designer for urban stormwater management which is part of the process of managing the water cycle. For example a stormwater management strategy for an urban area will include a wide range of measures to manage stormwater runoff volumes and flow rates, for example on-site detention, bio-retention, rain water, stormwater harvesting and infiltration systems. These may alter the inflows to and the design of the stormwater conveyance network as shown in Figure 9.5.1. These volume management measures (as described in Book 9, Chapter 4) can operate at different scales such as source and neighbourhood controls that alter inputs to conveyance networks and regional controls that mitigate outflows from conveyance networks.

This chapter focuses on the design and analysis of stormwater conveyance networks.

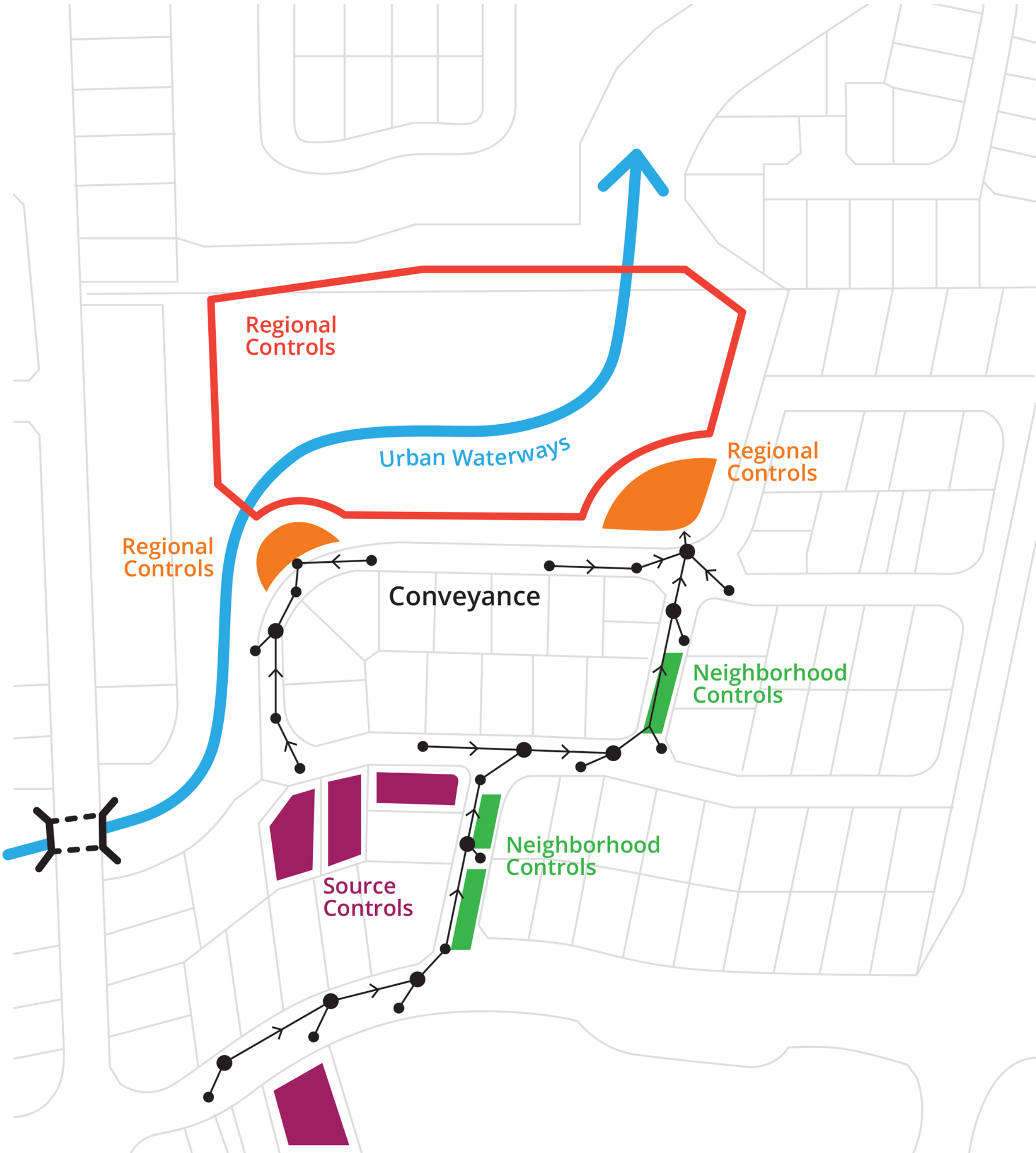


Figure 9.5.1. Stormwater Conveyance and Volume Management Within an Urban Stormwater Network

5.2. Design Philosophy and Objectives

Design of urban stormwater conveyance networks has been comprehensively addressed in guidelines within Australia and internationally. These guidelines include aspects of the design of stormwater infrastructure with different levels of detail that often concentrate on key areas of design focus (such as urban developments or main highways) or on problematic areas of concern that are specific to a locality or past events. It is generally the responsibility of the designer to select an adequate design procedure. However, the objectives and attributes of stormwater conveyance networks are often specified by the local approval authority. These authorities may base their guidance on other design specifications and guidelines such as Aus-Spec, Austroads, or the Victorian Infrastructure Design manual.

This section provides an overview of the philosophy and objectives for design of stormwater conveyance networks. The primary focus of this section is hydraulics and hydrology, and design safety requirements. Nevertheless, there are other important aspects that should be considered during the planning and design of conveyance networks. These include constructability, aesthetics, future maintenance, direct costs, long term economic factors, and the potential liability created by a conveyance network. The design should also account for the practicality of replacing conveyance infrastructure at the end of its design life.

A key hydraulic criterion is to define a conveyance network that restricts surface flows to safe limits. The primary design requirement is that stormwater depths should not be greater than a threshold value above the top of inlet pit or invert of a road gutter. This prevents inlet pits filling to the brim under design conditions which inhibits stormwater flows from entering conveyance networks. The threshold depth is typically set by the relevant approval authority and is in the order of 150 mm. Approval authorities also typically specify maximum velocities of surface flows and minimum velocities of flows in conveyance infrastructure.

In situations where surface flows are conveyed through public places, including footpaths, roads and public places, it is important to ensure that unacceptable hazards to people are not created (refer to Book 6, Chapter 7). Keeping the depth and velocity-depth attributes of surface flow within acceptable limits will minimise these hazards. When the primary purpose of a pathway is for conveyance of stormwater, it will usually be more efficient to convey flows in a dedicated watercourse that can accept higher velocity and depths of flows. These types of flow paths can be designed for dual uses (stormwater conveyance and public access) provided that the design ensures that people cannot be trapped by stormwater flows.

These limits are intended to ensure that stormwater conveyance networks operate at given levels of service without causing flooding of properties, nuisance or hazard to pedestrians and to traffic on streets. An approval authority typically specifies the design AEP of the minor and major storm events required for different land uses. Designs usually involve minor system capacity criteria for design of conveyance infrastructure and major system assumptions to ensure the urban area can safely cope with larger storm events as shown in Figure 9.5.2.

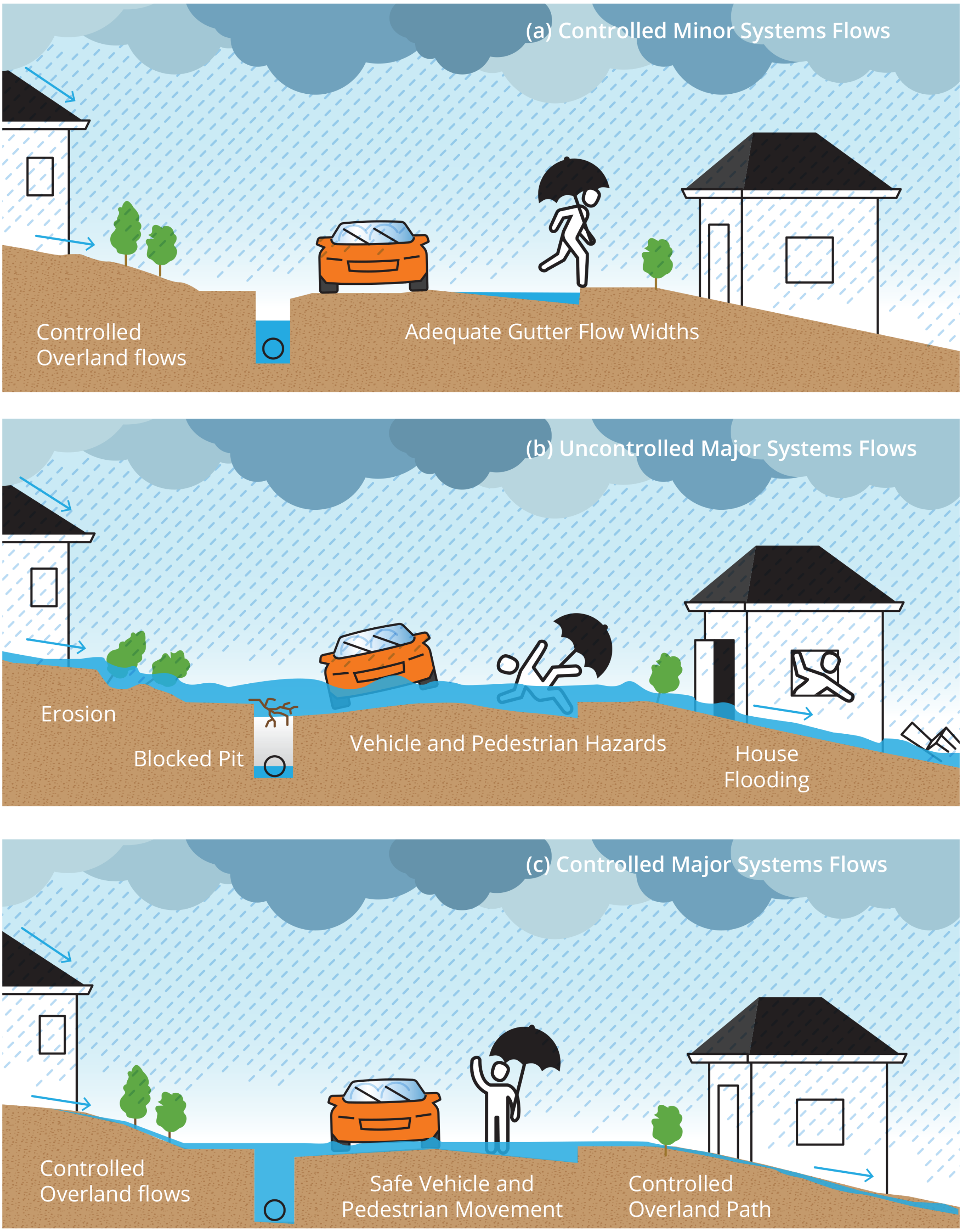


Figure 9.5.2. Minor and Major Concepts for Conveyance Networks

Figure 9.5.2 shows that the minor system is used to define the performance of the conveyance networks which include overland and bypass flows on roads, and performance of conveyance infrastructure (such as pipes and culverts). The major conveyance system includes the road profile and overland flow paths, and aims to ensure the safety of pedestrian and vehicle traffic whilst avoiding property damage and risk to life. In the absence of guidance from a consent authority, the design AEP storm events are selected to reflect the importance of a facility or urban area and the consequences of failure. Some examples are:

- Roof drainage systems: 5% AEP to 1% AEP;
- Conduit drainage systems through lots or sites: 0.5 EY to 1% AEP, depending on consequences of failure;
- Conveyance networks in streets: 0.5 EY to 5% AEP for minor flows, 2% AEP or 1% AEP for major flows (refer to Book 9, Chapter 3) (note that the street profile is part of the major conveyance network);
- Trunk conveyance networks: 1% AEP or higher, with checks on effects created by PMP storm events;
- Stormwater quality and sediment control devices: 4 EY to 1 EY but may address the full spectrum of rainfall frequencies (refer to Book 9, Chapter 3);
- On-site detention (refer to Book 9, Chapter 4): the requirements vary but should aim to improve the performance of stormwater management scheme at a sub-catchment scale; and
- Large detention basins (refer to Book 9, Chapter 4 and Book 9, Chapter 6) that may endanger lives if failure occurs: 1% AEP with checks using probable maximum precipitation (PMP) storm events.

Both design and analysis processes involve modelling the operation of a conveyance network that is subject to critical rare storm events that produce maximum flow rates for the selected AEP events. The selection of critical storm events will involve finding storm durations or particular storm patterns within ensembles or continuous sequences of rainfall that create maximum outputs for a particular location. Typically, the design of a conveyance network is shaped and sized to cater for critical storms for selected AEP events. This approach recognises that:

- It is not practical or economically feasible to design conveyance networks to be free of failure for all events. An attempt to do this would result in very large and expensive conveyance networks that would occupy a considerable land space. This would impact on the optimum provision other infrastructure services, such as water pipes and electricity conduits;
- Failures can occur in response to rare or extreme storm events or other factors such as blockages due to poor maintenance, and exacerbating circumstances such as high tide levels in coastal areas;
- A risk management approach should be adopted that accepts controlled failure;
- Ideally the acceptable level of risk should be set by community values and economic analysis, and;
- The effects of potentially rare failures should be limited by providing a ‘fail safe’ system that does not fail disastrously.

Analysis techniques should include sensitivity checks to ensure that damage and risks to lives due to failures are limited. Some failures of the network and overflows can be expected during major storm events, as shown in Figure 9.5.2, but the network should operate without causing safety hazards or large-scale property damage.

5.3. Conveyance Networks

The design or analysis process for conveyance networks requires observation of the real world situation; definition of the problem and objectives of the design; development of a conceptual model of real world behaviours; calibration using observed data; and predictions or design. A conceptual model of a conveyance network involves hydrological and hydraulic modelling including components such as inlet pits, pipes, open channels, roadways and storages. A general overview of the modelling and design process is presented in Figure 9.5.3.

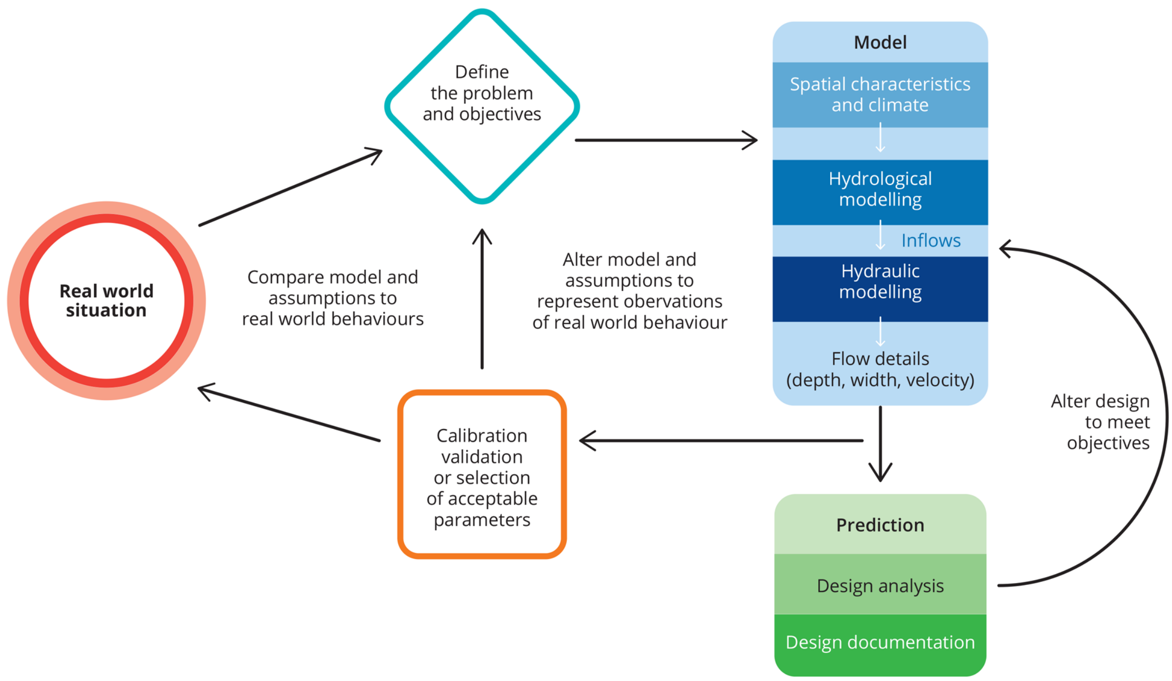


Figure 9.5.3. The Stormwater Design Process

Figure 9.5.3 shows that the stormwater design process includes hydrological modelling of rainfall runoff from urban surfaces to generate inputs to hydraulic modelling of the conveyance network. This process usually incorporates a hydrological model than translates design or real rainfall patterns into design flow rates and volumes of stormwater arriving at inlet structures within a conveyance network. A hydraulic model then converts these inflows into flow characteristics (depths, elevations, widths, velocities, and volumes) throughout the network. The design analysis then determines attributes of the conveyance infrastructure, including pipe diameters and invert levels. The steps in the conveyance design process include:

1. Define the real world situation to be modelled. This will include land use, demographics, topography, urban form, local climate, upstream and downstream conditions, and location within a river basin or waterway catchment.

2. Determine the objectives and design standards that should apply to the drainage network.

3. Locate any available rainfall runoff data that can be used to calibrate models used to design the drainage network or collate the most appropriate parameters for the catchment.

4. Choose the rainfall inputs, hydrological and hydraulic modelling methods for design or analysis:

   a. Rainfall inputs may be design storm temporal patterns of storm bursts or full volume storms, ensembles of peak burst or full storms, or long sequences of real rainfall (refer to Book 2 and ARR Data Hub).

   b. The hydrology and hydraulic models may be hand calculations but will typically be some form of computer model (refer to Book 5 and Book 7).

5. Analyse land uses, road and open space networks, and topography to develop the connectivity stormwater runoff processes throughout the catchment. This includes gathering information such as:

   a. survey and information defining topography;

   b. geotechnical and soil information;

   c. plans of the development or facility to be designed; and

   d. identifying constraints, such as easements and external drainage networks.

6. Define a model network of sub-catchments and drainage infrastructure that is an acceptable approximation of the real system.

7. Using topography, rainfall, land uses, the spatial location of other urban infrastructure and knowledge of the capacity of various drainage inlet structures, define the spacing of nodes in the conveyance network and the routing processes. The routing processes can include gutter flows, overland flows, bypass flows and pipe, culvert or channel flows. The routing processes can include gutter flows, overland flows, bypass flows and pipe or culvert or channel flows.

8. Calibrate or validate the hydrology and hydraulics of the existing catchment to any gauged data or nearby flood frequency information or accepted parameters for the area.

9. Use the model to design the capacity and spacing in inlet structures, and to size the conveyance infrastructure. This design process will be guided by the objectives and design standards that are applied to the project at Step 2. This process includes:

   a. definition of a trial layout of a drainage system made up of inlets, pipes, open channels, and storages; and

   b. using a model to define the sizes and locations of components.

10. Determine the adequacy and safety of the design for all relevant storm events.

11. Prepare plans, specifications and design reports and provide essential instructions on how to build the conveyance network.

1 Review the design, obtain approval from the required authorities and proceed with
2. construction or implementation.

Urban stormwater conveyance networks are usually a dendritic or tree-like structure that transports stormwater by gravity. Stormwater runoff is collected using inlet structures (pits) in different branches that converge at junctions along main lines and flow toward an outlet. Inlets structures located at the top of and along network branches:

- admit stormwater runoff into the conveyance infrastructure;
- provide locations where pipe diameters and directions can change;
- provide access for inspection and maintenance; and
- provide overflow points (if necessary).

Examples of underground pipe conveyance networks used in New South Wales and Queensland are provided in Figure 9.5.4.

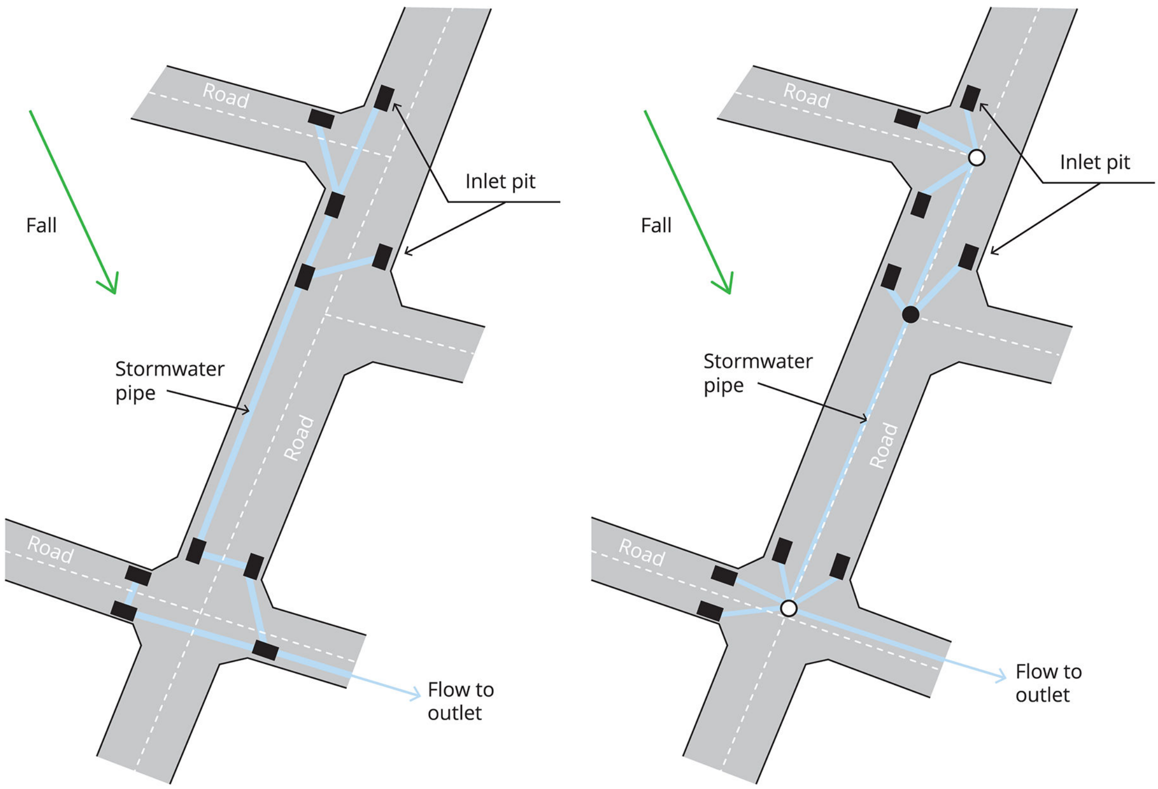


Figure 9.5.4. Examples of Configurations of Conveyance Networks in New South Wales (Left) and Queensland (Right)

Figure 9.5.4 shows different configurations of conveyance networks used in New South Wales and Queensland which highlights that a range of configurations are favoured across different jurisdictions. For example, in some Queensland jurisdictions, pipes are located under road centrelines and manholes at junctions in the conveyance network are used as collectors from inlet pits. Differences in terminology also occur across jurisdictions. For example, in New South Wales 'kerb and gutter' is used, while in Victoria and Queensland the term 'kerb and channel' is employed.

In some cases, maintenance holes, junctions or junction boxes (pits) are provided as nodes linking branches in the conveyance network. Other pits that are intended to overflow are called surcharge pits, overflow pits, or ‘bubble up’ pits. In established urban areas, looped networks may occur where additional pipes are added to provide additional conveyance capacity which can change the behaviour of the original conveyance network.

Conveyance infrastructure (for example: pipes, culverts, channels, and swales) are, mostly, constructed as straight sections with constant slope. Pipe and culvert conveyance infrastructure are available in standard dimensions supplied by the manufacturers. For example, the diameters of PVC pipes range from 90 mm to about 600 mm, and the diameters of reinforced concrete pipes start at 225 mm and increase to over 2 metres. Road authorities usually specify a minimum pipe diameter of 300 mm to 375 mm within road reserves to improve maintenance outcomes.

It is vital that conveyance networks include overland flow paths to control major stormwater runoff events. These overland flow paths should be within road profiles or through open space and pedestrian pathways. Flow paths through private property should be provided as a last resort and will require an easement (a legal instrument providing a right to drain stormwater through a property and permitting authorities to enter the site for maintenance). Overland flows directed through private property can create hazards and inhibit the development and value of the property as the required easement cannot be blocked or built upon.

Conveyance infrastructures (pipes) are designed to limit surface flows on roads to avoid nuisance to pedestrians and motorists. This process incorporates the design of roads including profiles of high locations (most often along road centrelines) and low locations (most often the inverts of gutters). The trapped low points in road networks require the provision of sag pits which will usually inform the required network of conveyance infrastructure (pipes) that can be realised by ‘joining up the dots’ between pits as shown in Figure 9.5.5.

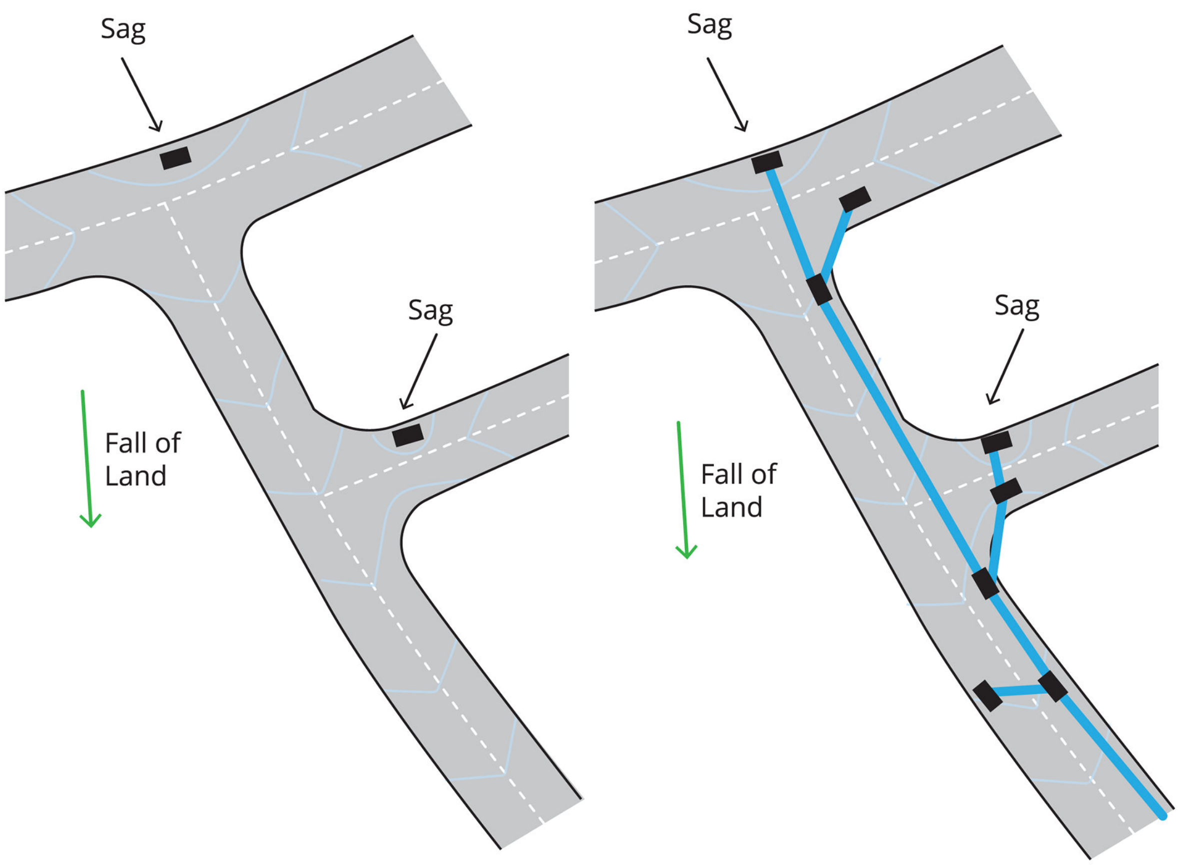


Figure 9.5.5. A Typical Configuration of a Conveyance Network

The configuration of an urban drainage network is demonstrated in a simple example of a single street in Figure 9.5.6.

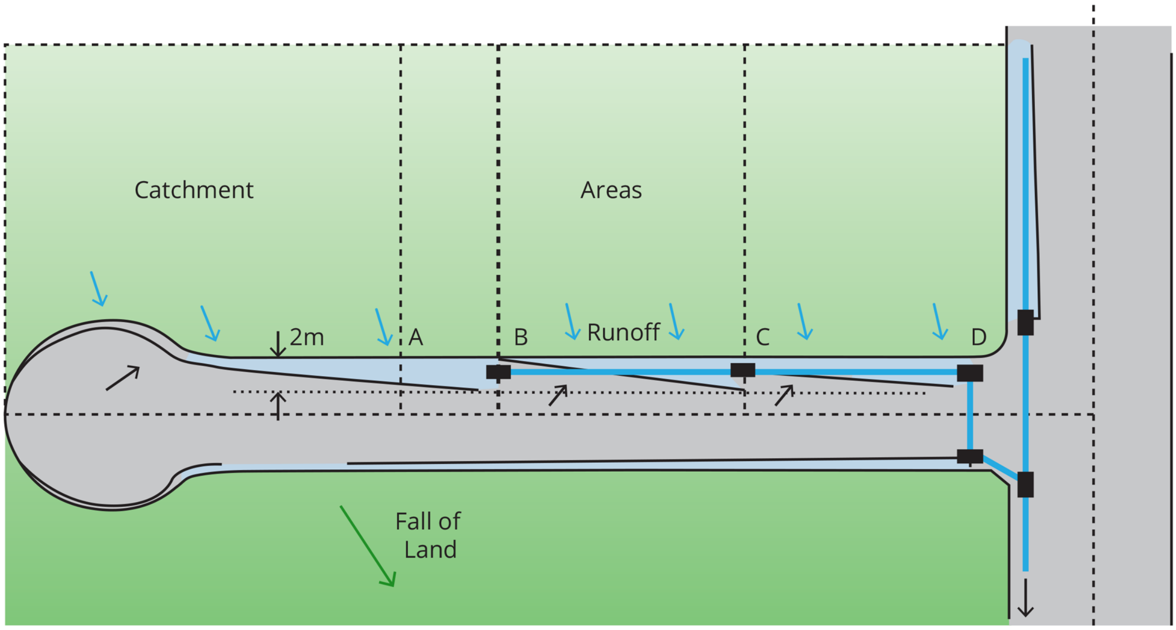


Figure 9.5.6. A Simple Example of a Stormwater Conveyance Network

Figure 9.5.6 indicates the location of inlet pits at the top of a conveyance network. The street gutters are part of the conveyance network and are utilised to transport stormwater towards inlet pits that are situated at intervals to ensure that acceptable flows are carried in road gutters. The width and depth of flows in gutters are limited to allow unimpeded access for pedestrians and vehicles. A maximum width of gutter flow of 2 m to 2.5 m with a maximum flow depth of 150 mm is generally acceptable. Local authorities typically provide guidance on these values. Locations of inlet pits to ensure adequate conveyance of stormwater may also be determined from percentages of stormwater runoff captured by each pit, and the depth of flow and the velocity-depth product of flows in the road gutter.

A designer typically prefers collecting all stormwater runoff from the upper side of the street as shown in Figure 9.5.6 with an inlet pit at Point D which avoids the need for a conveyance branch in the street. This possibility is evaluated by establishing a trial location (A) where stormwater runoff from the corresponding catchment is calculated and the corresponding width of gutter flow is estimated. The width of flows will increase along the gutter length as the areas of contributing catchments increase. A pit must be located whenever any of the criterion limits (such as flow width, depth, or velocity-depth product) are reached. Note that the design process is about limiting surface flows.

Capture of all stormwater runoff at inlet pit B reduces surface flow to zero just downstream of the pit, with surface flows increasing again along the gutter due to lateral inflows from the catchment. However, it is unlikely that on-grade pits will capture all stormwater runoff from catchment areas during minor storm events that create bypass flows downstream of the pit. This is shown at inlet pit C where the flow width increases and reduces due to the pit with a bypass, and some width of flow just downstream of the pit. The flow widths along the gutter will typically follow a saw-tooth pattern.

Figure 9.5.6 also highlights that an inlet pit must be located upstream of a tangent point at an intersection to prevent excessive surface flows at the kerb return. Bypass flows from this inlet pit are collected at inlet pit D. The other pits at the intersection are located along the path of surface overflows to collect both minor and major overflows. This configuration of inlet structures (pits) allows pedestrians to cross at street corners without being exposed to large widths of flows.

The location of inlet pits in the conveyance network may also be driven by a need to provide an inlet at a significant location, such as near a school with street crossings or at a change in road alignment. Aspects of good design practice include location of inlet pits upstream of driveways and avoidance of clashes with other services. A conveyance network also includes additional pipe connections from private property that should be incorporated in the design or analysis of the conveyance network for the street. This may include directly connected pipes from sources such as inter-allotment drainage, on-site volume management systems (such as onsite detention and rain water tanks), or major commercial developments. The first inlet pit in the conveyance network for the street may be receiving considerable pipe flow from upstream private property.

The designer needs to decide on the density of inlet pits in the conveyance network. This decision will typically be guided by the local authority. For example, the two arrangements of inlet pits at an intersection may be acceptable in two different scenarios as shown in Figure 9.5.7.

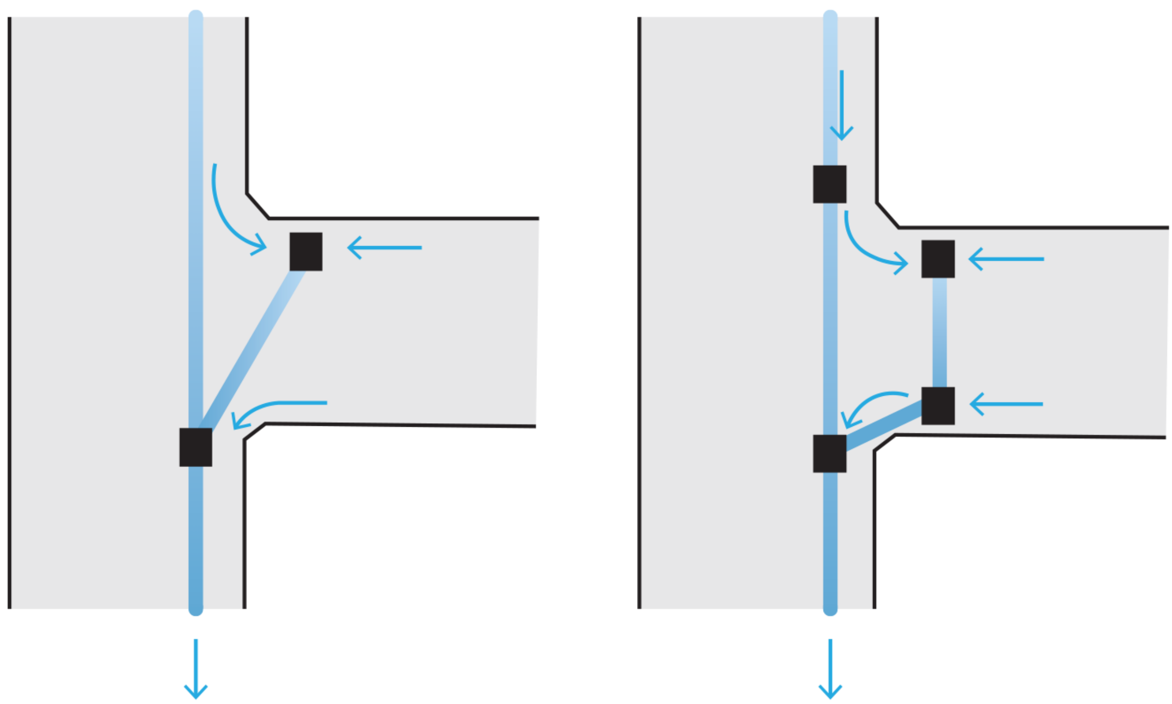


Figure 9.5.7. Example of Alternative Configurations of Inlet Pits at a Road Intersection

Figure 9.5.7 demonstrates different arrangements that may be required at an intersection that could use two or four inlet pits. The decision about the configuration of inlet pits is dependent on the magnitudes and consequences of the flows that may bypass the inlet pits. In a densely-developed area, where overflows or bypass flows may cause nuisance and damage, a greater number of inlet pits will be preferred. Fewer inlet pits can be used in a lower density development where surface flows are more easily managed and the consequences of overflows or bypass flows are small.

5.4. Design of Conveyance Networks with Computer Models

Design of urban stormwater conveyance networks has a long history in Australia. Hand calculations using the urban Rational Method was discussed in the 1958, 1977 and 1987 editions of ARR as the most utilised method for estimation of stormwater inflows to conveyance or drainage networks. There is significant ongoing concern about the reliable characterisation of the parameters (such as runoff co-efficient and time of concentration) underpinning the Rational Method due to insufficient rainfall runoff observations in urban areas (Coombes et al., 2015).

A transition into the computer age heralded the design and analysis of urban conveyance networks using computers to operate drainage software or spreadsheet manipulation that often implemented the Rational Method. The sizing of conveyance infrastructure was based on estimates of peak flows. Increases in computing power allowed greater access to software that integrated hydrology and hydraulics to more accurately analyse or design conveyance networks using hydrograph methods. Additional details about urban modelling approaches are provided in Book 9, Chapter 6.

The characteristics of contemporary urban stormwater management have evolved to be different to the objectives and design solutions for urban stormwater drainage or conveyance networks as envisaged in 1987 (Coombes, 2015). Since the 19th century, the Rational Method and hand calculations has evolved into modern rainfall runoff models (refer to Book 9, Chapter 3). The catchment area has been subdivided into sub-catchments. Average rainfall intensity derived from storm bursts has been modernised to include temporal patterns, spatial variation, relationships between different burst rainfall depths and durations, and the capture of partial areas effects. The runoff coefficient for estimation of stormwater runoff has been replaced with processes that account for the degree of urbanisation and spatial distribution of different land uses, addition of loss models to determine rainfall excess, accounting for pervious and directly or indirectly connected impervious surfaces, and inclusion of depression storages.

Rainfall runoff models have also incorporated connective components including:

- the shape of drainage networks;
- addition of drainage network conveyance, travel times and system storages;
- a separation of minor and major systems;
- response times of different components (such as roads and gutters); and
- bypasses of drainage pits and storages in sag pits.

These evolving models account for modern urban features including distributed storages such as rain water tanks, bio-retention and on-site detention; detention basins and the spatial distribution of urban features (refer to Book 9, Chapter 4). Modern design criteria include analysis of the volume, timing and frequency of stormwater runoff to determine peak flow rates, water quality and requirements. This is done to mimic natural regimes of volumetric flows to protect waterway health (Walsh, 2004; Walsh et al., 2016). Management of the volume of stormwater runoff and the frequency of runoff events from urban catchments is now seen as a key design objective to mitigate downstream flooding and protect the health of urban waterways.

Predictions of peak stormwater flows using the Rational Method may not adequately represent the fundamental processes occurring within contemporary urban catchments (Coombes et al., 2015). This concern is particularly relevant to modern stormwater management methods, such as Water Sensitive Urban Design (WSUD), that include cascading integrated solutions involving retention, slow drainage via vegetation, harvesting and reuse of stormwater and the disconnection of impervious surfaces. These distributed solutions within catchments alter runoff volume and timing in a variable manner throughout a catchment (refer to Book 9, Chapter 3). These dynamics are more likely to be revealed by advanced analysis methods. Importantly, provision of optimum designs for urban stormwater management is dependent on testing solutions across the full range of urban dynamics. The limited urban data available for characterising the parameters underpinning the urban Rational Method for average urban conditions remains a challenge.

The design procedure using computer models is typically implemented more easily and accurately than the simpler design methods. A main advantage is the ability of a computer model to rapidly perform design procedures once a system is set up and the necessary data is entered. In addition, use of computer software allows simultaneous analysis of both minor and major storm events to adequately size inlet structures and conveyance infrastructure, ensuring safe overland flow outcomes.

The procedures for design of conveyance networks have evolved from simplifying assumptions required for hand calculations, such as assuming that pipes are flowing full but not under pressure. Modern methods include more calculations and checks, and can apply unsteady flow hydraulic simulations throughout conveyance networks. These complex calculations are implemented using computers. The amount of calculations is now so large that simple numerical checks using hand calculations are not possible. However, ‘sanity checks’ can (and should) be made to compare results from models using simplified procedures such as estimating flowrates per unit area. These simple checks will provide estimates that are different to the results produced by computer models, however, this process should assist in avoiding gross errors.

Peak flowrates and hydrographs calculated by rainfall-runoff models are inputs to hydraulic models that determine the characteristics (elevations, depths, widths and velocities) of stormwater flows throughout catchments. Hydraulic modelling is based on physics and requires that the geometry of components of a conveyance network should be carefully defined. Key hydraulic concepts such as Continuity, Conservation of Mass, Energy, and the Bernoulli's Equation, are covered in Book 6, Chapter 2. The Friction Equations including Darcy-Weisbach, the Manning and the Colebrook-White Equations (Book 6, Chapter 2) are all important considerations for the hydraulic design of conveyance networks.

A range of hydraulic models can be used to design conveyance networks. The performance of the models can be illustrated by Hydraulic Grade Lines (HGL) and Eenergy Grade Lines (EGL, also called Total Energy Line, TEL). These grade lines are described in books on fluid mechanics and hydraulics and are useful for understanding flow phenomena.

The design of conveyance infrastructure is highly dependent on the capacity of inlet structures. Inlet structures (pits) are chosen, and then surcharges from each structure are calculated, which determines the possible cumulative surface flows throughout sub-catchments. A check is made to see whether the hydraulic characteristics of surcharges exceed performance objectives for the network (such as safety and access criteria). If the performance objectives are exceeded, the size of inlet structures needs to be increased using the next largest inlet structures and calculations proceed. During the design of inflows and associated dimensions of inlet structures, conveyance infrastructure (pipes) are also sized to ensure that the performance objectives are met. This process continues until a satisfactory level of surface flow is reached.

5.5. Inlet Structures

The performance of urban conveyance networks is dependent on the effectiveness of the inlet structures (pits) that capture stormwater runoff at regular intervals throughout the network. Relationships for the capacity of inlet structures determine the magnitudes of bypass flows and are an essential part of the design of conveyance networks. Designers should be concerned that flow widths and depths are within appropriate limits, both upstream and downstream of an inlet structure.

Designers need to consider that the effectiveness of inflow structures is impacted by the inflow of stormwater through grates or kerb inlets, and by the energy losses or pressure changes that are created by inlet structures (refer to Figure 9.5.8). Historically, pit losses were simplified as a single simple coefficient that approximates the reality of entry losses to the pit, losses within the pit and exit losses from the pit. A simple single coefficient is generally used for many different types of inlet structures.

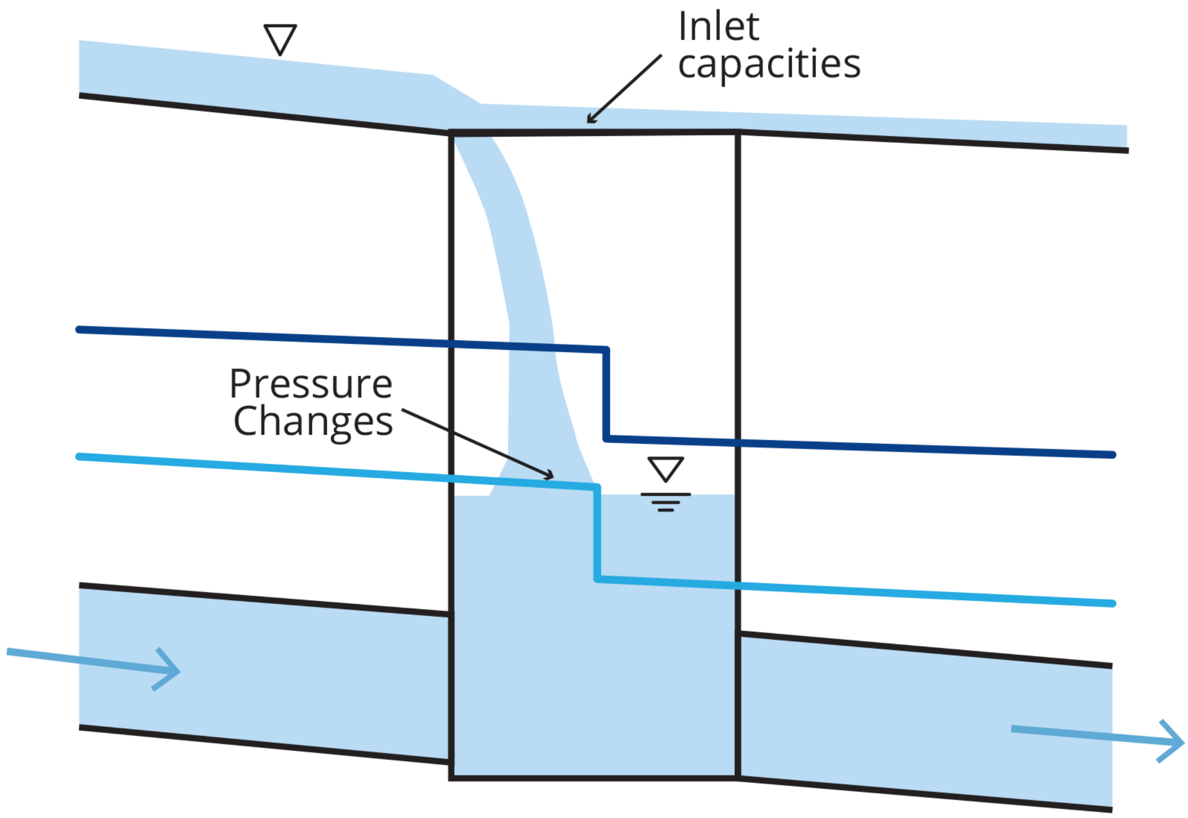


Figure 9.5.8. Idealised Hydraulic Issues Impacting on Inlet Structures

5.5.1. Types of Inlet Structures

A majority of urban stormwater runoff enters conveyance networks via inlet structures located in gutters and medians of roads. These inlet structures or drainage pits are classified by shape or configuration, and are also defined by location on a slope (on-grade pits) or in a depression (sag pits), as shown in Figure 9.5.9. On-grade and sag inlet structures are subject to different hydraulic processes. The behaviour of on-grade pits links inlet capacities to approaching flowrates and resulting bypass flows. Performance of sag pits is dependent on stormwater inflows, pipe outflows and depths of ponded water over pits which cannot escape without passing over footpaths or crowns of roads.

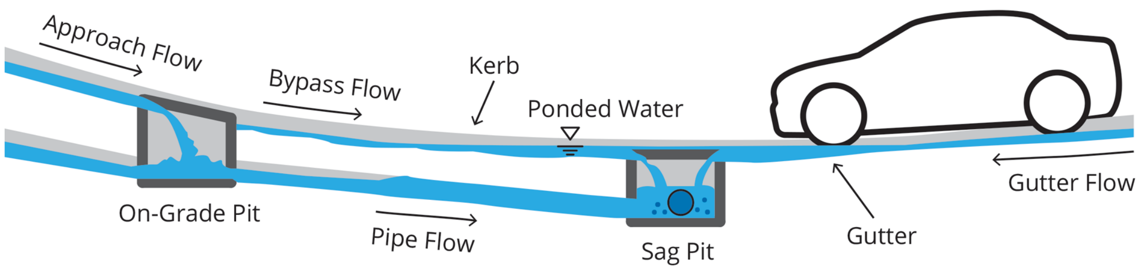


Figure 9.5.9. Basic Types of Inlet Structures

It is desirable for an inlet structure to maximise collection of stormwater runoff. However this objective must also include the safety and convenience of pedestrians, cyclists and motorists, and costs of infrastructure. Open pit structures that may provide the greatest inlet capacity are unacceptable in most environments. The design of inlet structures must not permit children to enter the pit or the conveyance network.

Grates and kerb inlet pits (also referred to as side entries or lintels) are typical inlet structures that are deployed either separately or in combination. Capacities of inlet structures can be improved by providing extensions to kerb inlets, deflectors (ribs or grooves that direct water into an inlet), depressed grates and gutters, or clusters of inlet structures that include adjacent installation of two or three standard pits. Grates and depressions of inlet structures should not be hazardous to road users, including cyclists, and their use should be avoided on busy narrow roads. Aspects of inlet structures for bicycle safety are discussed by the U.S. Federal Highway Administration (Burgi and Gober, 1977).

There is limited information on simple relationships available for the capacity of many types of inlet structures. Many investigations of pit entry capacities have utilised hydraulic models. A range of significant historical studies were published by Burgi and Gober (1977), the Australian Road Research Board (1979), NSW Department of Main Roads (1979) and Marsalek (1982). More general information about capacities of inlet structure are provided by Searcy (1969), Jens (1979), Marsalek (1982), Mills and O'Loughlin (1986), and Argue (1986).

More recent laboratory experiments have examined capacities of different inlet structures at the Manly Hydraulics Laboratory in NSW and at the University of South Australia. The relationships obtained from laboratory tests do not extend to flow rates that may occur in extreme flood events such as 1% AEP or probable maximum floods. However, these relationships are still useful for most design problems as inlet structures in urban areas are predominantly used to admit inflows from minor or more frequent events into conveyance networks.

The US Federal Highway Administration (NHI, 2013) has published the general procedure for determining inflow capacities of on-grade pits in their Hydraulic Engineering Circular No. 22 (HEC-22). The efficiency of various grate types and impacts on inlet capacities for a range of approach grades and velocities are important considerations for urban conveyance networks. In addition to grate and kerb inlets, the capacities of slotted drain inlet structures are also relevant for locations where interception of wide sheet flow is desirable and low sediment and debris is expected. The HEC-22 pit inlet procedures are a useful source of information to aid design of inlet structures.

5.5.2. Inlet Capacities

The hydraulic behaviour of on-grade and sag pits is quite different. These differences are discussed below.

Sag Inlet Pits

The capacities of sag pits are generally independent of upstream gutter slopes and are governed by weir and orifice equations which are dependent on the depth of ponding. The weir equations apply to flows that enter the pit at its edges or at the edges of bars in a grate. Alternatively orifice equations are applied when water ponds above the inlet structure at depths typically exceeding about 0.2 m. The depth of ponding increases to a threshold level and stormwater will overflow as bypass flow by passing over a ‘weir‘ such as a road crown or driveway hump or wall.

The approach and cross-fall grades of roads can affect the availability of storage volumes surrounding sag pits which can indirectly affect the overall behaviour of sag pits. These issues can be considered using hydrodynamic analysis of sag pits as small detention structures. Sag pits must have sufficient inflow capacity to accept the total inflows of stormwater runoff to avoid undesirable ponding of stormwater in intersections to limit obstruction to turning traffic, onto footpaths, into adjacent private properties or basement car parks, or over the crown of a road during a minor storm.

Basic calculations for determining approximate inlet relationships for grated sag pits were derived by Searcy (1969). However, it is preferable to utilise the HEC 22 procedures rather than the sag pit Equation (9.5.1) and Equation (9.5.2) when side entry inlet relationships are required.

For a grate,

$$Q_i = BF \times 1.66\ Pd^{1.5} \qquad \text{up to about 0.12 m of ponding } (d < 0.12) \tag{9.5.1}$$

or

$$Q_i = BF \times 0.67A(2gd)^{0.5} \qquad \text{over 0.43 m of ponding } (d > 0.43) \tag{9.5.2}$$

where

Q_i is the inlet flow rate (m^3/s),

BF is the Blockage Factor

d is the average depth of ponding (m),

P is the perimeter length of the pit excluding the section against the kerb (m) (bars can be disregarded),

A is the clear opening of the grate (m^2), i.e. total area minus area of bars, and

g is acceleration due to gravity (approximately 9.81 m/s^2).

The relationship for inlet capacity between depths of 0.12 and 0.43 is described by ARR 1987 as indefinite and Equation (9.5.1) was recommended in that situation. For an inlet structure that is not located in a depression, the following relationships are recommended:

For ponding up to 1.4 times the height of the inlet:

$$Q_i = BF \text{ x } 1.66Pd^{1.5}h\ (d\ \leq 1.4h) \tag{9.5.3}$$

or

For ponding greater than 1.4h (d > 1.4h):

$$Q_i = BF \text{ x } 0.67A\left(2g\left(d - \frac{h}{2}\right)\right)^{0.5}(d > 1.4h) \tag{9.5.4}$$

where Q_i is the inlet flow rate (m^3/s),

h is the height of the inlet

BF is the Blockage Factor

d is the average depth of ponding (m)

L is the inlet width (m),

A is the clear opening of the grate (m^2), i.e. total area minus area of bars, and

g is acceleration due to gravity (approximately 9.81 m/s^2).

Charts of the inlet capacity of depressed kerb inlets at sag points are provided by Searcy (1969) and in (NHI, 2013).

On-Grade Inlet Pits

Calculation of relationships for inlet capacity of on-grade pits is more complex than for sag pits as several factors can change the capacity of inlets. These factors include:

- grade of the approach gutter (or channel) which will vary flow velocity;
- road cross-fall which impacts the flow width and consequently the maximum allowable flow depth at the inlet;
- roughness of the gutter and road pavement (or channel);
- efficiency of the grate; and
- entry conditions leading into the pit chamber such as gutter depressions (Figure 9.5.10) and the angle of the throat (inlet to the pit) (Figure 9.5.11).

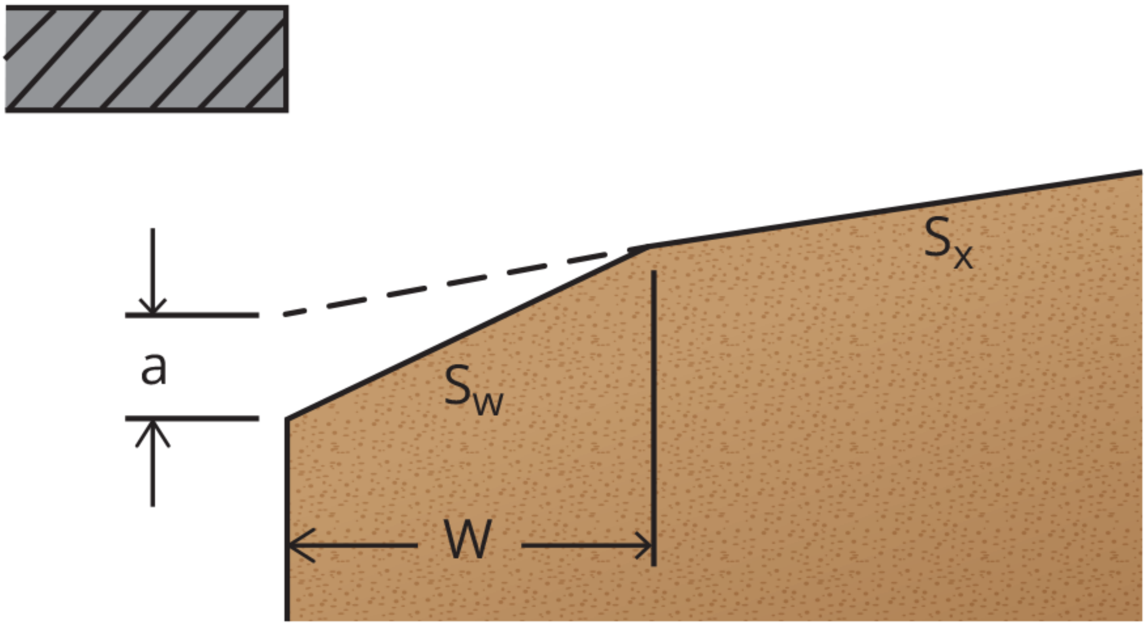


Figure 9.5.10. Kerb Inlet Gutter Depressions from HEC-22 (NHI, 2013)

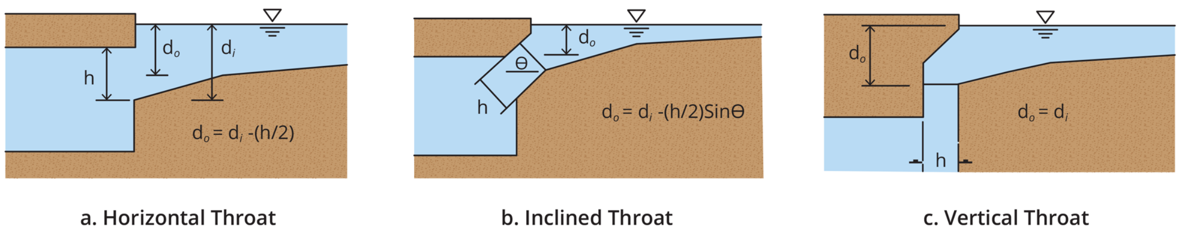


Figure 9.5.11. Kerb Inlet Throat Angles from HEC-22 (NHI, 2013)

The basic calculations to determine approximate relationships for inlet capacities of grate, side entry and combination inlets are provided by Searcy (1969). However, Equation (9.5.3) and Equation (9.5.4) should not be used in preference to HEC 22 procedures which have been hydraulically tested, and where the efficiency of various grate types is provided along with calculations for throat entry conditions. As an illustration, typical relationships for 1 m and 2 m on-grade kerb inlets are shown in Figure 9.5.12 that were derived using the HEC 22 procedures.

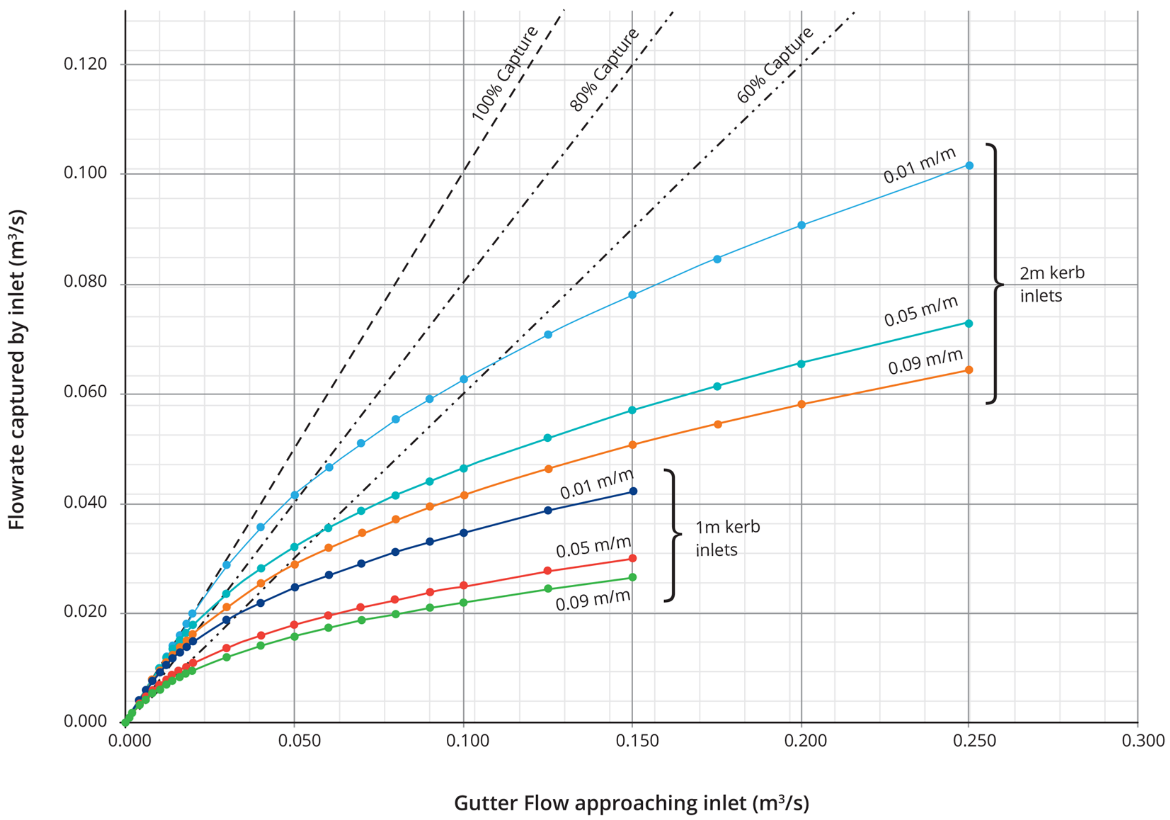


Figure 9.5.12. Inlet Capacities for On-grade Pits

Additional Information

Many different types of inlet structures are used across Australia and this chapter has only discussed some of the configurations. It is recommended that local capacity relationships, knowledge and experience, and types of inlet structures should be employed in designs for urban stormwater conveyance networks. In the absence of mandated design procedures that may be provided by a local authority, preferences should be given to local knowledge and experience, and to laboratory based methods. The designer and local authority should also accept first principles hydraulic analysis and evolving science in the selection of inlet capacities.

Additional resources available to the designer include those provided local and state authorities such as Vic Roads, the (QUDM, 2013) and from older resources including the National Capital Development Commission (1981), the Victoria Country Roads Board (1982) and the New South Wales, Department of Housing (1987).

The usual pit entry capacity relationships may not be adequate for analysis of conveyance networks subject to major rainfall events. In these situations, larger depths of surface flows, velocities and loads of debris may occur, and the inlet capacities of pits will be make for additional discussion). A blockage factor of 50% is generally applied for sag pits for minor and major systems in situations when experimental results or observations are not available. The blockage factor for on-grade pits can vary from 0% and 20% in response to local conditions. Additional advice on blockage factors is provided by Weeks et al. (2013) as shown in Table 9.5.1. Higher blockage factors are often applied for events rarer than the 1% AEP.

Table 9.5.1. Suggested Design and Severe Blockage Conditions for Inlet Pits Book 6, Chapter 6

Type of structure		Blockage conditions	
		Design blockage	Severe blockage
Sag kerb inlets	Kerb inlet only	0-20%	100% (all cases)
	Grated inlet only	0-50%	
	Combined inlets	Capacity of kerb opening with 100% blockage of grate	
On grade kerb inlets	Kerb inlet only	0-20%	100% (all cases)
	Grated inlet only (longitudinal bars)	0-40%	
	Grated inlet only (transverse bars)	0-50%	
	Combined inlets	10% blockage of combined inlet capacity on continuous grade	

Ultimately relationships for the capacity of inlet structures determine the magnitudes of bypass flows and are an essential consideration in the design of conveyance networks. Designers must be concerned that flow widths, depths and product of depths and velocities are within appropriate limits at locations upstream and downstream of an inlet structure. These factors can be controlled by the careful location of inlet structures and by limiting bypass flows using infrastructure with sufficient inlet capacities.

5.5.3. Energy Losses

Significant pressure losses may be created by inlet structures and junctions in conveyance networks. Hydraulic losses are generally reduced when open channel flows occur in conveyance infrastructure (pipes) and benching or smooth transitioning is provided within inlet structures. Higher losses occur at inlet structures when conveyance infrastructures (pipes) are full and surcharging in response to pressure flows. These losses at pits are offset by the increased capacity of pressurised pipes and the entire pressurised conveyance network may cope with greater flow rates. Energy losses at inlet structures are expressed as a function of the velocity V_0 in the outlet or downstream pipe:

$$h_L = k\,.V_0^{\,2}/2g \tag{9.5.5}$$

Where:

h_L is the loss in metres,

k is a dimensionless energy loss coefficient, and

g is acceleration due to gravity (m/s^2).

This energy loss at the inlet structure creates a change in the total energy line (TEL) as shown in Figure 9.5.13. The associated change in the hydraulic grade line (HGL) is likely to be different in response to different pipe diameters and flow rates upstream and downstream of the structure. The position of the HGL is important to designers as it determines the location of the water surface and the degree of surcharge or overflow which may occur at that location in the conveyance network.

The change in pressure head is estimated as:

$$\Delta P/\gamma = k_u.V_0^{\,2}/2g \tag{9.5.6}$$

Where:

$\Delta P/\gamma$ is the pressure head change (m) relating to a change of pressure of ΔP kN / m^2 and the specific weight of water kN/m^3, and

k_u is a dimensionless coefficient of change in pressure.

A similar relationship can be applied to water levels within inlet structures which may be slightly higher than the HGL level due to the conversion of some kinetic energy to pressure energy when stormwater flows through a pit:

$$WSE = k_w.V_0^{\,2}/2g \tag{9.5.7}$$

Where:

WSE is the elevation of the pit water surface (m) relative to the downstream HGL elevation, and k_w is a dimensionless coefficient.

These effects are illustrated in Figure 9.5.13.

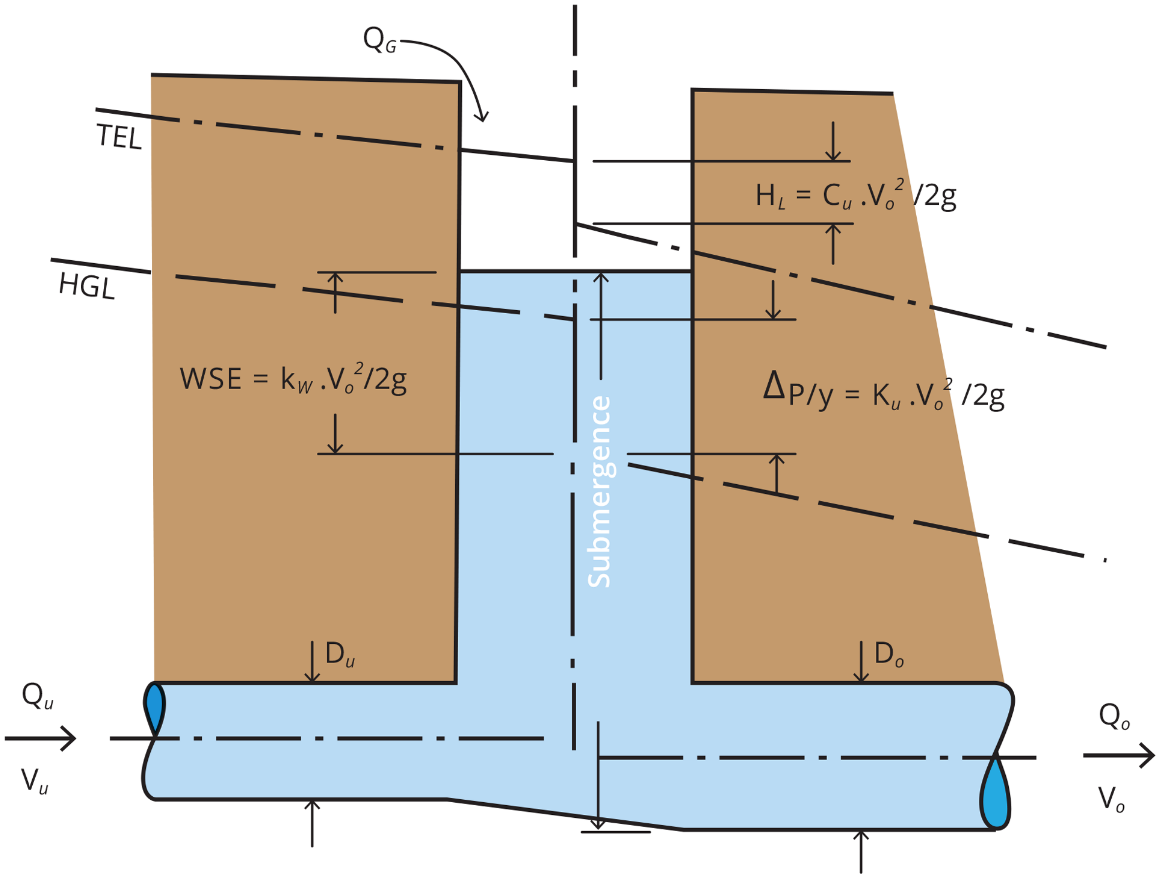


Figure 9.5.13. Idealised Grade Lines at an Inlet Structure

Where Q_o is the downstream discharge, V_o is the downstream flow velocity, D_o is the downstream pipe diameter, Q_g is the surface inflow to the pit, D_u is the upstream pipe diameter, Q_u is the upstream flow rate and Vu is the upstream flow velocity. The parameters k_u and k_W are similar for most configurations of inlet structures and the water level in a pit can be assumed to coincide with the HGL level. The arrangement of grade lines in Figure 9.5.13 is an idealised situation that assumes all changes occur at the centreline of the inlet structure. Losses actually occur across the structure and immediately downstream in the conveyance infrastructure. The convention in measurement of the performance of inlet structures is to project grade lines (measured by manometers in the upstream and downstream pipes) forwards or backwards to the pit centreline and to accept the difference as the overall loss or pressure change.

Available Methods of Determining Pressure Loss Coefficient k_u

Studies using hydraulic models can be used to derive reliable values of energy losses and pressure changes for different types of pits and junctions. A significant study by Sangster et al. (1958) dealt with pipes flowing full and produced a set of design aids for a selected configuration of inlet structures which are now called “Missouri Charts”. Hare (1980); Hare (1983) produced information on other configurations. The charts are complex and provide many possible geometric configurations of inlet structures. Careful judgement is required to select the appropriate chart for a particular configuration of a structure, and in practice, iterative calculations are required to converge to a suitable value of the pressure loss coefficient.

This iterative process can be quite time consuming for large conveyance networks. Attempts have been made to replace dependence on charts with semi-analytical methods. These range from relatively simple methods suggested by Argue (1986), Hare et al (1990) and Mills and O'Loughlin (1998) to more in-depth methods suggested by Parsell (1992) and the US FHWA HEC-22 procedure from which the algorithm described by GKY and Associates Inc (1999) and Stein et al. (1999) has been developed. The FHWA HEC-22 procedure was developed using research and laboratory efforts improving the methodologies of the 'Corrective Coefficient Energy-Loss Method' (Chang and Kilgore, 1989) and the 'Composite Energy Loss Method" Chang et al. (1994). It is also the only method which considers part-full and full pipe flow, drops in pits and other situations.

A summary paper by O'Loughlin and Stack (2002) compared the different algorithms and could not find significant differences which suggested that no single method was superior. However, the information indicated that a viable algorithm can be developed, and that further testing and development is required for the methods to acceptably match the full range of configurations of inlet structures provided by the Hare (1983). The FHWA algorithm appears to provide a significant advance in the determination of head losses and pit pressure changes in stormwater conveyance networks. Comparisons with alternative algorithms and experimental data indicated that simpler methods may provide equivalent results for losses.

Determining Pit Pressure Losses in Practice

Determining pressure changes in practice is complex due to the many possible geometric configurations of inlet structures. Geometric configurations of pits can vary according to:

- number of pipes entering pits (0, 1, 2, 3 or more);
- horizontal change of direction at the pit;
- vertical drop in the pit between inlet and outlet pipes;
- ratios of incoming and outgoing pipe diameters;
- a number of secondary factors, including slopes of pipes, shape and size of inlet structure, depths of sumps in the structure below the invert of the outgoing pipe, streamlining (or benching) of the pit and the entrance to the outlet pipe, and location of the confluence of the incoming pipes.

Variances in flows are impacted by:

- magnitudes of flow and velocity;
- ratios of grate flow entering the top of structures compared to the outflow; and
- tailwater levels.

The design calculations typically need to be repeated to achieve converging values. When designing to satisfy a freeboard requirement, revised coefficients may lead to circular alteration of pit and pipe inlet capacities which requires the designer to intervene.

Initial Estimates of k_u Before Commencing Iterative Calculations

An analysis of the hydraulic grade line of a pipe requires an estimated value of k_u at each inlet structure. Some government authorities may provide suggested values and experienced designers are likely to have developed 'rule-of-thumb' methods for determining these initial estimates of k_u. Engineers are encouraged to use these methods in hydraulic design wherever the methods have proven to be effective.

Guidance for initial estimates of k_u is provided in Figure 9.5.14 for a range of common pit configurations. These are not absolute or recommended values for final analysis of a network and are only indicative starting points of iterations required to converge to a final value. These estimations assume shallow pipes with typical minimum covers and no increases in outlet pipe diameters. Deeper inlet structures may increase values of k_u and increases in outlet pipe diameters may reduce values of k_u

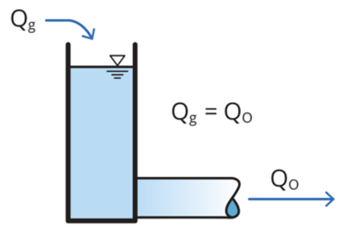
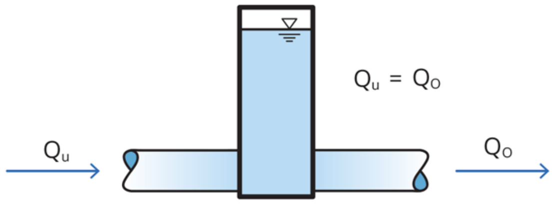
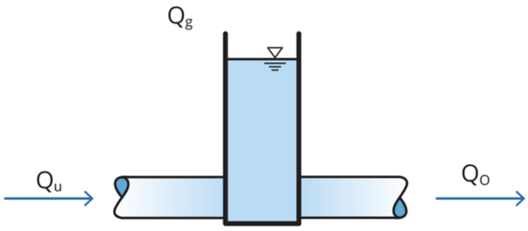
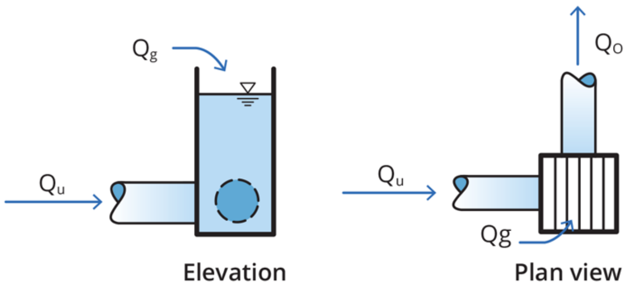
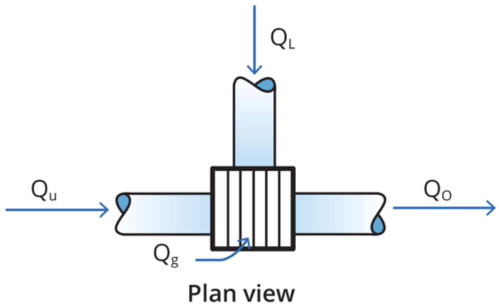
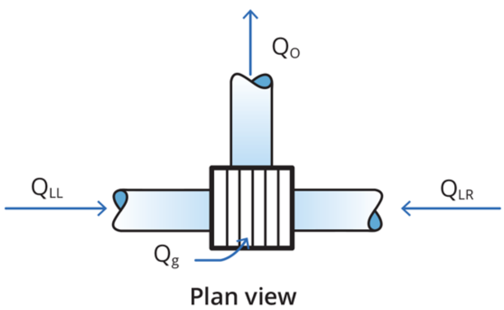
Pit Configuration	Initial k_u	Pit Sketches
First pit at the top of a line	4.0	
Well-aligned junction pit with straight through flow, no sidelines, no grate inflow	0.2	
Well-aligned pit With straight through flow, no sidelines, 50% grate inflow	1.4	
Pit with a 90° right angle direction change, no sidelines, 50% grate inflow	1.7	
Pit with a straight through flow, one or more sidelines	2.2	
Pit with a right angle direction change from two opposed inflow pipes	2.0	

Figure 9.5.14. Approximate Pressure Change Coefficients, k_u, for Inlet Structures

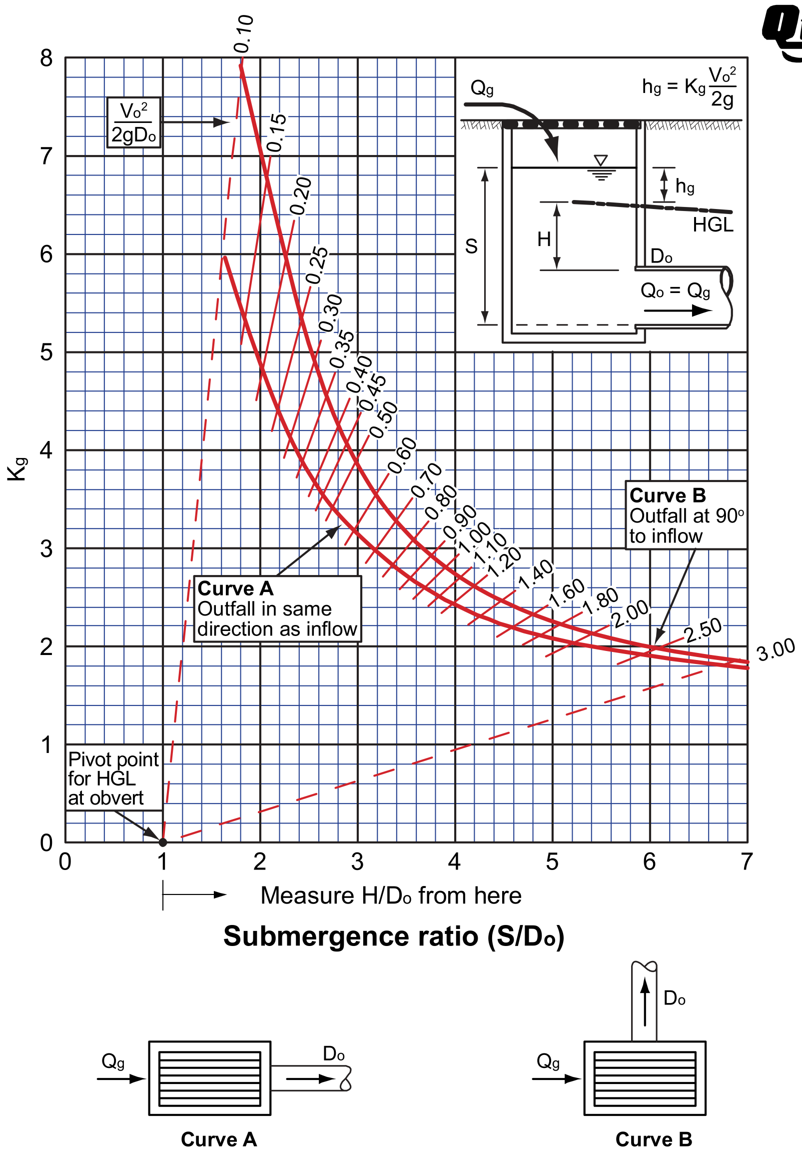
Simplified Approach

As discussed earlier, simplified design methods are available such as those presented by Mills and O'Loughlin (1998), Hare et al. (1990), and Argue (1986). Although these simpler methods may provide similar results to more complex semi-analytical methods, further laboratory research and development was recommended to account for the full range of pit configurations considered by the original Missouri Charts (Sangster et al., 1958) and by Hare (1980). Whilst simplified design methods may be considered for use during simple, non-critical pit and pipe network designs, use of Missouri Charts and Hare's results is preferred.

Recommended Approach

The Missouri Charts (Sangster et al., 1958) and the results from Hare (1980) remain widely accepted and are relevant to an estimated 85% of the possible configurations of inlet structures. The example charts presented in Figure 9.5.15 and Figure 9.5.16 are based on this information (QUDM, 2013). The first chart (Figure 9.5.15) was derived from the original Missouri Chart 2 with modification from the Department of Transport (1992) for an inlet structure with grate flow only. The pressure change coefficient k_u depends on the submergence ratio S/D_0 and iterative calculations are required.

The second example chart (Figure 9.5.16) was modified from the Missouri Chart 4 to include the results from(Hare, 1980). The inlet structure accommodates flows straight through the pit for a submergence ratio S/D_o of 2.5 and also considers inflows through grates. Here k_u depends on the ratio D_u/D_o and provides flow ratios Q_g/Q_o ranging from 0 to 0.5. A correction factor needs to be added from Table 9.5.2 when the submergence ratio S/D_o does not equal 2.5.



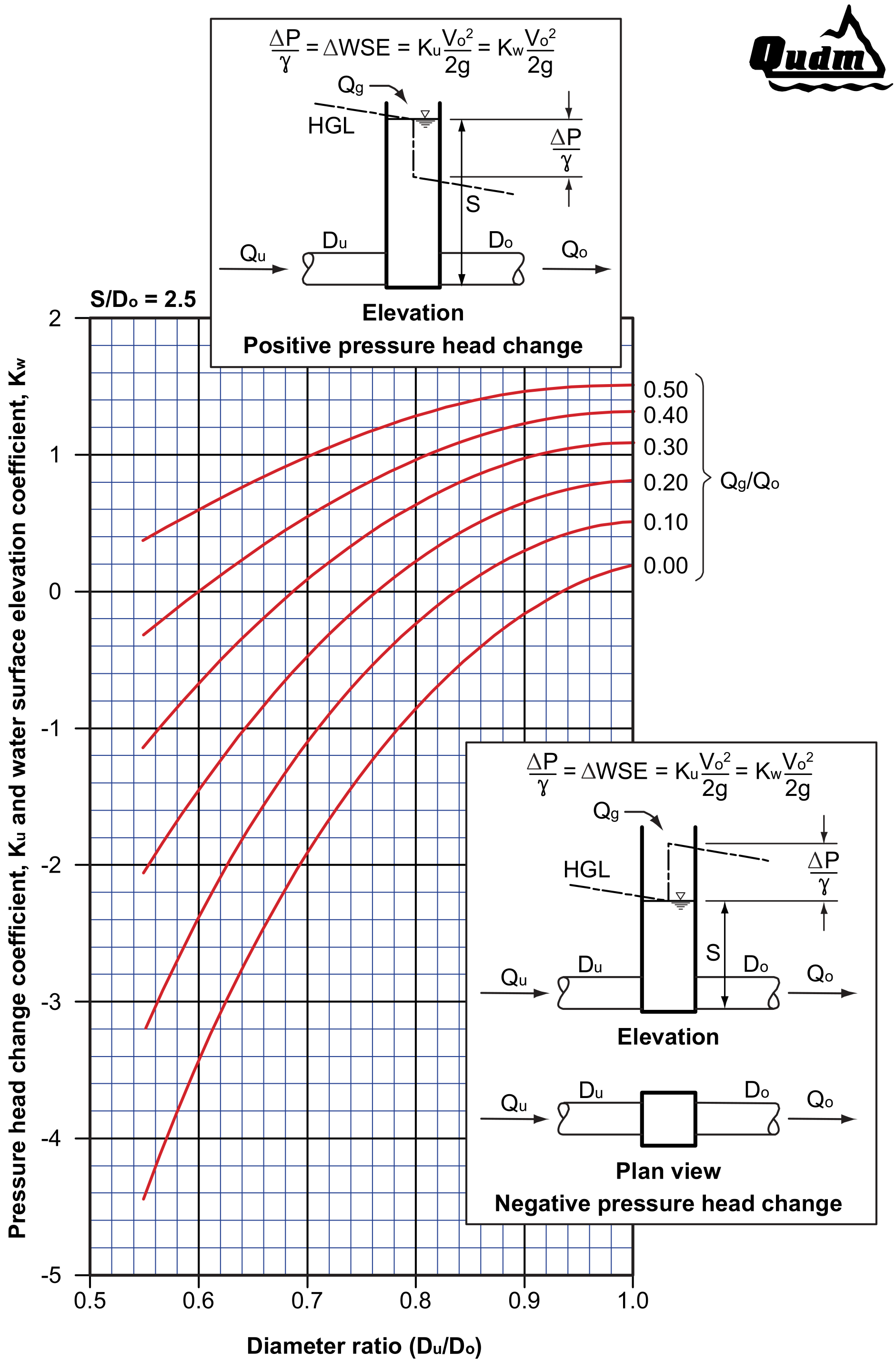
Pressure head change coefficients for rectangular inlet with grate flow only modified from DOT (1992)

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

Figure 9.5.15. Pressure Change Coefficient Chart (QUDM, 2013)



Pressure head change and water surface elevation coefficients for straight through flow for submergence ratio, $S/D_o = 2.5$ (Source: Hare, 1980)

Chart No. A2-4

Figure 9.5.16. Pressure Change Coefficient Chart (QUDM, 2013)

Table 9.5.2. Correction Factors for k_u and k_w for Submergence Ratios (S/D_o) not Equal to 2.5 (QUDM, 2013)

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	-0.06	-0.09	-0.12	-0.15
3.5	0.00	-0.04	-0.08	-0.12	-0.16	-0.20
4.0	0.00	-0.05	-0.10	-0.15	-0.20	-0.25

Additional influencing factors become apparent as configurations of inlet structure become more complex; such as interpolation coefficients for intermediate grate flow ratios, presence of deflectors and additional lateral or sideline pipes. The second chart (Figure 9.5.16) shows that k_u can be negative in situations where the outlet pipe is larger than the inlet pipe and "pressure recovery" occurs due to the lower downstream flow velocities than the upstream inflow velocities.

Large energy losses and pressure changes can be avoided by attention to simple rules in detailed design and construction. One principle is to ensure that jets of water emerging from inlet pipes do not impinge directly on pit walls. Wherever possible the stormwater jets from inflow should be directed into outlet pipes. Hare (1983) states that changes of flow direction should generally occur on the downstream face of pits, rather than at the upstream face or centre. Losses may be reduced by use of curved pipelines, precast bends and slope junction fittings at changes of flow direction. Typical loss factors for these fittings are:

- tee – k = 1.15 for energy loss expression $kV^2/2g$
- 90° double mitre bend – k = 0.47
- 60° double mitre bend – k = 0.25
- 45° single mitre bend – k = 0.34
- 22° single mitre bend – k = 0.12

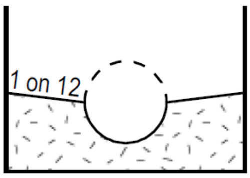
Benching

The recommended Missouri Charts do not include the effect of benching to reduce energy losses. Potential decreases in pressure change coefficients as a result of benching are provided in Figure 9.5.17, (Table 7.16.4 in QUDM (2013)).

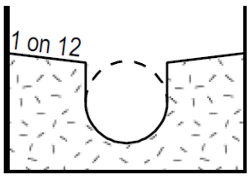
Potential decrease in pressure change coefficient as a result of benching

Access chamber type [3]	Potential decrease in pressure change coefficient (%)	
	Half-height benching [1]	Full-height benching [2]
Straight through	30	40
90° 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:



Note [1]: (a) –
Half-height benching



Note [2]: (b) –
Full-height benching

Note [3]:
Results based upon testing of square pits.

Figure 9.5.17. Decrease in Pressure Change Coefficient as a Result of Benching (QUDM, 2013)

Computer Models

Various procedures have been implemented in computer software. Some unsteady flow computer programs allow for pressure losses in rather simplistic ways, such as increasing pipe friction factors to include estimated pressure losses. Other complex procedures employed by computer software include:

- iterative processes based on Missouri Charts, geometry and hydraulic results;
- semi-analytical algorithm based approaches; and
- numerical methods.

5.6. Conveyance Infrastructure

Urban conveyance networks collect rainfall runoff from urban surfaces (properties and adjacent roads) and utilise gutters, road surfaces, pipes, culverts and channels to convey stormwater to downstream infrastructure or receiving waters. This section discusses the design of conveyance infrastructure.

5.6.1. Hydraulic Models to Define Flow Characteristics

The complexity of conveyance networks requires that simple calculations based on energy gradients are often replaced by more complex procedures. Rainfall runoff is collected at multiple entry points (inlet structures) and accumulates throughout the conveyance network.

The necessary calculations combine these inflows and route them throughout a network by determining the water depths and velocities in the conveyance infrastructure. Simpler methods or models can do this for steady flows with unchanging flow rates whereas more complex models are required for unsteady and time-varying flow ates. Hydraulic grade lines (HGL) and energy grade lines (EGL) can be used to define flow depths, pressures and energies in conveyance networks as shown in Figure 9.5.18. Hydraulic models must allow for overflows when water levels exceed limits or pass over barriers. Additional information about hydraulic models is provided in Book 6 and Book 9, Chapter 6.

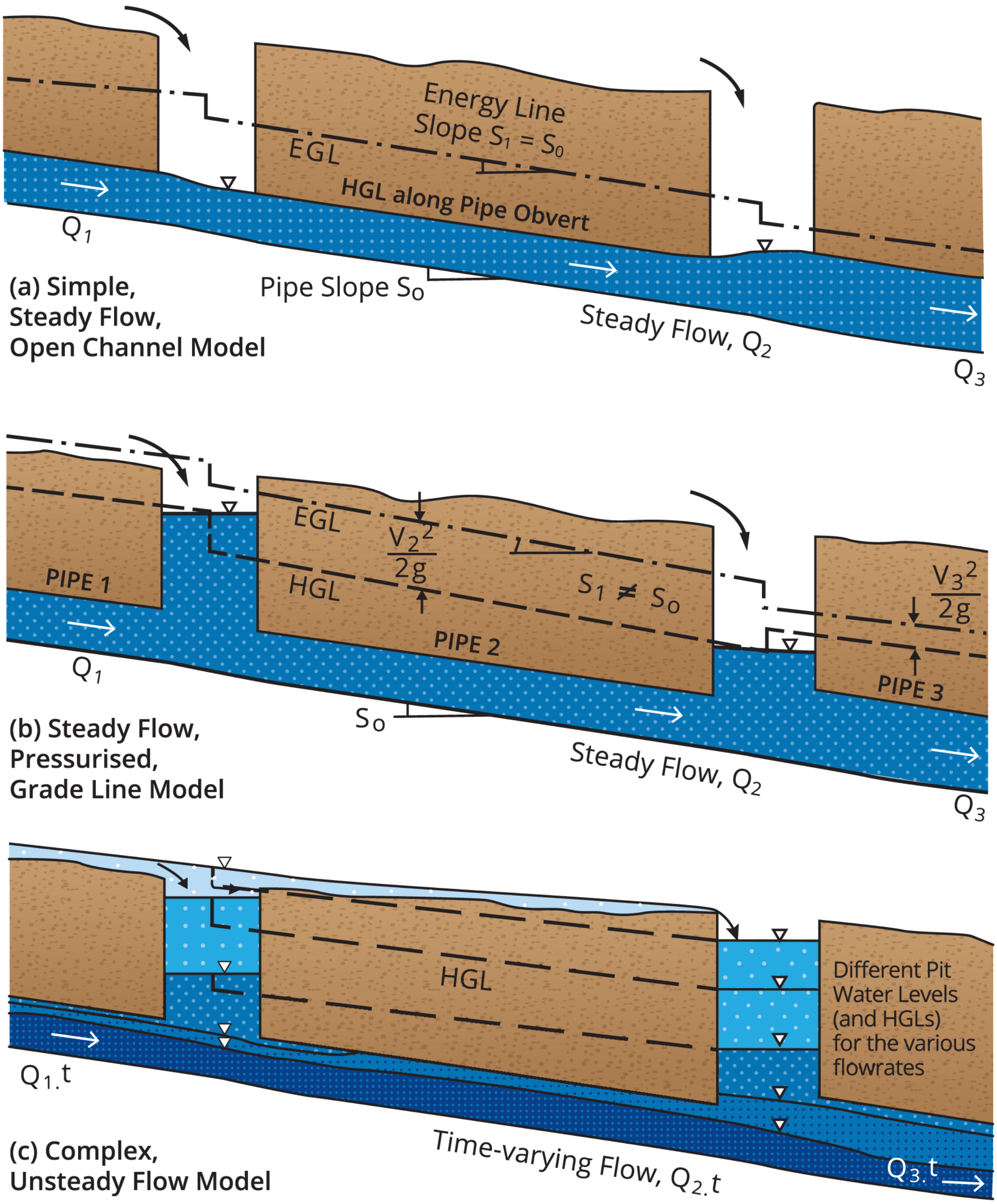


Figure 9.5.18. Schematic of Three Hydraulic Models of Conveyance Networks

Figure 9.5.18(a) demonstrates a simple model that accepts peak inflows derived from a hydrological model. It is assumed that steady flows occur in each pipe reach or link. Hydraulic grade lines are assumed to be located at the obvert (upper inside surface) of pipes and the flow condition is described as “flowing full but not under pressure”. Allowances for local losses are provided by a small drop (up to 90 mm, depending on change of flow direction) within inlet structures. The capacity of pipes can be calculated easily by applying a friction formula such as the Manning Equation and accounting for the grade of the pipe. The conveyance network is assumed to behave as a network of open channels and no allowance is made for upstream or downstream surcharges.

Figure 9.5.18(b) shows a second approach to hydraulic analysis that also assumes steady-state conditions where peak flows occur as pressure flows in pipes and the HGL is located above or along the pipe obvert. This method includes energy losses and pressure changes at inlet structures that are likely to be greater than open channel flow assumptions where water levels are below pipe obverts. Capacity of pipes is also dependent on downstream water levels which may create backwater effects on flows in pipes.

These methods accept peak flow from hydrological models and assume that peak flows occur simultaneously throughout the conveyance network. Flow rates are constant within each link and the calculated HGLs and EGLs represent upper envelopes of these flows. This process will usually estimate lower pipe capacities than unsteady flow assumptions.

Figure 9.5.18(c) presents unsteady flow processes that are created by the inflow to the conveyance network of full hydrographs typically generated in computer models and real rainfall depths and patterns. The simulations account for the changes in water levels and flow characteristics in pipes and the network throughout storm events. These processes include dynamic effects such as fast-travelling waves generated by changes in flow conditions that can create shock losses in the conveyance network. This model is applied using computers that process and solve finite difference computations. A steady flow system is assumed to be independent of time and only requires one set of calculations. However calculations in computer models of unsteady flows are repeated for many time steps and pipe reaches are divided into several sections during the calculation processes.

All three hydraulic models can be utilised for design and analysis. The first and simplest method can be used for design of small networks where downstream conditions may ultimately be varied to account for the actual behaviour of the conveyance network. These adjustments may not be possible for design of a fixed conveyance network, and the estimated capacities and impacts of conveyance network may be incorrect.

The assumed steady-state flows and a connected hydraulic grade line throughout a network of the second method is more suitable for basic design and analysis tasks. This method is likely to provide more efficient designs as it more closely reproduces real hydraulic behaviour and allows for surcharging of pits and pressure flows. This process may be used as a checking procedure by working backwards from the receiving water level towards the top of the catchment.

This model was presented by ARR 1987 (Pilgrim, 1987) as the preferred hand calculation method for hydraulic design of simple pipe networks. Calculations typically involve two iterations for a conveyance network. The first iteration commences at the top of the catchment, accumulating the flows arriving at each inlet structure, and allows for possible bypass flows at pits. The calculated flows through the conveyance network are used to determine the sizes of pipes and the invert levels at their ends whilst ensuring that HGLs do not rise above a limit, usually 0.15 m below the surface level of inlet structures. This design procedure involves a series of trials with increasing pipe sizes selected from the commercially available diameters. The smallest commercially available pipe diameters that meet the design requirements are typically selected.

The iteration of the calculations commence at the outlet using a set tailwater level and project the HGLs upward towards the top of the catchment by considering the HGL slope due to pipe friction and the local pressure changes at each inlet structure. The previously calculated flow rates, pipe diameters and water levels in pits can be used in design charts such as the Missouri and Hare charts to determine local pressure changes at pits (refer to Book 9, Chapter 5, Section 5). When the upstream process of calculations reaches an inlet structure with two or more pipe branches, the calculations progress separately and upstream in each branch.

This projection process can be employed for part-full pipe flows, and for pressurised and full-pipe flows. However, the straight water surface profiles assumed for part-full flows will not be exact. A more accurate procedure is to project water surfaces upstream using the gradually-varied flow methods commonly called backwater curve computations.

Some designers of conveyance networks are still using the simple, steady flow procedures. However the unsteady flow models produce more realistic behaviours in response to hydrographs and flow volumes that are essential for analysis volume sensitive systems that include volume management facilities (refer to Book 9, Chapter 4). Modelling using unsteady hydraulic (and hydrology) assumptions is the preferred method for detailed analysis of conveyance networks that need to respond to strict constraints and where realistic modelling of network behaviour is needed. This approach is also essential for analysis of existing conveyance networks to replicate an existing deficit in performance or to reproduce a known flooding problem. There are many software products currently available that can be utilised for these types of analysis and design (refer to Book 9, Chapter 6).

5.6.2. Overland Flow

Conveyance networks receive and include overland flows. Overland flow is conveyed as sheet flow across land surfaces or in an overland flow path within a channel or swale. Sheet flow is typically produced when rainfall exceeds the volume of depression storages and infiltration capacity of a catchment resulting in overland flows travelling towards a receiving watercourse or an inlet structure in a conveyance network. Overland flows can also be escaping floodwater when the capacity of a conveyance network or watercourse is exceeded.

Overland flow paths typically convey stormwater when the capacity of the minor system conveyance network is exceeded as bypass flows between inlet structures along a kerb and gutter in a street, along swales in rural or grassed areas, or sometimes undesirably through private property. Calculations for overland flows are similar to open channels in that they can be defined as a number of different channel sections with constant cross-sections and slopes. However a key difference between overland flow paths and open channels is that overland flow paths are typically limited to shallower flow depths to meet safe design criteria. Open channels typically convey stormwater at greater depths and flow rates.

Urban stormwater management may combine buffer strips or vegetated swales or bioretention with overland flow paths as cost effective methods to facilitate attenuation of flows and removal of pollutants.

Limitations of Depth and Width of Overland Flows

The depths, widths and velocities of overland flows should be limited to meet objectives for safety and erosion. A range of conditions may be applied when a cross-section of a road is to be used to convey major and minor flows and the limiting factor is deemed to be the most restrictive criteria. These criteria include risks to pedestrians, particularly children, and the importance of the road for transport purposes. The following conditions should apply when guidance from a consent authority is not available:

- The depth of stormwater flows at the kerb (d_g) should be limited to the lower side of a street to prevent uncontrolled overflows from entering properties. For streets with 150 mm high kerbs and a footpath with a substantial slope towards the gutter, a suitable limiting depth may be 200 mm or to the height of a water-excluding hump on a property driveway plus an appropriate freeboard. In addition a maximum width of flow should not be exceeded in the carriageway. Greater depths may be tolerated where a street is significantly lower than the land on both sides and in tropical areas with greater intensity rainfalls. A suitable freeboard should apply to floor levels of habitable rooms in properties adjoining the road.

- The product of depth and velocity ($d_g.V$), with V being the average velocity in the gutter, should not exceed 0.4 m^2/s for safety of pedestrians, 0.3 m^2/s for stability of parked vehicles (depending on size), or as directed by the consent authority (refer to Book 6, Chapter 7).

- Depths of stormwater flows should not exceed the height of the crown of the road during minor storms or where flows are to be contained on one side of a street. This includes locations that include ponding of stormwater such as at sag pits. Depending on the importance of the road (local, collector, arterial) and the importance of access, limits on width of flow of 2 to 2.5 m are typical.

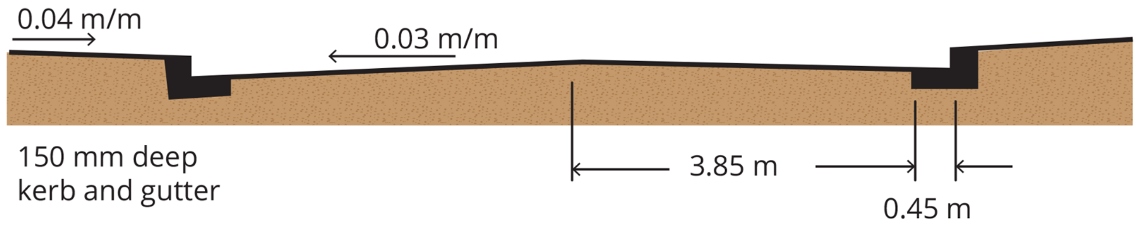
- Widths of flows may be limited to allow clear lanes in the centre of a road for passage of vehicles. Flow depths should not exceed the height of the crown of a road by more than 50 mm for major overland flow paths not considered part of the trunk drainage system and in new development areas.

Dimensions of Flow

The Manning Equation can be used to calculate flows in trapezoidal style overland flow paths. Sheet flow is commonly estimated using a version of the kinematic wave equation for flow distances up to 130 m and then sheet flows are then concentrated into some form of gully or defined overland flow path (NHI, 2013).

Equations for road gutters can be extended to calculate flows along full road cross-sections during major events. For a given flow rate, the normal depth corresponding to steady, established flow can be found by simple iterative calculations using a friction formula such as the Manning Equation. Although these assumptions may not be entirely valid, the errors involved may be generally acceptable.

Design charts of the flow capacities of roadway cross-sections can be prepared using Equation (9.5.8). Allowable zones are defined by various limiting conditions and criteria as shown by the example in Figure 9.5.19.



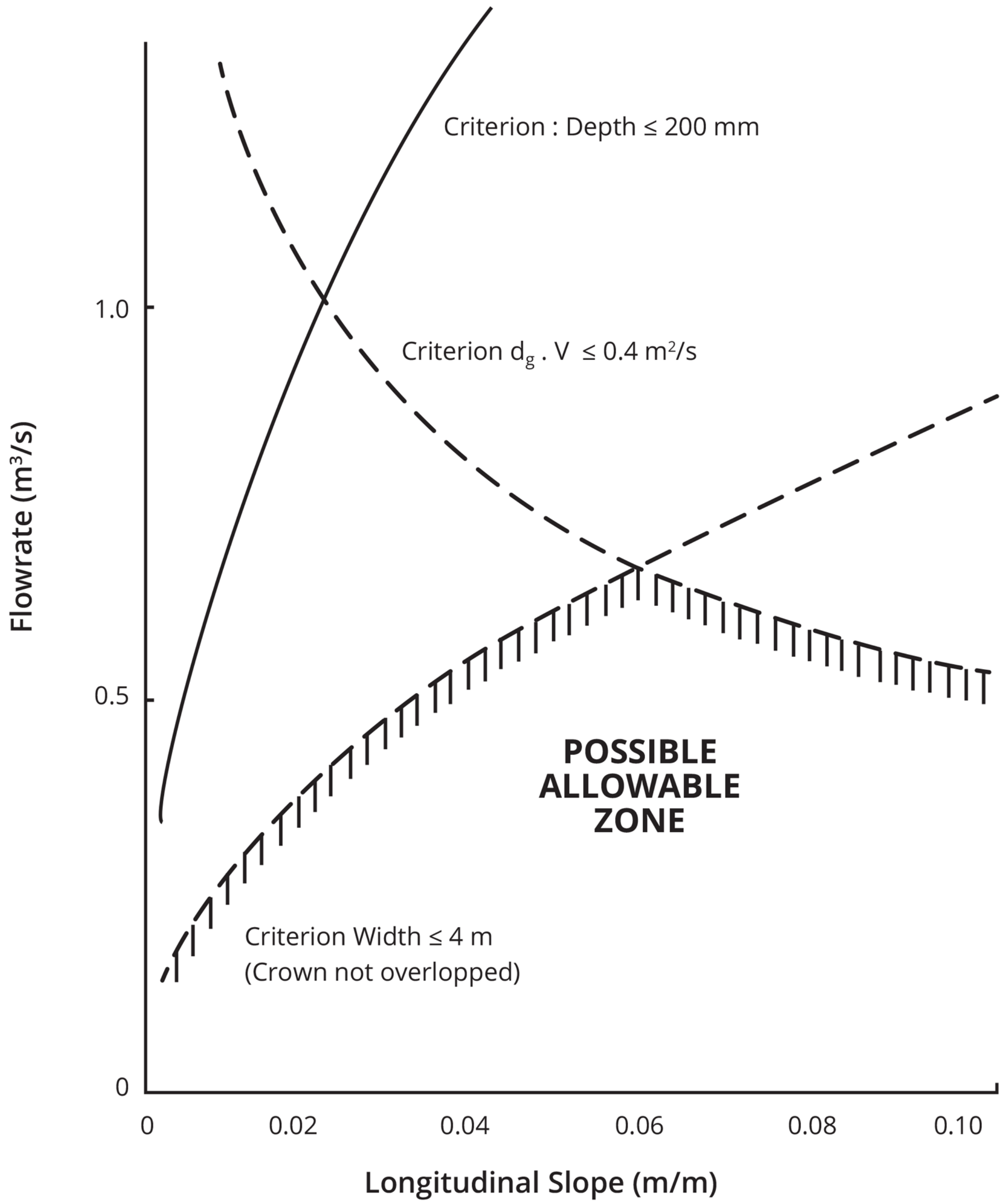


Figure 9.5.19. Flow Capacity Chart for One Side of an 8 m Carriageway with 3% Cross-fall

Gutter and Roadway Flow Equation

The following general equation developed by the U.S. Bureau of Public Roads (Searcy, 1969) is recommended to determine flows in streets. With reference to Figure 9.5.20(a), the equation is:

$$Q = Q_{ABC} - Q_{DBF} - Q_{DEF} - Q_{GEH} = 0.375F \left[\left(Z_g/n_g\right).\left(d_g^{8/3} - d_p^{8/3}\right) + \left(Z_p/n_p\right).\left(d_p^{8/3} - d_c^{8/3}\right)\right].S_0^{1/2} \qquad (9.5.8)$$

where Q (m^3/s) is the total flow rate which is estimated by dividing the section as shown in Figure 9.5.20(a) and applying the equation by Izzard (1946) for a triangular channel with a single cross-fall:

$$Q = 0.375Fd^{8/3}S_0^{1/2}Z/n \qquad (9.5.9)$$

Where:

F is a flow correction factor,

Z_g and Z_p are the reciprocals of the gutter and pavement cross-slopes (m/m),

n_g and n_p are the corresponding Manning's roughness coefficients,

d_g and d_p are the greatest gutter and pavement depths (m),

d_c is the depth of water on the road crown, and

S_o is the longitudinal slope (m/m).

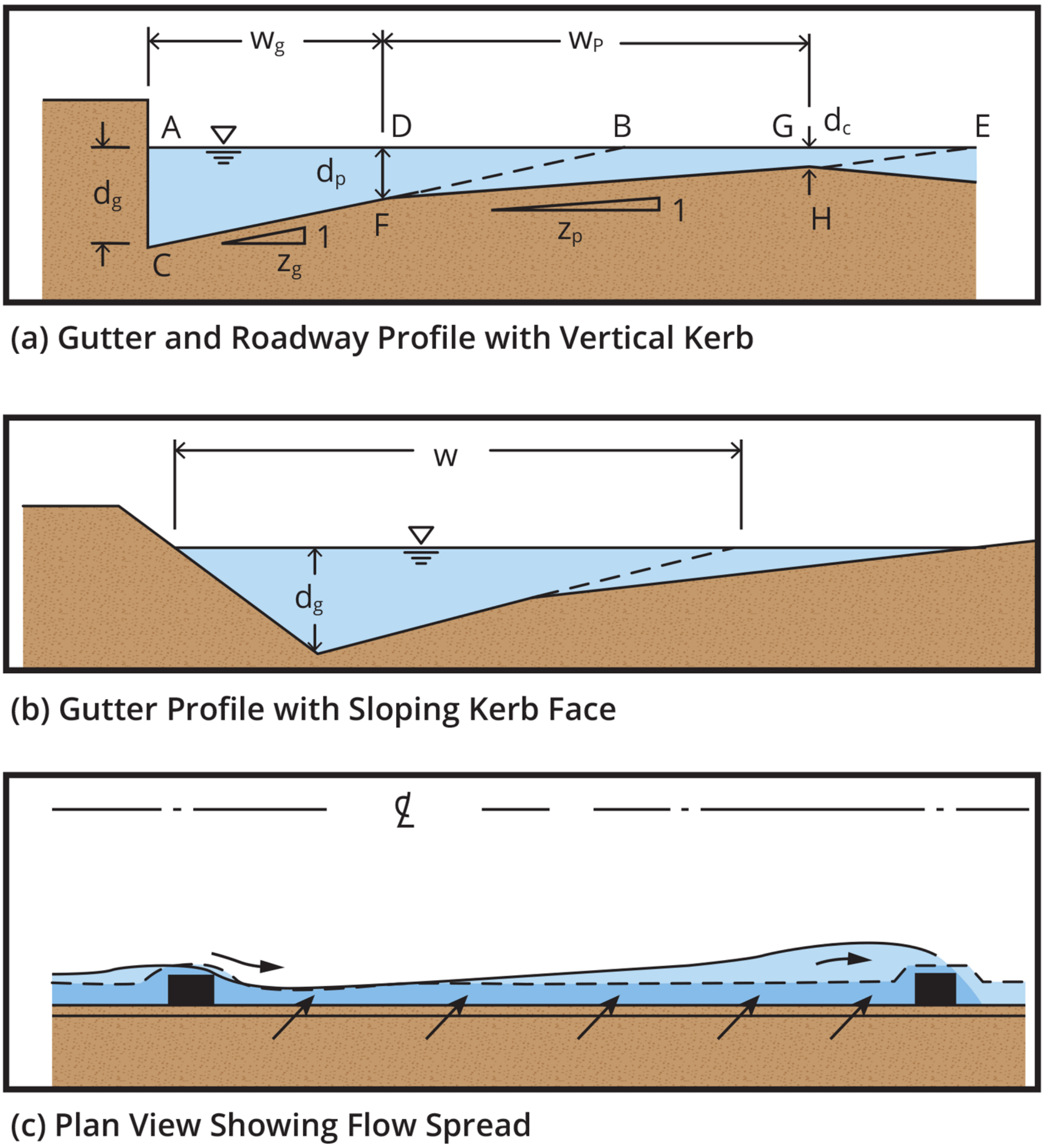


Figure 9.5.20. Gutter Flow Characteristics.

Equation (9.5.8) can be applied in simplified form when flows are contained in a gutter or on one side of a road. Clarke et al. (1981) estimated values for F of about 0.9 for simple triangular channels and 0.8 for gutter sections of the type shown in Figure 9.5.20(a). These assumptions may be used in the absence of more precise information. Typical values of Manning's n are 0.012 for concrete, 0.014 for asphalt, 0.018 for flush seal and 0.025 for stone pitchers (Dowd et al., 1980).

Consider the face of the kerb to be vertical in situations where the face of a kerb is relatively steep. Equation (9.5.8) can be applied to “lay-back” kerbs with sloping faces by assuming that zg is equal to w/dg as defined in Figure 9.5.20(b).

Open channel flow equations, such as the Manning Equation, can also be used to determine flows in lined gutters or unlined drains or swales.

Flow depths and widths for a specified flow rate can be determined using Equation (9.5.8). Velocities are estimated by dividing the flow rate by the corresponding flow area. Travel times for stormwater conveyance can be derived by dividing gutter length by flow velocity. Distributed lateral inflows as shown in Figure 9.5.20(c) can generate flow rates and characteristics such as width, depth and velocity that vary along a gutter. In this situation the average flow velocity occurs at about 60% of the distance along the gutter towards the inlet structure. Gutter flow calculations that use of the total flow arriving an inlet structure will overestimate velocities impacting on the structure.

Other Considerations

Gutter flow times depend on flow rates and it is necessary to specify a time in order to estimate a flow rate. A set of iterative calculations are required. In these calculations, a velocity or time is assumed, and a flow rate calculated. Then a check is undertaken to determine whether the total time of flow in the overland and gutter flow paths agrees with the original assumption.

A precise calculation of gutter flows must allow for concentrated inflows such as bypass flows from an upstream pit at the upper end of the gutter or an outflow from a large site at some point along the gutter. A representative design flow rate must be estimated to permit calculation of the average velocity and travel time.

Parked vehicles and driveways may interrupt and widen surface flows. The limited experimental evidence available suggests that these effects are localised. Allowance for this effect may be needed for streets where close parking of vehicles is likely but specific allowance does not appear necessary at other locations. The design process should account for possible future alterations to gutter and road profiles including resurfacing of roads. Effects of possible pit blockages must be assessed at locations where overflows may cause significant damage.

Aquaplaning or hydroplaning is also an important consideration, especially for highway drainage. This occurs when the tyre’s inability to shed water from the contact patch is exceeded, resulting in a layer of water building between the tyre and road surface leading to a loss of traction that prevents the vehicle from responding to control inputs. Although aquaplaning is dependent on other geometric factors of road design, adequate sizing and placement of inlet structures and cross culvert drainage systems is also a significant factor in reducing the risk of sheet flow occurring on roads. For guidance on aquaplaning, or highway drainage in general, refer to sources such as Austroads (Austroads, 2013), road transport authority guidelines for each state and territory, the FHWA or UK Highway Agency.

It is also important to consider the longevity of an overland flow path and this is especially relevant for flow paths through private property. Blockages are likely to occur due to lack of maintenance, or by post construction modifications such as from garden beds and mulch, or by modifications designed to enclose domestic pets.

It is often necessary to locate structures within minor overland flow paths including property fencing, sound-control barriers and above ground services. When designing overland flow paths that may contain these types of structures it is important to consider the potential for flows to be redirected by these barriers.

5.6.3. The Hydraulic Grade Line (HGL) and Energy Grade Line (EGL)

The hydraulic (HGL) and energy grade line (EGL) concepts are derived from the Bernoulli Equation and assist with the analysis of complex flow problems. The HGL is determined by plotting the relationship for pressure head P/γ and height above an arbitrary datum z at key locations in a conveyance network using the following equation:

$$HGL = P/\gamma + z \tag{9.5.10}$$

Where P is pressure and γ is specific density of water.

Similarly, the EGL adds the velocity head $V^2/2g$ to the HGL to provide a relationship for EGL that can be derived at key locations in the conveyance network:

$$EGL = V^2/2g + P/\gamma + z \tag{9.5.11}$$

Where V is the average velocity in a conduit and g is gravity.

The vertical distance to point (such as the centre of a conduit) below the HGL represents the pressure head or pressure energy at a point. Negative heads or partial vacuums may occur at siphons and the conduit is above the HGL. The HGL coincides with the water surface for open channel flows, except at points such as brinks of weirs where non-hydrostatic conditions prevail. Water rises to the level of the HGL in an inlet structure (pit) that acts as a vertical riser.

The EGL is located above the HGL and represents the total energy (velocity + pressure + potential) available to the flow that is expressed as a height (metres) equivalent to flow energy per unit weight in joules (or newton-metres) per newton.

Grade lines typically slope downwards in the direction of flow in conveyance networks and slope represents energy losses due to pipe friction. The HGL and EGL are parallel for steady flows. The grade lines generally have a different slope to the pipe in closed conduits under pressure (with the HGL above the pipe). The grade-lines are parallel to open channels that are subject to steady and uniform flows since the friction loss equals the potential energy loss represented by the slope of the conduit.

Changes in the shape or direction of conduits create turbulence and local losses that are represented as sharp drops in EGLs. Significant energy losses are typically assumed to act at the centre of inlet structures in analysis of conveyance networks. The HGL is also assumed to change at the centre of inlet structures as illustrated in Figure 9.5.21. These assumptions differ from the actual location of losses at the entry and exit of inlet structures (pits).

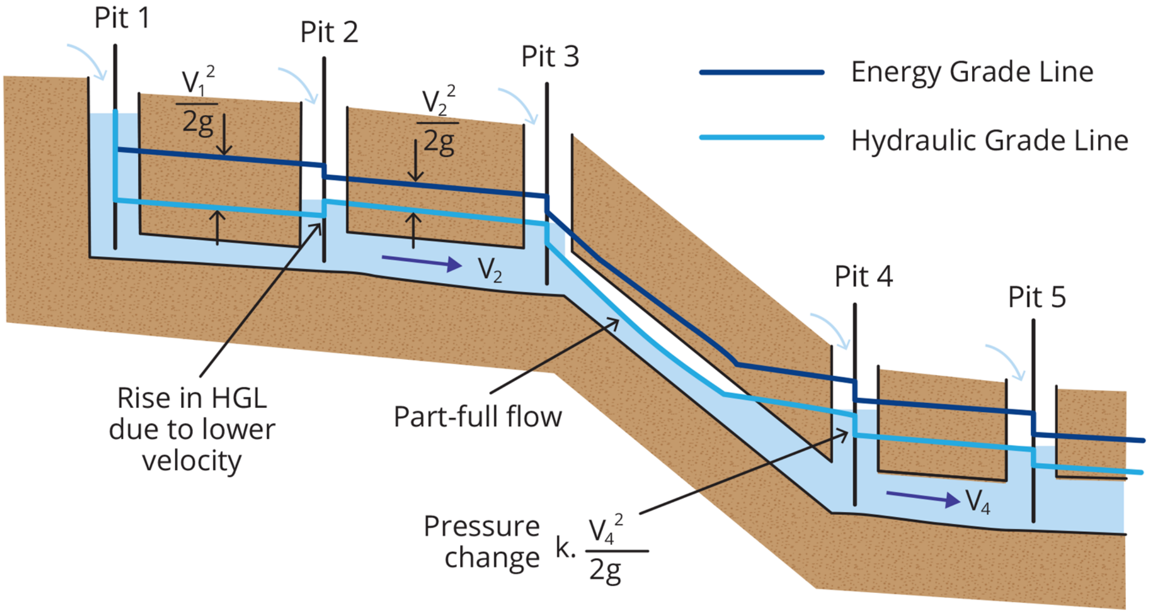


Figure 9.5.21. Flow Behaviour in a Surcharged Pipe System Showing Energy Grade Lines and Hydraulic Grade Lines

5.6.4. Flows Through Conveyance Networks

Local Losses

Changes in the shape or direction of conduits can create turbulence and local losses that are represented as sharp drops in the EGL. Losses occur at entrances and exits to pits, pipe bends, and at contractions, expansions, junctions, and valves in conveyance networks. Except for expansions and contractions of conduits, these losses have the following relationship:

$$h = k \, . \frac{V^2}{2g} \tag{9.5.12}$$

where h is the loss in m, and k is a loss coefficient multiplied by the velocity head of the downstream flow.

The loss factor k is dependent on the geometry of entrances to a conduit. A square-edged entrance will usually have a factor of k_e = 0.5 and for a rounded entrance the factor is approximately 0.2. The factor at a pipe exit k_{exit} is usually 1.0 as it is assumed the entire kinetic energy of flows will be lost as the pipe discharges into a larger body of water or atmosphere.

The losses at bends depend on the radius of the bend and have a typical value of k_b = 0.5. Contractions in conduits (decreases in pipe diameter) are subject to low levels of losses with a typical factor k_c of 0.05. Expansions in conduits (increases in diameter) generate higher losses h_L that are dependent on the upstream V_u and downstream V_d velocities:

$$h_L = K_{\exp} \, . \frac{(V_u - V_d)^2}{2g} \tag{9.5.13}$$

where k_{exp} is about 1.0 for abrupt pipe expansions.

Valves have variable loss factors which can become very large as the valve closes.

Full Flows in Conduits

The estimation of flow rates through conveyance networks that are flowing full is made by relating the available energy or head to the losses as expressed by the velocity head. The following calculation shows how flowrates can be determined from the available head and the assumed energy losses along a 300 mm pipe discharging from a reservoir as shown in Figure 9.5.22.

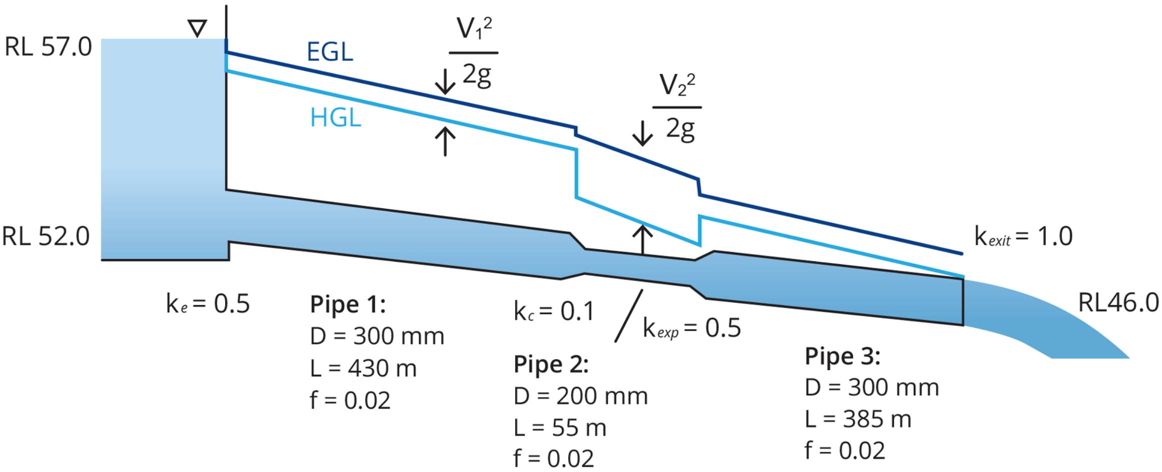


Figure 9.5.22. Example of Full Flows in a Pipe

The pipe diameter reduces from 300 mm to 200 mm at the middle of the pipe branch. The energy loss at the following expansion is assumed to be 0.5 times the velocity head in the downstream pipe and all friction values (f) in the Darcy-Weisbach Equation are set at 0.02.

The water level in the reservoir is 57.0 m above a height datum and the total head available is 57.0 - 46.0 = 11.0 m. The various losses are all functions of the velocity heads in the pipes. Since $V_3 = V_1$ and $V_2 = V_1 . \frac{A_1}{A_2} = V_1 . \left(\frac{D_1}{D_2}\right)^2$, the sum of the losses will be:

$$\left(k_e + \left(f . \frac{L}{D}\right)_1 + k_c + \left(f . \frac{L}{D}\right)_2 . \left(\frac{D_2}{D_1}\right)^4 + k_{\exp} + \left(f . \frac{L}{D}\right)_3 + k_{exit}\right)\frac{V_1{}^2}{2g} = 11 \text{ m} \tag{9.5.14}$$

$$\left(0.5 + \left(0.02.\frac{430}{0.3}\right) + 0.1 + \left(0.02.\frac{55}{0.2}\right) . \left(\frac{0.3}{0.2}\right)^4 + 0.5 + \left(0.02.\frac{385}{0.3}\right) + 1.0\right)\frac{V_1{}^2}{2g} = 11 \text{ m} \tag{9.5.15}$$

Thus,

$$
\begin{aligned}
&84.28 \cdot \frac{{V_1}^2}{2g} = 11 \text{ m}, \\
&V_1 = \left(\frac{11.0 \times 19.60}{84.28}\right)^{0.5} \\
&= 1.60 \text{ m/s and} \\
&Q = V_1 A_1 \\
&= {}^{\pi}\!/_4 \cdot (0.3)^2 \cdot 1.60 \\
&= 0.113 \text{ m}^3/\text{s}
\end{aligned}
\qquad (9.5.16)
$$

The Manning Equation can also be used with friction losses expressed by $\left(2g\, n^2 \frac{L}{R^{4/3}}\right) \text{x} \frac{V^2}{2g}$ since slope of the energy gradeline is S = h^f/L.

Equations using conservation of mass, energy and momentum can be constructed to describe the state of an entire conveyance network that includes multiple pipes and inlet structures. These equations are solved to provide information about the pressures and velocities throughout a conveyance network which can be visualised as EGLs and HGLs. More complex partial differential equations are required to cope with unsteady flows that change with time.

Conduits Flowing Partially Full

Conduits that are flowing partially full in stormwater conveyance networks can exhibit complex behaviours. A maximum flow capacity is achieved when conduits are operating at less than full flows. However it is not good practice to design conduits with partial flows as disturbances may eliminate free surfaces in conduits and cause a transition to pressurized full flows that may lead to surcharges.

This assumes that flows in conduits are open channel flows with atmospheric pressure at the surface. Submergence at the entrance and tailwater levels affecting the outlet of conduits generates further complications. In addition, large air bubbles and air pockets can occur in conduits that operate in partially full conditions resulting in pressures that can be above or below atmospheric pressure. The theory of open channel hydraulics is addressed in Book 6.

Complex Procedures

A more complex and correct procedure for analysis of conveyance networks is to apply partial differential equations of unsteady flow varying in space (the distance along a conduit) x and time t that is defined as steps or intervals. These numerical models divide river, channel or pipe reaches into segments and define the transfer of mass and momentum between adjacent segments using the *Saint Venant Equations* for conservation of mass and momentum in unsteady flows as described in Book 6, Chapter 2. The equations must be solved iteratively using finite difference or finite element models and matrix calculations that may require longer computing times.

These more complex calculation processes are quite different from water surface projection methods such as the ‘standard step’ procedure. Nevertheless the same outputs are produced such the HGL levels at points along a conduit and at different times during a flow event. The equations allow for pipe friction and local losses, and also incorporate pressure changes at inlet pits and junctions.

Modelling of urban conveyance networks is typically carried out using a range of computer software packages that provide different levels of rigour or precision which involve trade-offs between speed and accuracy. However the designer should be aware of other important considerations such as stability. Unlikely high or low pressures, water levels, and flow rates are generated when iterative calculations become unstable. The usual way of achieving stable results with a computer model is to choose a shorter time step or adjustment of factors affecting the relative time steps in space and time. Small errors in volumes or flows (typically < 1%) can be accepted in order to achieve faster running times.

Priessmann Slot

Methods of analysis must allow for flows that change from partially full to full conduit flows and back again. Modelling procedures that account for unsteady flow regimes employ the Priessmann Slot assumption. This mixed flow problem is simplified by the addition of a hypothetical slot in the pipe which allows the depth of flow to exceed the pipe diameter and provides pressurized flow effects (Yen, 1986; Butler, 2004). The width of the slot must not be too wide to significantly impact on continuity and should be determined to ensure that the gravity wave speed equals the pressure wave speed.

The hypothetical slot allows the analysis of the conveyance network to be treated as an open channel flow problem. However, a limitation of this approach is that it cannot accurately simulate the formation and impact of air pockets or negative pressures results from shocks.

Outlet Structures

Regardless of whether flow within the conduit is full or part full, suitable transition is required at the end of a conveyance conduit, where flow discharges to the receiving environment. The transition structure, or outlet structure should accommodate potential for high velocity and/or turbulent flow. This can be achieved through armouring of the surface using material such as rock or concrete, along with gradual transition of geometry from that of the conduit, to that of the receiving channel or basin. Energy dissipation and/or flow dispersion can be achieved at the same time using appropriate outlet structure design. This is particularly necessary where stormwater settlement processes are expected in a receiving basin structure such as a bio-retention basin or constructed wetland.

The outlet structure may also represent an opportunity for removal of gross-pollutants prior to stormwater passing into a receiving channel or structure. This can be achieved using various forms of screens, baskets and mechanical filters. The impact of these structures, whether clear, partially blocked or fully blocked on the hydraulic performance of the conveyance infrastructure needs careful assessment at the design stage.

5.6.5. Culverts

The simplest conveyance network is a single-pipe culvert which is a common component of highway and railway networks that is located wherever an embankment crosses a waterway or drainage path. These transport crossings may only involve a single pipe (or multiple parallel pipes). However, the hydraulic calculations can be complicated. Culvert hydraulics are comprehensively described by Normann et al. (2005). The treatment of culvert hydraulics (or headwalls) is divided by two flow conditions:

1. Inlet controls – dependent on the orifice effect at the culvert entrance; and

2. Outlet controls – dependent on full, pressurised flow conditions through the pipe or on high tailwater levels.

Multiple culverts may be connected by pits or junctions in a similar manner as a large conveyance network (refer to Book 9, Chapter 5, Section 6)

Inlet Control

Inlet conditions for culverts are created by the vena contracta effects shown in Figure 9.5.23.

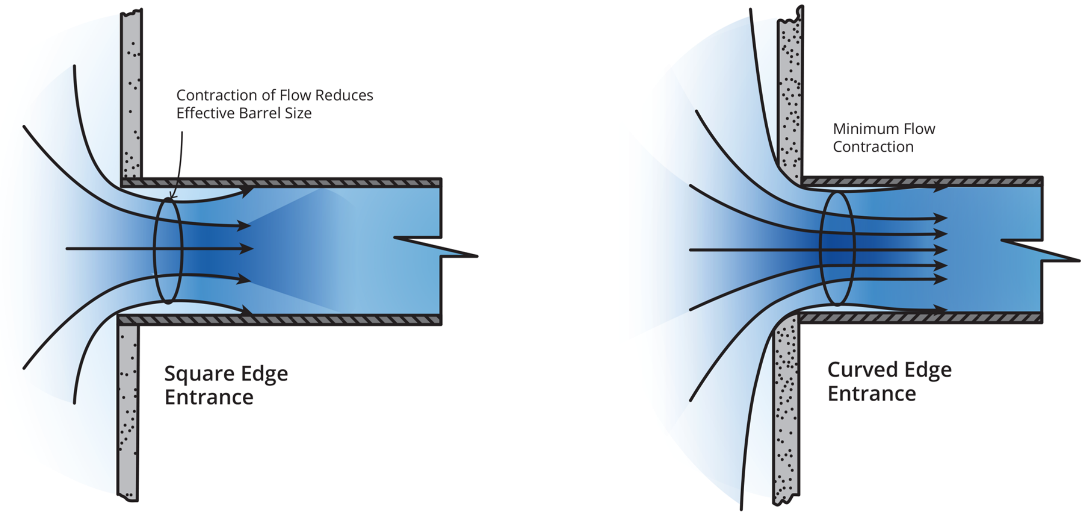


Figure 9.5.23. Vena Contracta or Contraction at a Culvert Entrance

The streamlines of flows entering a culvert cannot turn abruptly and the curvature of flows continues into the culvert creating a jet with a diameter less than that of the culvert. This process reduces the available cross-sectional area of flows and the overall flow rate. The ratio between the jet and the pipe diameters is 0.6 for a square-edged entrance. Values for other entrance types are shown in Figure 9.5.24.

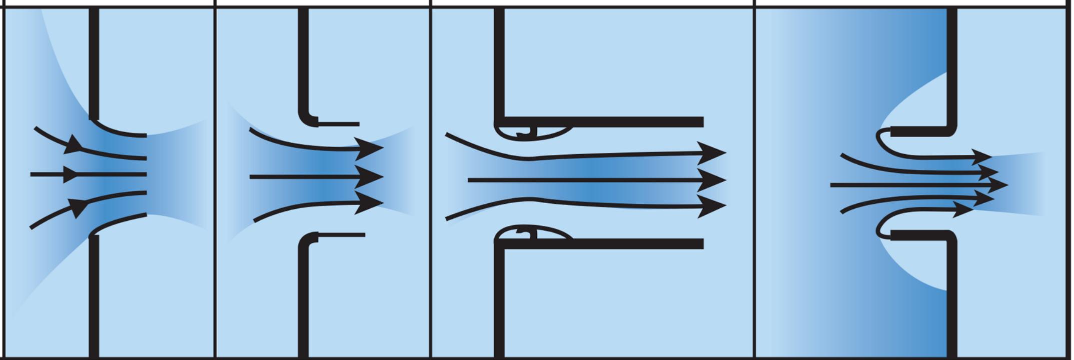
Orifices and their Nominal Coefficients				
	Sharp edged	Rounded	Short tube	Borda
				
C	0.61	0.98	0.80	0.51
C_c	0.62	1.00	1.00	0.52
C_u	0.98	0.98	0.80	0.98

Figure 9.5.24. Orifice Coefficients (Vennard and Street, 1982)

The correction coefficient for the reduced area is C_c and C_u is the factor for the velocity being less that the theoretical value of $V = \sqrt{2gh}$ where h is the pressure head on the orifice (m) and g is the acceleration due to gravity (m/s^2). The overall correction coefficient is $C = C_c.C_u$.

The general case of inlet control is presented in Figure 9.5.25 where it is observed that the culvert barrel has a greater capacity than the entrance as it is flowing partially full. As indicated, Figure 9.5.24 shows that the capacity of the culvert can be improved by modifying the entrance by rounding sharp edges and changing the streamlines. These improvements may be useful in situations when additional capacity is required.

The general equation governing orifice flow for a circular pipe is:

$$Q = VA = C\left(\frac{\pi D^2}{4}\right)(2gh)^{0.5} \tag{9.5.17}$$

where C is the correction factor (dimensionless),

D is the pipe diameter (mm),

h is the head on the orifice, usually taken from the upstream water surface to the centre of the orifice (m), and

g is gravitational acceleration (9.81 m/s^2).

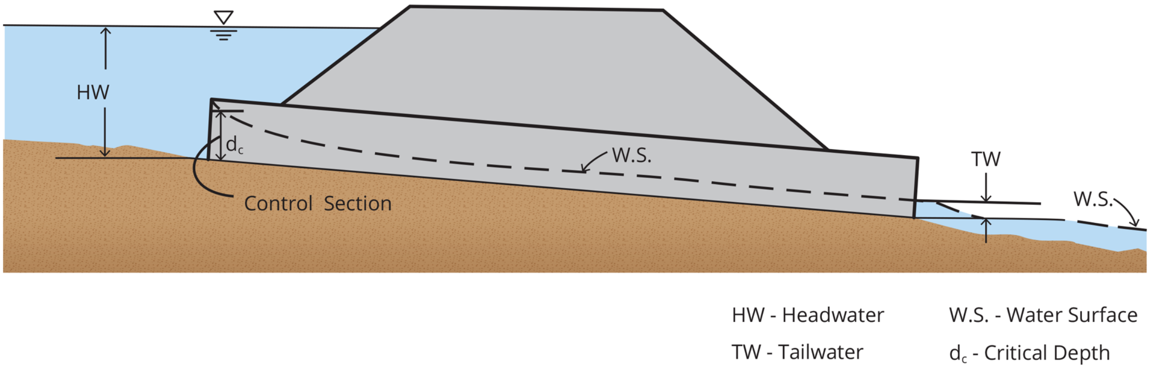


Figure 9.5.25. Example of Inlet Control (U.S. Department of Transport, 2005)

The hydraulics is more complicated when the entrance to the culvert is not completely submerged. This may involve three different states depending on the headwater height above the invert HW and the culvert diameter or height D:

- Partially full flow for HW < 0.8D is a weir type flow as water pours into the pipe;
- Partially full flow with 0.8 < HW < 1.2D is similar to weir flow; and
- Fully submerged inlet flow for HW > 1.2D is an orifice flow.

The stated limits of 0.8D and 1.2D are approximate. These three zones lead to the behaviour demonstrated in Figure 9.5.26 where the inlet control relationship changes depending on the headwater elevation. It is also possible to have two different flow rates at the same water elevation which depends on whether the culvert is operating as an inlet or outlet controlled system. These states can also depend on whether flows are increasing or decreasing.

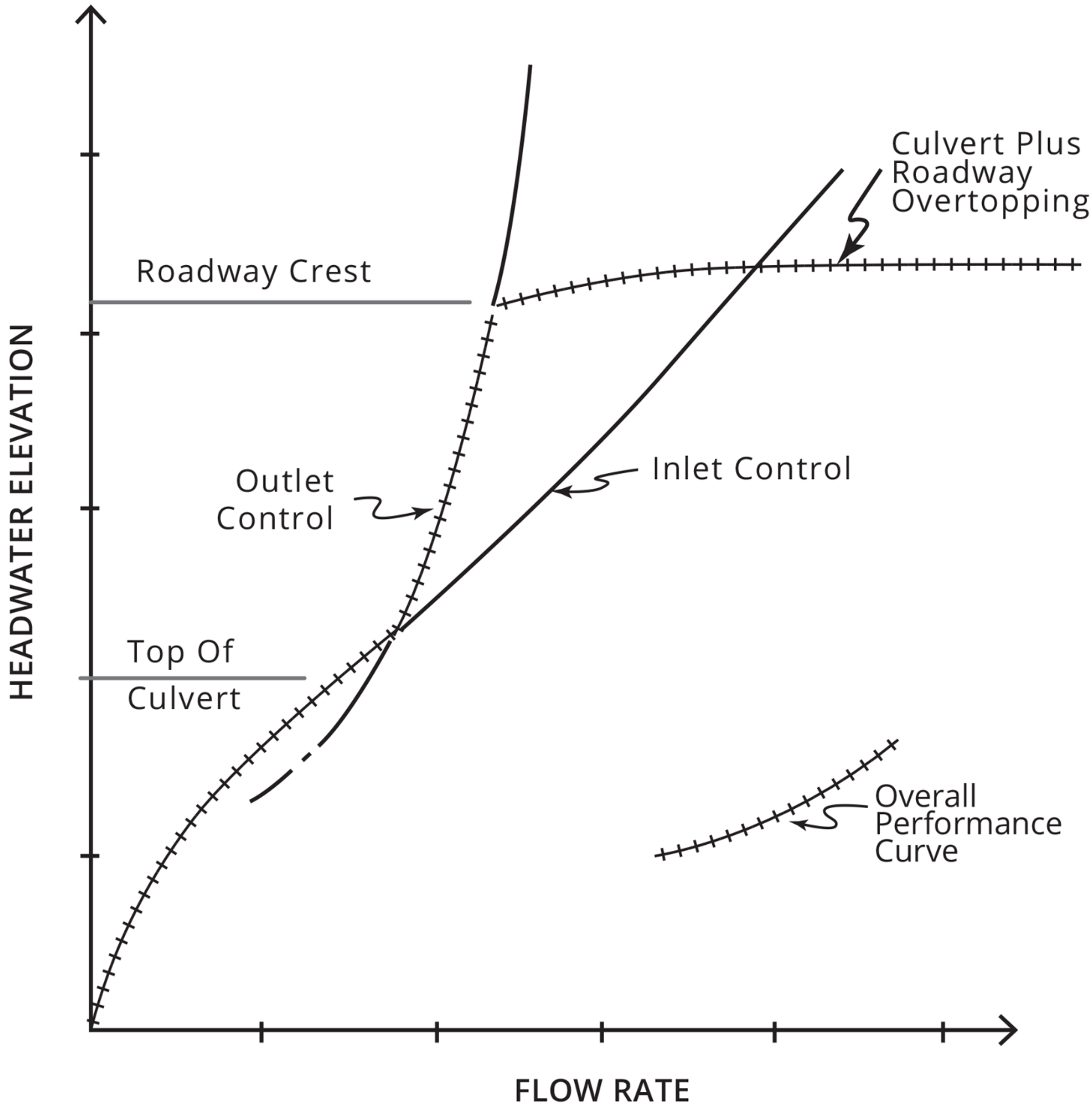


Figure 9.5.26. Inlet Control versus Elevation of Headwaters (U.S. Department of Transport, 2005)

A range of design aids are generally available in the form of nomographs used to calculate headwater levels for various situations involving circular, box and other types of culverts. A better approach is to use computer software to model culvert hydraulics.

Outlet Controls

Outlet control occurs when a culvert is not capable of conveying as much flow as the inlet can accept. The controlling section is generally at the culvert exit where subcritical or pressurised flow conditions are occurring or further downstream of the culvert due to tailwater conditions. Two outlet-controlled situations are provided in Figure 9.5.27. The difference between upstream headwater and the tailwater levels drives the flows through the culvert. Energy losses are added and equated to the available head.

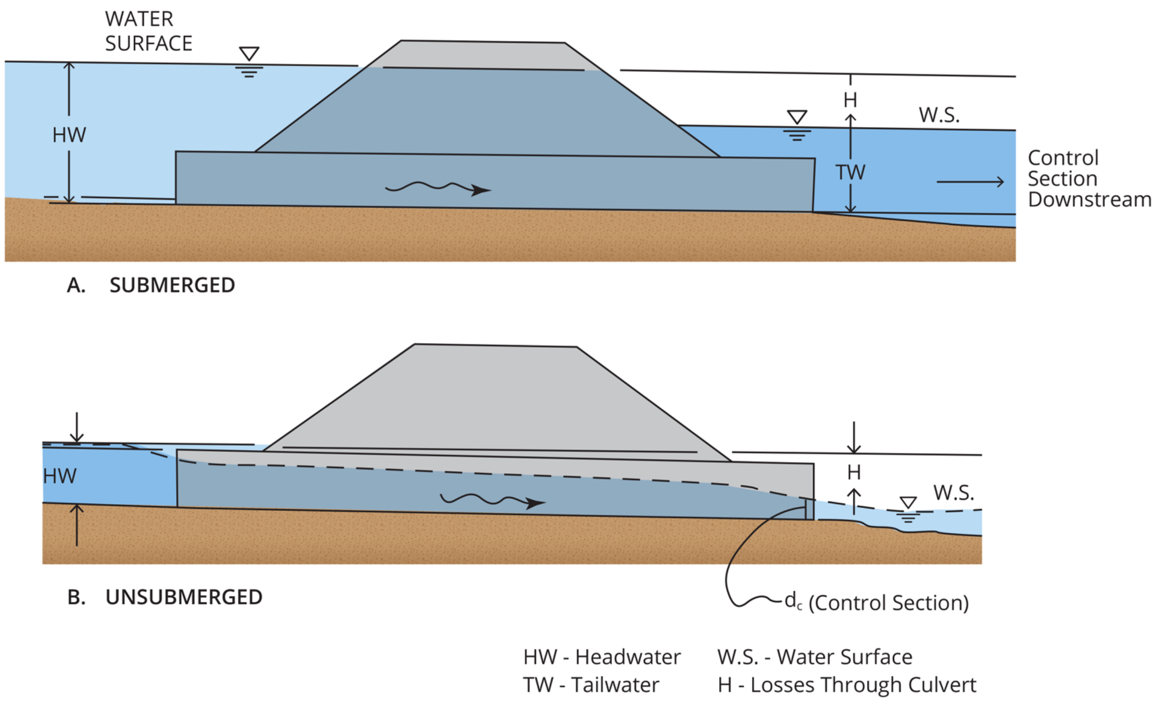


Figure 9.5.27. Example of Outlet Control Situations (U.S. Department of Transport, 2005)

These calculations involve backwards projection of the HGL that commences at the tailwater level if this submerges the outlet. Different computer models make various assumptions for free outfalls. It is assumed that the level will be half way between the pipe obvert and the critical depth, and it is necessary to determine that critical depth from nomographs or equations. However other computer models assume that it is the lower of (a) the critical depth and (b) the normal depth.

A weir equation is applied to allow for overtopping of road embankments:

$$Q = C_w L_w H^{1.5} \tag{9.5.18}$$

where C_w is a weir coefficient, depending on the weir shape (Figure 9.5.28),

L_w is the width or length of the weir perpendicular to the direction of flow, and

h is the height of water above the weir crest (m).

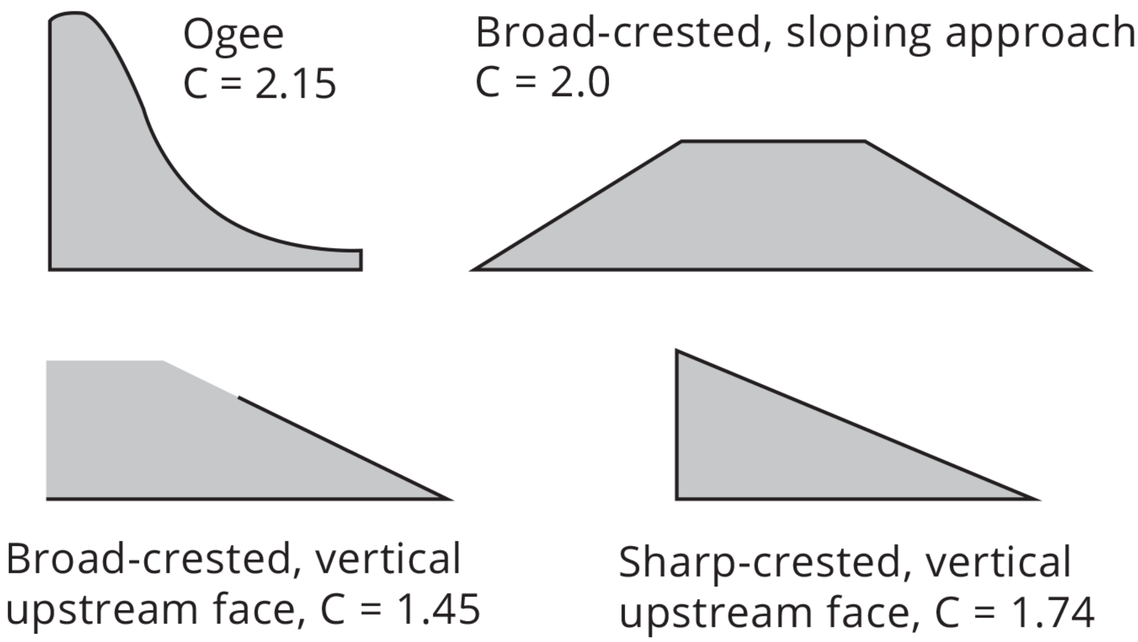


Figure 9.5.28. Shapes of Weir Crests (Laurenson et al., 2010)

Culvert and overflow weir outflows can be combined into a composite relationship as shown in Figure 9.5.26. This calculation should account for inlet and outlet controls and usually the most conservative relationship that provides the lowest flow rate for a given depth is accepted.

The real behaviour of a culvert is more complex and involves a phenomenon called 'priming'. As upstream water levels rise, culverts tend to remain under inlet control until they run full. As upstream water levels decline, culverts tend to remain at full flows in an outlet control configuration until there is a sudden reversion to inlet control and decline in headwater level.

Since culverts are often used as outlets for detention basins and conveyance networks. The relationships presented above can be applied to specify the elevation and discharge relationships needed for routing of flows through volume management facilities.

5.7. References

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Chapter 6. Modelling Approaches

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The authors collaborated with Mikayla Ward and Sophia Buchanan to produce the Brownfield and Greenfield case studies.

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6.1. Introduction

Urban stormwater management responds to an increasing number of performance objectives including to mitigate property damage, avoid risks to human life, enhance the amenity of urban settlements, and protect surrounding environments (refer to Book 9, Chapter 3, Section 3 and Book 9, Chapter 5, Section 2). This involves consideration of the full spectrum of rain events, from frequent to rare (refer to Book 9, Chapter 3), from the perspective of flooding, water quality, provision of infrastructure, protection of environments, and enhancing amenity of urban areas. The assessment of urban stormwater behaviour, performance against objectives and associated design tasks, involves complex analytical problems that are better resolved using a computer-based model system.

A computer model involves use of software or a complex spreadsheet. Compared with hand calculation, computer models permit rapid numerical calculation across large spatial and temporal domains, while facilitating testing of multiple suites of parameters and inputs (refer Book 9, Chapter 3, Section 4). This in turn allows the model to be calibrated to best represent the real world conditions that are under assessment. Models can be a useful tool to assist our thinking, and can be readily documented and reviewed, ultimately leading to better assessments and design outcomes.

Reliable estimates are nevertheless conditional upon best practice application of the computer model. It is important to remember that models are only tools to guide our thinking about design and management. The purpose of this chapter is to provide guidance on the selection and application of modelling approaches within urban catchments, having regard to the techniques described in other books of ARR. The chapter is structured as follows:

- Book 9, Chapter 6, Section 2 describes tasks that are characteristic of urban modelling.
- Book 9, Chapter 6, Section 3 discusses current trends in urban modelling. This may assist with planning a long-term strategy for technology adoption, research, and training. A general description of the types of computer models commonly applied in urban stormwater practice is provided as an aid for model selection.
- Book 9, Chapter 6, Section 4 provides a framework for application of computer models to urban stormwater catchments. This discussion includes guidance for each segment of the catchment, from the watershed, through the urban stormwater network, and into the receiving waterway.

This chapter is not intended to duplicate content in other chapters. Where relevant detail is available elsewhere, references to other books and chapters are provided.

In the context of this chapter, an 'urban model' can be defined as a conceptual or computer-based modelling system that performs hydrologic, hydraulic, water balance, or water quality calculations, across a catchment significantly disturbed by urban development and associated infrastructure. This modelling system may operate across all the significant scales of urban areas from allotment to neighbourhood to precinct to region. Urban infrastructure of most direct relevance includes increased impervious surfaces, modification to natural conveyance areas (e.g. pits, pipes, and open channels), and volume management infrastructure (e.g. rainwater tanks, bioretention, and basins).

Emerging urban stormwater analysis and solutions are based on a systems approach that incorporates multiple linked scales (Book 9, Chapter 3). The USEPA (2008) highlights that past practice of designing individual items of stormwater infrastructure at a single centralised scale has been inadequate for managing urban flooding and water quality in waterways. Stormwater management needs to be designed as a system that integrates structural and non-structural attributes of design with site characteristics and performance objectives.

More recently, the USEPA (2008) established that green infrastructure solutions distributed at multiple scales throughout urban catchments partially disconnected impervious surfaces. They also contributed to improved stormwater quality and avoided flood damages (Atkins, 2015). These insights are consistent with earlier Australian applied research finding that both the peak flows and volumes of stormwater runoff are required for the design of stormwater infrastructure (Goyen, 1981), and the local scale was the basic building block of cumulative urban rainfall runoff processes (Goyen, 2000).

Many methods for modelling stormwater runoff are based on regional scale assumptions and processes. However, inclusion of local scale processes in analysis improves knowledge of within catchment outcomes and whole of catchment responses.

It is suggested that a catchment with less than 10 percent impervious surfaces, or with less than 10 percent of the natural conveyance areas modified, would not be considered an 'urban catchment'. In which case, the advice in this chapter may have less relevance. However, each catchment is different, some natural or rural catchments contain sub-catchments that are urbanised (for example, in semi-urban areas). The relevance of this chapter to a specific modelling investigation needs to be determined by the reader through application of judgement and experience.

6.2. Urban Modelling Tasks

Typical urban modelling tasks are introduced in this section to establish context for subsequent discussion. In particular this section focusses on those modelling tasks that are not typically required when modelling rural and natural catchments. This section should be read in conjunction with Book 5, Book 6 and Book 7 where the reader can find information about modelling tasks and assumptions that are common across all catchment types (i.e. urban, semi-rural, rural, and natural).

There are some important differences between modelling of urban catchments compared with modelling other types of catchments. Urban areas can include:

- A larger proportion of impervious surfaces (refer Book 9, Chapter 6, Section 2).
- Stormwater conveyance infrastructure. This includes a network of inlet structures and non-natural flow paths that provide for greater concentrations and velocities of flow (refer Book 9, Chapter 6, Section 2).

- Numerous hydraulic structures. This includes infrastructure for waterway crossings, temporary storage of volume, water harvesting and treatment of runoff (refer Book 9, Chapter 6, Section 2, Book 9, Chapter 6, Section 2 and Book 9, Chapter 6, Section 2).

- A greater variety of land uses at different scales with different connectivity to catchment outlets (refer Book 9, Chapter 6, Section 4).

The density of land uses and associated infrastructure within an urban catchment also changes with time. The urban modelling process must therefore consider the information needs of the stakeholder and ensure the temporal scenarios being modelled are relevant.

There are also differences relating to the availability and use of model input data. Modelling in urban areas has intensive requirements related to representation of urban form, land uses, and stormwater infrastructure. Therefore, collection and collation of input data can become a significant component of the overall urban modelling task (refer Book 9, Chapter 6, Section 2).

6.2.1. Impervious Surface Estimation

One of the defining characteristics of an urban catchment is the presence of impervious surfaces such as roads, buildings, footpaths, and driveways. These surfaces have an associated reduced infiltration loss and decreased lag in hydrologic response in comparison to pervious surfaces (i.e. landscaping, lawns, open space) or natural catchments. Book 4, Chapter 2, Section 7 provides further discussion of the effects of impervious cover on runoff from urban areas.

Hydrologic modelling of urban areas requires an estimate of the proportion of impervious surfaces across each catchment and sub-catchment to be modelled. As described in Book 5, there are two main types of impervious surfaces that exist within urban areas:

1. Impervious areas which are directly connected to the conveyance network or urban waterway – referred to as Directly Connected Impervious Areas (DCIA).

2. Impervious areas which are indirectly connected to the conveyance network, typically where impervious surface runoff flows over pervious surfaces before reaching the conveyance network (e.g. a roof that discharges onto a lawn). These are referred to as Indirectly Connected Impervious Areas (ICIA). Alternatively, the responses of these impervious surfaces are disconnected from sub-catchment outlets by volume management measures (refer Book 9, Chapter 4).

These two configurations of impervious surfaces provide different hydrologic responses with Directly Connected Impervious Areas contributing to runoff more quickly than Indirectly Connected Impervious Areas (refer Book 5, Chapter 3, Section 4).

For large urban catchments, isolating the separate hydrologic effects of these two types of impervious surfaces is challenging. Book 5, Chapter 3, Section 4 instead describes a concept referred to as Effective Impervious Area (EIA) that encompasses the combined hydrologic effect of both directly and indirectly connected impervious areas. The estimated EIA value for a catchment is calculated and then applied to hydrologic calculations using the adopted modelling software.

The approach described in Book 5, Chapter 3, Section 4 involves estimation of EIA via linear regression of site stream flow gauge and rainfall data. In situations where there is insufficient available data to allow this technique to be used, the ratio of EIA to Total Impervious Area

(TIA) has been established for a collection of gauged catchments that allows EIA to be estimated based on an estimate of TIA (Refer Book 5, Chapter 3, Section 4).

TIA is a measurable catchment feature that is typically estimated using GIS methods (refer Book 5, Chapter 3, Section 4). The selection of a technique for estimation of TIA will depend on catchment scale, data availability, accuracy requirements, and whether the catchment scenario being investigated relates to an existing or future condition.

From Book 5 the recommended ratio of EIA/TIA for the majority of urban catchments sits within the range of 50% and 70%. For example, if the TIA for an urban catchment was measured to be say 55% then the EIA for that same catchment would be somewhere between 27.5% and 38.5% of the total catchment area.

However, when the EIA approach is used, it is important that the characteristics of the catchment under investigation are compared to those of the catchments that have been used to establish the recommended EIA/TIA ratio. Different catchments have different stormwater management standards and land use patterns that may alter the overall degree of connectivity between impervious surfaces and the drainage network serving the catchment. Where there is higher connectivity, the EIA is also expected to be higher.

For some catchment investigations where there is strong connectivity between the impervious surfaces and the downstream drainage system, the measured TIA value may be the more suitable impervious surface value to be used for hydrologic modelling purposes. For example, the analysis of a sealed carpark surface, where the entire impervious area is directly connected to surface inlets, is more appropriately undertaken using a TIA estimate.

Also, where the scale of the catchment is small, for example an individual parcel of land or a small development site, the use of TIA values in conjunction with a sub-catchment definition that reflects actual stormwater connectivity may be more appropriate. To avoid over estimation, designers should only use TIA for small scale catchments when they are satisfied that all the impervious flow is directly connected. The effect of any volume management infrastructure should also be explicitly reflected in these model simulations.

Consideration also needs to be given to the overall need for accuracy when deriving estimates of impervious cover. The majority of techniques applied by designers typically under or over-estimate actual impervious cover by between 10 and 20 percent (Roso et al., 2006).

Predicted peak discharges and runoff volumes are sensitive to error in impervious cover when modelling low rainfall events with both event based and continuous simulation models. Roso et al. (2006) observed that a difference in impervious surfaces of +/- 10 percent from actual conditions, can result in typical errors of 13% in peak discharge and 25% in runoff volume. These errors decrease in situations where rainfall depths are higher and infiltration losses less significant.

It is also noted that where a catchment has significant impervious cover, the variability of runoff is reduced in comparison to a similar pervious catchment since infiltration losses have less influence.

Additional discussion of configurations of impervious surfaces is provided in Book 9, Chapter 6, Section 4. Local observations or information about connectivity of impervious surfaces should be applied in models wherever possible.

6.2.2. Conveyance Infrastructure

Urban areas typically contain a significant amount of stormwater conveyance infrastructure, including numerous stormwater inlet structures feeding a network of other conveyance infrastructure such as street gutters, pipes, open channels, roadways, and overland flow paths through open spaces. These are linked together to form a continuous and distributed network from source to receiving waterway (refer Book 9, Chapter 5).

While natural waterway conveyance is increasingly sought as a design objective for new urban areas, traditionally urban drainage systems have been designed to transfer runoff quickly and within a minimum corridor, often partly underground. This containment of flows within conduits that have artificial linings and unnatural slope leads to faster average flow velocity, greater volume, and significantly altered flood hydrographs compared to those from comparably sized rural and natural catchments.

In order to accurately represent the hydrologic and hydraulic behaviour of an urban area, the influence of conveyance infrastructure on routing and flow behaviour should be included within the adopted urban flood model. For most applications, the model should be capable of describing the effect of conveyance infrastructure on flow characteristics such as flow depth, velocity, direction, surface level, and the hydraulic grade line showing hydraulic losses including their position and size. Other important information includes the split of flow between the minor and major flow path, maximum flow widths in gutters, maximum allowable flow velocity in pipes, the location and direction of any diversions and breakouts, and the extent of property inundation.

The effect of conveyance infrastructure on these flood characteristics varies across the different types of urban flood models.

Some models reflect the performance of conveyance infrastructure explicitly, which requires that the designer input a detailed physical definition of conduits and their hydraulic characteristics. The typical data that must be collected and input to these models include:

- Conduit type;
- cross-sectional dimensions (e.g. pipe diameter, channel width and depth, profile);
- length;
- slope, or sufficient elevation data to allow slope to be calculated using length; and
- hydraulic parameters (e.g. mannings ‘n’, viscosity).

This information must be gathered for each relevant piece of conveyance infrastructure that is part of the network being investigated. In some cases, this data may be readily accessible in an asset database. In other circumstances this data may require collection via ground survey. When data cannot be obtained due to inaccessible structures, assumptions regarding the network geometry may be required.

A schematic representation of the overall conveyance network, including connectivity between inlets, conduits, and junctions is then constructed within the model.

These models can be data intensive, but they also have potential to provide detailed and accurate descriptions of flood behaviours.

Depending on the type of user interface and pre-processor associated with the adopted model software, some of these data requirements may be automatically harvested from other raw input data. For example, a three dimensional surface model may be used to establish roadside gutter profile and slope automatically. Even so, some information will be required such as in this case the plan position of the roadside gutter.

For large urban areas, particularly those that have become densely developed over a long period of time, the task of collecting and collating all the dimensions of all conveyance infrastructure can be a major undertaking. Further complexity and effort arises since each inlet structure has a potential hydrologic sub-catchment that must be defined and input to the model. It is also possible that the size of the sub-catchment may change as flow rate increases. An exception is a rainfall-on-grid model approach where sub-catchment definition may not be required, but even still, substantial effort is required to ensure each inlet structure is capturing a realistic amount of runoff.

In some cases the burden of this infrastructure definition task can be reduced through use of simplified models and assumptions that do not explicitly model the performance of all conveyance infrastructure items. For example, the capacity of underground drainage may be an assumed proportion of the total runoff hydrograph or in some cases totally ignored. This approach can be acceptable if the capacity of the underground system is small relative to the size of floods being investigated. In this case the model construction may instead focus on a more accurate definition of surface-based conveyance infrastructure and overland flow paths.

Other models can provide flood estimates using an even more implicit description of conveyance infrastructure. For example, rating curves and stage hydrographs may be used for selected locations in conjunction with run-off routing hydrologic estimates. In this case less physical data needs to be collected.

Any decision to simplify the description of conveyance infrastructure within a model needs to be made recognising the accuracy requirements of the investigation and the risks associated with any limitations that may be introduced. It is important that the impacts of simplifying models and associated assumptions are fully understood. This is further discussed in Book 9, Chapter 6, Section 3.

6.2.3. Waterway Crossings

Waterway crossings are urban infrastructure for the purpose of allowing access across a natural or man-made waterway. The most commonly encountered waterway crossings comprise of causeways, culverts and bridges that are constructed as part of a vehicular, rail or pedestrian transport system.

Waterway crossings can have considerable hydraulic impact for floods within the range where the crossing structure causes the cross-sectional area of the waterway to be substantially reduced. In these circumstances additional energy is required to pass flow through and/or over the structure causing increased pressure head upstream of the crossing (afflux). Afflux is flow dependant and will change across the range of potential flood discharges. This afflux can cause a significant storage volume to be engaged upstream of embankments which can therefore also heavily influence downstream flood behaviour.

A comprehensive description of hydraulic behaviour at waterway crossings and other hydraulic structures is found in Book 6, Chapter 3.

As well as causing afflux locally around the structure, the hydraulic behaviour associated with waterway crossings can also have an impact on:

- Floodplain storage and hydrograph attenuation;
- tail water levels for upstream drainage;
- cross-catchment diversion of flow; and
- bed scour and local stream morphology.

These impacts are not necessarily confined to those that are in the immediate vicinity of the investigation site or study area and may impact areas upstream or downstream. A comprehensive urban flood investigation should therefore consider the impact of each existing or proposed waterway crossing in the catchment (and adjoining catchment in the case of cross-catchment diversion) and whether they could have an impact on local flood behaviour.

Once the relevant waterway crossings have been identified, the urban modelling task is then to suitably define the crossing structure within the model. This will normally include the physical dimensions and shape of the waterway opening beneath the crossing deck and any obstruction caused by associated railings, embankments, and utility services. Models may also assist with identifying locations where bed shear stress increases are likely and the design of scour protection measures (refer to Book 6, Chapter 3).

Consideration also needs to be given to blockage potential of the overall structure and which blockage scenarios may be required in order to fully describe potential flood behaviour. Book 6, Chapter 6 provides further detail regarding blockage considerations.

As with conveyance infrastructure, some types of urban models may estimate the flood behaviour impacts of the waterway crossing in an implicit manner through use of rating curves and stage hydrographs. The impact of any such simplifications and assumptions on model accuracy needs to be considered when selecting an appropriate model platform for the investigation.

6.2.4. Volume Management Infrastructure

Volume management infrastructure comprises of discrete facilities, primarily for the purpose of controlling peak discharge and volume. They can be located at almost any point within a drainage network and are linked by conveyance infrastructure and/or natural waterways. A comprehensive description of typical volume management infrastructure facilities is found in Book 6, Chapter 4.

The hydrologic and hydraulic impact of these facilities can be significant and will vary according to the design of the facility and size of the flood. For the urban designer, the task associated with this infrastructure is the physical description and schematisation of the facility within the model. This will normally include:

- Storage characteristics and how the volume stored varies with depth; and
- outlet characteristics and how the outlet influence depth and volume of water stored in the facility

The way this model task is completed will depend on the type of model being used, but most commonly involves entering a form of definition table describing storage volume with depth along with details regarding the physical dimensions and elevations of the outlet structure.

Depending on the intended purpose of the urban modelling task, consideration should also be given to antecedent conditions, whether the storage is partly utilised prior to the onset of the storm burst and whether there is potential for blockage of the outlet structure at some point in time and to what extent.

The hydrologic and hydraulic impact of a volume management facility may be distant from its physical location (upstream or downstream). The designer must consider inclusion of all volume management facilities that could potentially impact the investigation site. Also, as proposed storage volumes increase, the critical storm duration and pattern may correspondingly change, necessitating the inclusion of additional rainfall scenarios into the suite of model tests.

6.2.5. Water Quality Treatment Performance

An increasingly common urban modelling task is the assessment of the water quality treatment performance associated with a water treatment facility such as those described in Book 9, Chapter 4.

The facilities that perform this function are often co-located or are an integral part of a volume management infrastructure facility. Where this is indeed the case then similar model inputs are required such as the basin storage and outlet characteristics. However, a different model platform may be necessary since the treatment process targets smaller storms and occurs over longer time periods. For example, event based hydrologic models may not be a suitable basis for these assessments. Instead a continuous simulation-based model would be more suitable.

In addition, further information is required to define the treatment characteristics of the facility. These are mostly based on empirical relationships that simply associate the performance of the facility with its size or alternatively retention curves that relate inflow and outflow concentrations of pollutants. The pollutants of most interest are gross pollutants, nutrients (Total Nitrogen and Total Phosphorus), and Total Suspended Solids.

6.2.6. Data Collection and Collation

A well organised data collection and collation process is essential in the modelling process. It not only ensures that the modelling is fit for purpose, but it documents the sources of data and how the data was interpreted and used in the model. Models often evolve as improvements are made or processes are changed to better represent different components. This task is much simpler if a good data management process has been used.

It is important that the data management system properly documents the source of the data, the format, and the date of acquisition. Book 1, Chapter 4 provides comprehensive advice on the use of data. A key challenge in urban catchments is that many urban drainage components cannot be put directly into a model but need to be schematised. Examples include converting a basin drawing into a stage storage table or representing a complex pit system. It is important that the data management system properly documents this process so the interpretation and schematisation is properly documented and can be reviewed or refined later. While data can be classified in many ways, there are three broad types of data:

- Model inputs such as rainfall and temporal patterns that change between events;
- model components such as pipes, storages, terrain information and land use data; and
- observed data such as observed peak flood levels and flows

The digital age has changed many aspects of data collection with data often being easier to find but often the original data sources are unclear with merged data sets representing the largest part of this problem. This same problem exists in the model development process where many data sets are interpreted and merged. While most urban catchments are ungauged recent observed flood data can often be found on social media and older historical flood information can be found in scanned historical records and newspapers.

6.3. Model Selection

There is a wide range of conceptual modelling approaches, software platforms and systems available to the urban designer. Each platform has different capabilities and strengths. It is not the role of this Guideline to recommend specific conceptual modelling approaches, software packages or prescribed flood estimation methods. However, the guidance contained in this chapter does seek to classify the available options into categories and highlight the current strengths and weaknesses of each to support a decision on the adoption of an appropriate platform or estimation procedure for the task at hand. The authors are mindful that the science and practice of urban stormwater management will continue to evolve, and new models and data will become available. The guidance in this chapter should not be perceived to be excluding new and innovative approaches.

6.3.1. Overall Trends in Urban Modelling

The last 30 years has seen fundamental changes in the way urban stormwater assessment and design tasks are undertaken. It is reasonable to assume that similar change will occur over the next 30 years. Recognising that the decision to adopt a specific urban catchment model platform can have significant implications for personal research and training, this section provides introductory level discussion about these trends. It is expected this will support more informed choices related to adoption of a model platform, either for a specific investigation project or for a longer-term strategic assessment program.

6.3.1.1. Computing Power

In response to the overall computing requirements of society, urban modelling designers now have access to faster computers with enormous numerical computation capability. This has arisen through improvements to computer processors (CPUs) including 64-bit computing and multi-core processing. More recently the use of Graphics Processing Units (GPUs) has led to further substantial processing improvements. New opportunities are also arising with the advent of high-performance computing services, including on the cloud. The transition from hand calculations to widespread availability of computing power to assist in designs is a major change in stormwater management practice since ARR 1987 (Book 9, Chapter 3, Section 3).

As these computing advances have occurred, urban modelling software platforms have been adapted to harness some of the available computational speed increases. This permits the modelling designer to consider:

- Increasing the physical size of the model domain. For example, model a larger urban catchment;
- increasing the spatial and temporal resolution of the model to allow for finer grained numerical calculations that account for location and connectivity of different land uses;
- longer time-series of rainfall;
- a greater number of catchment scenarios;

- tighter integration of hydrologic and hydraulic computation;
- more model iterations to support improved calibration and sensitivity analysis; and
- less conceptualisation and closer alignment to complex physical processes.

It can be expected that computational capabilities will continue to increase into the future and that urban modelling software platforms will continue to be refined and improved to harness more of the available capacity.

Currently, computing power is such that it is reasonable to expect that most urban hydrologic model simulations, even relatively complex ones, can be undertaken within seconds or minutes. It can therefore be assumed that pure hydrologic investigations are already unconstrained by computing power regardless of the choice of model platform.

Computing power is still somewhat of a constraint for hydraulic simulations. Some of the more complex finer resolution or larger domain hydraulic model simulations can take hours or days per simulation. This may constrain the design of an urban hydraulic modelling investigation and also means that due care must be taken when selecting a hydraulic model platform. A hydraulic platform and method should be chosen that has computational efficiency to match the problem at hand. Models with very long run times should be carefully managed as they usually preclude comprehensive testing, checking or calibration.

The future will permit very large multi-catchment spatial domains to be modelled at the finest level of temporal and spatial resolution necessary, with sufficient speed to allow simultaneous and exhaustive exploration of hydrologic and hydraulic scenarios.

This trend may outpace our ability to improve the underlying science and gather sufficient quality input data, and to respond with more informed design and management solutions. Consideration will also need to be given to whether the ultimate outcomes of investigations are improved by aggressively pursuing the full capabilities of available computing power.

In other words, at some point in the future, further improvements to computing power may cease to provide any material value to urban modelling designers. Substantial further research and data collection, for a range of urban catchment scales, is necessary to ensure theory is able to keep pace with computing power.

6.3.1.2. Alignment to Physical Hydrologic and Hydraulic Processes

The underlying methods that are applied using computer-based models have experienced a trend away from conceptual and simplified deterministic techniques to methods that more closely align with the actual physical processes that are occurring.

Some examples of this trend are:

- A move away from isolated storm bursts with a single pattern, towards consideration of pre-burst rainfall and more complete storm bursts including an ensemble of equally likely but different temporal patterns. This leads to more robust design and resolves some of the issues that arise when trying to maintain probability neutrality between rainfall and flood (refer Book 3 and Book 5). The future will see this trend continue with designs becoming increasingly based on complete storms and continuous recorded or synthetic rainfall sequences.
- The use of direct rainfall, also referred to as 'rainfall-on-grid' approaches which attempts to explicitly resolve the accumulation of runoff progressively down the catchment, removing

the need to pre-identify flow paths and sub-catchments. This is a useful way to ensure flow paths are not inadvertently omitted from an investigation. With further research and software development this approach may in time also eliminate the need for hydrologic models to undertake surface routing. At this stage however, there is inadequate evidence that a direct rainfall approach should be relied upon for this purpose with many parameters being scale and approach dependent (refer Book 5).

- The hydraulic models applied in practice have increasingly changed from one-dimensional to two-dimensional representations of the floodplain surface. This allows a more realistic definition of potential flow paths which in turn improves the representation of flood behaviour (refer Book 5 and Book 7).

With continuation of this trend it can be anticipated that model platforms will eventually converge on more accurate representations of rainfall runoff and flood processes, requiring different model inputs, parameters, and application techniques. Again, this will only occur with adequate research and software development effort and data collection for a range of urban catchment scales.

6.3.1.3. Statistical Approaches

There has been increasing awareness and understanding of the need to consider the joint probability of model assumptions and physical processes (Kuczera et al., 2006). This has given rise to techniques such as Monte-Carlo sampling and ensembles of rainfall patterns to reduce potential probability distortions and gain better appreciation of model uncertainty (Book 3). For simple urban models or where the design objectives have limited sensitivity to model results these approaches may not be warranted.

These approaches should be considered where better appreciation of natural variability and uncertainty is required. This may include sensitive urban areas, a major waterway crossing, large flood mitigation proposal or hydrologic design of regional scale water quality infrastructure.

Machine learning algorithms are also being used for the prediction of stream flow using statistical information drawn from historic rainfall and stream gauge data, providing an alternative approach to hydrologic modelling.

6.3.1.4. Accumulation of Longer Periods of Recorded Data

With the passage of time, longer periods of recorded data have become available to allow refinements of design rainfall, losses, and more informed model calibration (Book 2, Chapter 3, Section 4). Over time this will allow a better understanding of model performance and uncertainty, particularly within those catchments where data has been recorded. There will be diminishing situations where models are left uncalibrated for the want of historic data.

6.3.1.5. The Internet and Spatial Information Systems

Since the 1987 version of ARR, the internet has emerged to become a ubiquitous part of life. The internet provides urban modelling designers a new potentially more effective method for:

- Accessing and disseminating research, including international practice;
- gathering model input data;
- processing of simulations (using cloud processing technology); and

- storing and communicating information arising from model investigations.

Furthermore, modelling software platforms that have traditionally been tied to a single computer, are now able to be offered as internet-based services. Into the future other new applications will be found for the internet that cannot be fully anticipated at this time but will likely support further improvements in the application of urban models.

In parallel to the internet, an associated trend that has also emerged is a deeper interest and reliance on Geographical Information Systems (GIS). These systems are used for the storage, handling and display of physical catchment data, catchment parameters and infrastructure data.

Spatial information systems have become an important support technology for the application of urban models, with most platforms leveraging these tools for pre-processing and post-processing of data, storage of data, data display, data enhancement and the preparation of information products for stakeholders.

6.3.1.6. Information Needs of the End User

The information needs of the end user have become more complex. A greater number of aspects are of interest. For example, the extent, depth, and level of floodwaters are now typically supplemented by velocity, combinations of velocity and depth (hazard), volume and timing. Enhanced datasets are also now prepared such as risk and planning controls. These results are often required at many additional locations distributed across urban catchments rather than at selected locations at the bottom of catchments.

Urban modelling designers should consider how the model software platforms they use can be used to accommodate these growing information needs.

6.3.2. Types of Urban Models

Notwithstanding the potential future trends in urban modelling described above, today’s industry designers already have a greater array of model platforms and estimation options than available in the past. However, each option differs in the quality of spatial representation they are capable of achieving, as well as the capability with which they can represent different physical flood processes. Accordingly, some models or methods may or may not be suitable for a specific urban modelling task.

Some of the more common types of models and methods are listed in Table 9.6.1. For each type, a generic classification of its capability is also provided. This is a snapshot in time of the capability of these models and will change with time. This classification is based on the examples in Table 9.6.2. A subsequent section describes the performance of these models at different spatial scales.

Table 9.6.1. Common Types of Urban Models

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Hydrology	Rational Method	Limited	None	None	Peak flow only – scalar quantity, single lumped catchment, requires 'Time of Concentration' assumption, only suitable for small catchments. It has best capabilities where there is no storage present.	RATHGL, PCdrain
Hydrology	Time Area Method, Extended Rational Method	Moderate	None	None	Suitable for small catchments only. Can be extended as a collection of linked sub-catchments.	ILSAX, DRAINS
Hydrology	Runoff Routing	Strong	Moderate	Limited	Full event hydrograph, empirically derived lag parameters, non-linear routing capabilities. Structure hydraulics can be moderately capable for discrete structures but not for continuous conveyance networks.	RORB, RAFTS, WBNM, URBS, HEC-HMS
Hydrology	Continuous Simulation	Strong	Moderate	Limited	Continuous multi-year runoff sequence, comprehensive infiltration loss models. Limited capability for rare to very rare floods unless utilised with replicates of conditioned synthetic continuous rainfall (such as DRIP)	XP-RAFTS, MUSIC, PURRS, Systems Framework
Hydrology and Hydraulics	Hydrology coupled to 1D hydraulic model	Moderate	Moderate	Strong	Not always emulating full capability of the underlying hydrologic model	DRAINS, PCdrain, XP-SWMM

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Hydrology and Hydraulics	Direct Rainfall ('rainfall-on-grid')	Limited	Moderate	Strong	Does not require pre-defined flow paths. Sensitive to topographic data pre-processing and surface roughness assumptions. Not suitable for 'greenfield' subdivision drainage design.	TUFLOW, MIKE21, SOBEK, ANUGA
Hydrology and Hydraulics	Runoff routing coupled to two-dimensional hydraulic model	Moderate	Strong	Strong	Requires pre-defined understanding of flow paths in order to establish initial model. Requires input and output procedure between two model software packages.	RAFTS with MIKE21, WBNM with TUFLOW, XP STORM with TUFLOW, DRAINS with TUFLOW
Hydraulics	One-dimensional hydraulic model	None	Moderate	Strong	Simple channel or pipe behaviour only. Limited where complex flood storages exist.	HEC-RAS, MIKE11, SOBEK
Hydraulics	Two-dimensional hydraulic model	None	Strong	Strong	Complex flow behaviour including breakout and diversion. Flow transitions and hydraulic jumps. Principally surface flow.	TUFLOW, SOBEK, ANUGA, MIKE21, HEC-RAS 2D, RMA, RiverFlow2D
Hydraulics	Pipe network models	None	Moderate	Strong	Specialist models for underground drainage networks, storage routing performance best where flow is contained within the minor system.	SWMM, XP-STORM, DRAINS, PC drain, MIKE URBAN

Focus	Urban Model Type	Estimation Capabilities (also refer Table 9.6.2)				Example Model Platforms (where relevant)
		Runoff Generation and Surface Routing	Channel and Storage Routing	Structure Hydraulics	Other specific capabilities or limitations	
Water Quality	Water quality model	Moderate	Limited	Limited	Additional capabilities related to pollutant generation and removal. Hydraulic capabilities can be extended by coupling to 1D hydraulic model. Runoff generation less suited to event based flood estimates.	MUSIC, EPA-SWMM

Table 9.6.2. Generic Classification of Model Estimation Capability

Flood process	Limited Capability	Moderate Capability	Moderate Capability
Runoff generation and surface routing	Average intensity or burst Cursory treatment of infiltration losses Surface characteristics not fully represented	More complete storm Infiltration losses Surface characteristics partially represented	Full storm or rainfall sequence Infiltration losses Spatial distribution of rainfall Surface characteristics well represented (including surface wave speed)
Channel and storage routing	Channel characteristics not represented No explicit calculation of flood storage and its attenuation effects	Channel characteristics partially represented Storage behaviour partially represented including attenuation effects and spatial influences	Channel characteristics and flood wave speed well represented Storage behaviour well described including complex hydraulic behaviour and attenuation effects

Flood process	Limited Capability	Moderate Capability	Moderate Capability
Structure hydraulics	Basic hydraulic structures only Rating tables Manning's formula for open channels.	Small range of hydraulic structures Basic topographic representation	Wide range of hydraulic structures Resolves shallow water equations (1D or 2D or both)

When selecting a particular model or technique, the designer should in the first instance look to match the estimation capabilities of the model, whether they be 'limited', 'moderate' or 'strong', with the nature of urban modelling problem that is being investigated.

For example, if channel routing and structure hydraulics are not aspects of the problem that need to be investigated, then the model selected need not have any capabilities in these areas. Equally, if it is expected that a particular problem will require significant capabilities in (for example,) runoff generation, then a model with 'strong' capabilities in this area should be considered.

Where the estimation capabilities are identified in Table 9.6.1 as 'limited', significant caution must be adopted. As a minimum they should be applied by, or under the direct guidance of, a designer who fully understands the limitations of these approaches. The tolerance for error in the results should be considered and if greater accuracy is required then an alternative more capable model platform applied.

As always, the level of experience of the designer is a significant factor. Someone with significant experience and familiarity with a specific model may be able to extend its capabilities to a level that achieves an acceptable level of estimation accuracy that is beyond its normal capabilities if deployed by an average or less advanced user.

6.3.3. Model Scale

Urban models are constructed at different spatial scales depending on the size of the overall catchment to be analysed and the nature of the performance objectives being sought. Typically the smallest catchment a designer will consider is that of a single small parcel of land with a single dwelling. The type of model assessments that are normally undertaken at this scale include model calculations to assist with design of internal drainage systems and small volume management facilities (e.g. rainwater tanks and OSD).

At the other end of the spectrum of potential scale, an urban model may be constructed to represent all the stormwater catchments spanning an entire suburb or even a small city. These larger models are often used for the purpose of regional flood mapping, establishing flood levels for development purposes or the design of large-scale stormwater and road crossing infrastructure.

When evaluating which type of model to adopt for a particular urban modelling project, the spatial scale of interest is an important factor to consider since some particular models may not be capable of competently representing all the complexity that is encountered at the scale of interest.

Consider four example spatial scales with each physical footprint increasing by an approximate order of magnitude as shown in Table 9.6.3 below.

Table 9.6.3. Typical Urban Model Scales

Lot	Site	Neighbourhood	Precinct
A small parcel of land with 1 or 2 buildings.	A large parcel of land with multiple buildings. Sometimes a small number of 'lots' combined.	Many parcels of land each with at least one building. Many 'lots' and potentially some multi-building complexes.	Hundreds of parcels of land each with at least one building. A large number of 'lots' and multi-building complexes combined. Several neighbourhoods.
e.g. single detached dwelling or duplex up to 1,000m^2 in area	e.g. large townhouse complex covering an area up to 1 hectare	e.g. a residential subdivision stage or a neighbourhood covering an area up to 10 hectares	e.g. a small suburb covering an area of 100 hectares

As a model's spatial scale increases from 'lot' through to 'precinct' the more likely that the catchment being modelled will contain a greater range of features of relevance to stormwater behaviour such as:

- Public roads acting as overland flowpaths
- A larger variety of different land uses and associated connectivity
- Large capacity conveyance infrastructure
- Large basins and volume management infrastructure
- Urban waterways
- Urban waterway crossings

In conjunction with this increase in the number of stormwater features, it follows that the potential number of rainfall runoff processes encountered in a larger scale model will also increase. In this context flood generation processes include damaging floods as well as much smaller floods that are relevant to yield and water quality assessment.

Table 9.6.4 below provides a list of the flood generation processes encountered at each of the four spatial scales described above. This listing is non-exhaustive and only provided to demonstrate that there is a larger number of potential flood generation processes that can be expected to occur as spatial scale increases from 'lot' scale to 'precinct' scale (growing from approximately 8 to 32 in the example listing provided in Table 9.6.4).

Some degree of simplification of these flood generation processes normally occurs when preparing an urban flood model. The flood generation processes listed in Table 9.6.4 have different levels of importance and influence when trying to decide whether any simplifications are possible. Each process has been indicated in Table 9.6.4 by one of two different symbols as follows:

A very important flood generation process. A model constructed at this scale should have the capability to competently address this flood process.	1

A flood generation process that is less important. This process may be omitted or simplified if accuracy of model estimates is not critical	2

Further discussion regarding model simplification is included in Book 9, Chapter 6, Section 3.

Table 9.6.4. Example Flood Generation Processes at Different Model Spatial Scales

Example Flood Generation Processes	Lot	Site	Neighbourhood	Precinct
Overland flow routing across surface of lot	2	2	2	2
Conveyance capacity of roof gutters and downpipes	2	2	2	2
Routing through internal underground drainage	2	2	2	2
Runoff generation from impervious surfaces within lot (e.g. roof)	1	1	2	2
Runoff generation from pervious surfaces within lot (e.g. garden)	1	1	2	2
Conveyance capacity of internal underground drainage	1	1	2	2
Routing through temporary and/or permanent storage connected to dwelling (source control)	1	1	2	2
Storage outlet behaviour including use of stored water for internal and external private demand (source control)	1	1	2	2
Overland flow routing between multiple lots		2	2	2
Routing through open surface drains and driveways		2	2	2
Routing through inter-allotment drainage		2	2	2
Runoff generation from impervious surfaces within common areas (e.g. common driveway)		1	2	2
Runoff generation from pervious surfaces within common areas (e.g. landscape areas)		1	2	2
Conveyance capacity of open surface drains and driveways		1	2	2
Conveyance capacity of inter-allotment drainage		1	2	2
Capacity of inlets to the internal underground system and potential bypass		1	2	2
Routing through temporary and/or permanent storage within common area (source control)		1	2	2
Storage outlet behaviour including use of stored water for external demand within common areas (source control)		1	2	2

Example Flood Generation Processes	Lot	Site	Neighbourhood	Precinct
Overland flow routing across the sub-catchment surface			2	2
Routing through roadside gutters and table drains			2	2
Routing through underground drainage and trunk drainage			1	2
Routing through major overland flow paths			1	2
Conveyance capacity of roadside gutters and table drains			1	2
Runoff generation from impervious surfaces (neighbourhood scale)			1	2
Runoff generation from pervious surfaces (neighbourhood scale)			1	2
Capacity of inlets to the road drainage system and potential bypass			1	2
Capacity of inlets to the trunk underground drainage system and potential bypass			1	2
Conveyance capacity of underground drainage and trunk drainage			1	2
Conveyance capacity of major overland flow paths			1	2
Routing through temporary and/or permanent storage within public areas (neighbourhood control)			1	2
Storage outlet behaviour including use of stored water for external demand within public areas (neighbourhood control)			1	2
Runoff generation from impervious surfaces (precinct scale)				1
Runoff generation from pervious surfaces (precinct scale)				1
Routing through large open channels and urban waterways				1
Conveyance capacity of large open channels and urban waterways				1
Performance of culverts and bridges including impact of blockage and diversion				1
Routing through temporary and/or permanent storage within public areas (regional control)				1
Storage outlet behaviour including use of stored water for external demand within public areas (regional control)				1

2. Opportunity for model simplification (refer Book 9, Chapter 6, Section 3)

Most model platforms have some limitations on which processes they can represent. A decision will be required at the commencement of model preparation as to whether the selected model and the available data are capable of achieving the required level of accuracy and reliability.

As a result of the expected increase in the number of flood generation processes with scale, if a catchment investigation requires investigation across a large spatial scale, then the designer can expect that a model or method with 'strong' estimation capabilities across multiple flood process areas will be necessary (refer Table 9.6.1 and Table 9.6.2).

For example, the Rational Method, with 'limited' runoff generation and surface routing capabilities, is not likely to be suitable for a 'precinct' scale estimate of peak flow as it cannot adequately simulate the array of flood processes that are encountered, even in the simplest of catchments. However, it may be suitable at a 'lot' scale in circumstances where storage routing is not critical.

If volume management infrastructure forms part of a solution, or if an understanding of potential impacts on downstream flooding are required, then a 'strong' hydrologic estimation method such as a runoff-routing model should be used. For most urban modelling at this point in time, a runoff-routing model coupled to a two-dimensional hydraulic model or pipe network model will provide the strongest estimation capabilities across a wide range of model scales.

The resolution of model inputs and boundary conditions also needs to be considered. There is little value in developing a high-resolution model with coarse lumped inflows or considering the performance of a complex system using a single temporal pattern.

6.3.4. Flood Magnitude

The capability of each type of model also varies with magnitude of the flood being considered. For the smallest of floods, including frequent storms and runoff events, the model's capabilities should include consideration of infiltration losses including for some applications the recovery of soil moisture profiles during inter-event periods and baseflow. The importance of this capability may change depending on the level of impervious cover within the catchment, becoming decreasingly important as impervious cover increases.

These capabilities are principally the domain of runoff-routing and continuous simulation models. Other processes that effect total runoff volume such as harvesting and use of rainwater may also be important considerations for smaller flood magnitudes. Figure 9.6.1 indicates the likely range of effectiveness for the different types of hydrologic models against flood magnitude on x-scale and model scale on y–scale.

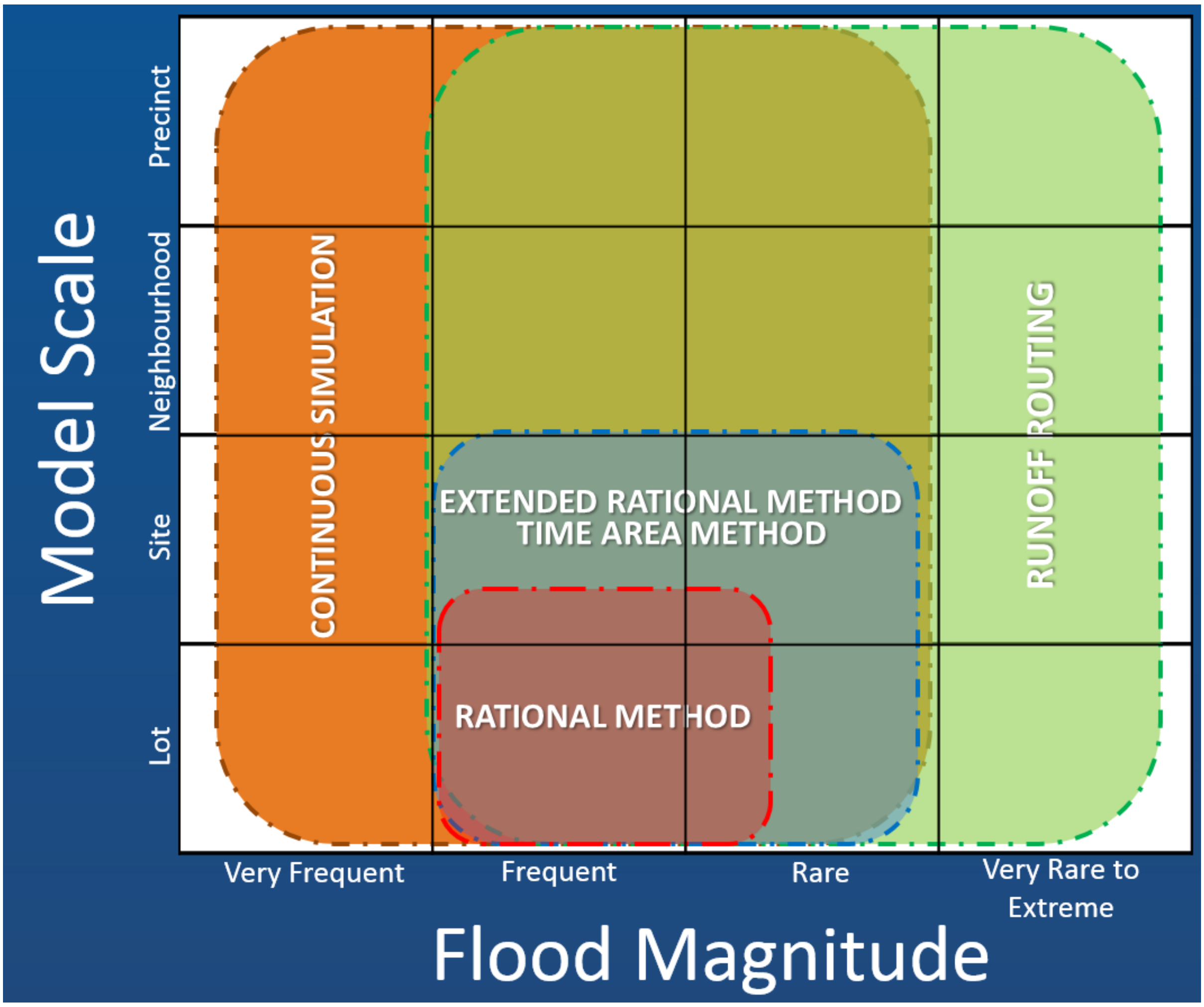

Figure 9.6.1. Types of Urban Hydrologic Models and their Likely Application Range

As the magnitude of flooding that is of interest increases, different hydrologic model requirements emerge since the importance of antecedent soil moisture and rainfall diminish. Typically, runoff-routing models applied using discrete rainfall bursts or more complete storms would be used.

For the companion hydraulic calculations during small floods, and where flooding is confined to the pipe network or a simple channel, a pipe network model and/or 1D channel hydraulic model will normally be adequate. Even some hydrologic model packages have the capability to undertake basic hydraulic calculations.

For hydraulic calculations associated with large floods that exceed the normal capacity of a channel, or where substantial overland flows develop, a 2D hydraulic model may have more utility since the likelihood of complex flow patterns increases.

Further detailed information regarding hydraulic models is included in Book 6, Chapter 4.

6.3.5. Choosing a Model

A stepwise process is suggested below to assist with identifying the types of models that may be suitable for a specific urban modelling problem. Consideration of each step in the flowchart shown in Figure 9.6.2 will help to progressively reduce the number of candidate model types that may apply.

Figure 9.6.2. Stepwise Flowchart for Selecting an Urban Stormwater Model

Where there are multiple options arising from this process, the simplest model, capable of the necessary calculations should be favoured. Other model selection criteria include, availability of sufficient input data and parameter research, output data capabilities, availability of other required functionality (e.g. water quality calculation), cost, and designer familiarity with the model. A hydraulic model involves a more explicit representation of flow routing and how storage is represented in the catchment. Generally, a hydraulic model will be required where there is a need to understand both flow and flood levels. one-dimensional pipe and channel models only provide this information at key locations but are well suited to 'greenfields' subdivision design, while a two-dimensional model provides a detailed spatial representation of surface stormwater processes and may be more suited to brownfields investigations.

6.3.6. Model Simplification

In conjunction with selecting a type of model that has the necessary estimation capabilities and is well suited to the model scale of interest and associated smaller scale influences, consideration must also be given to the degree of model simplification that might be appropriate.

When modelling at small spatial scales it is simpler to closely represent each flood process and its associated physical features and drainage connections explicitly. As spatial scale increases it is sometimes possible to adopt some model simplifications to manage data requirements and the general complexity of the modelling task. For example, when building a ‘precinct’ scale model it may be possible to omit or simplify ‘lot’ scale processes. However, models should not be simplified unless that consequences of spatial averaging, deterministic assumptions and judgements is well understood. Where simplification is undertaken, efforts should be made to fully understand the impacts of simplification and limits on validity of the model outputs. For example, by comparison of results against a more detailed sub-model or results generated by an alternative model.

Experience and careful judgement are required when choosing to omit or simplify those processes that are suggested as being less important. In general, the omission or simplification of such a process should only occur when the investigation does not demand highly reliable estimates, for example, for preliminary sizing of structures or where flood risks are low. Table 9.6.4 indicates those flood generation processes that may be less important and therefore could be considered as an opportunity for simplification at each different spatial scale.

6.3.7. Model Resolution

Closely related to consideration of model simplification is the interrelated consideration of model resolution. Resolution can in this context have multiple aspects.

Firstly there is spatial resolution of the model. For a hydrologic model this will relate to the minimum size of sub-catchments. For a hydraulic model this will relate to the density of sampling of the ground surface.

The adopted spatial resolution of a model will govern the density of reporting locations i.e. where model results are output by the model software. It may also influence model accuracy. Through experience a designer will develop an understanding of the optimum model spatial resolution for each type of model and to what degree spatial simplification can be tolerated.

Then there is the temporal resolution of the model and the ability to extract output time series that are fit for purpose. For example, the temporal resolution necessary for regional water supply planning may be lower than required for calculation of stormwater harvest yield from a small catchment. In this case a degree of temporal simplification to daily or monthly data may be acceptable for a regional water supply planning task.

Again, through experience a designer will develop an understanding of the optimum model temporal resolution and to what degree temporal simplification can be tolerated.

6.4. Application to Urban Modelling

Stormwater management is subject to ongoing evolution and change. There has been substantial change to the practice and science of stormwater management since 1987 as discussed in Book 9, Chapter 3. This version of ARR combines 30 years of additional data with evolving science and professional capability to accommodate changes in professional and community aspirations. This process has provided a range of new methods, data and resources that can assist the designer to address the local challenges of managing stormwater runoff in urban areas.

Drainage networks (also discussed in Book 9, Chapter 5) are now considered to be part of more comprehensive stormwater management approaches (refer Book 9, Chapter 3) that respond to multiple water cycle objectives including protecting waterways, mitigating flood risks, provision of water resources, managing the quality of stormwater runoff and enhancing the amenity of urban areas. These approaches respond to a need to manage urban water balances (discussed in Book 9, Chapter 2) and to also incorporate a range of storage measures (refer Book 9, Chapter 4) that aim to manage flooding, stormwater quality and provide additional water resources.

This section provides a framework for application of modelling approaches to urban stormwater catchments. The framework provides guidance for key segments of catchments from the behaviour of land uses within sub-catchments that flow to inlet structures, through urban stormwater networks, and into the receiving waterway.

A range of approaches are now available to determine the configuration of measures in a linked stormwater management system than may include a conveyance network, volume management strategies and non-structural measures. These methods can range from simple procedures to detailed computer modelling. The application of new rainfall data and methods to modelling approaches is discussed with reference to the different approaches to the design of stormwater management measures and systems.

6.4.1. Urban Modelling Frameworks

An increasing range of modelling frameworks and approaches are available to urban designers (refer Figure 9.6.1). The urban stormwater design process, as outlined in Book 9, Chapter 5, Section 3 (refer Figure 9.5.3), should be modified to respond to the characteristics of a particular project. Selection of a modelling framework will depend on the purpose of the analysis, scale and complexity of the project, availability of data and the consequences of failure, and includes:

- Hydrological models that translate rainfall into stormwater runoff and evaluate behaviour of storages;
- hydraulic models that evaluate or design the transfer of stormwater flows through networks of infrastructure and across land surfaces;
- hydrology models that include simple pipe hydraulics or one-dimensional hydraulic models;
- linked hydrology and hydraulic models that include detailed two-dimensional surface flows with hydrodynamic conveyance networks;
- rainfall-on-grid models;
- continuous simulation of rainfall runoff and physical processes to evaluate behaviour of integrated solutions and account for antecedent conditions, water quality and associated performance issues; and
- approximate empirical relationships or peak runoff assumptions used to design and evaluate components of urban catchments.

We should be mindful that all models are an approximation of reality that can be used to enhance our understanding about the likely stormwater behaviours for particular urban scenarios. The different hydrological and hydraulic models can be classified by their outputs of peak flowrates, hydrographs, flood depths or continuous sequences of stormwater runoff. These models can also be distinguished by the methods used to route rainfall runoff towards inlet structures in urban conveyance networks or stormwater volume management measures. Models can also be described by different spatial detail such as lumped, semi-distributed or distributed inputs (Figure 4.2.5, Book 4, Chapter 2, Section 6). Lumped catchment models approximate the behaviour of the catchment using single average inputs and assumptions. Semi-distributed models employ a range of sub-catchments with different attributes and assumptions. In contrast, spatially explicit details are included in distributed models – this detail may include the range of different land uses and properties in an urban model or a grid of equal size and shape used throughout the model. An emerging type of distributed hydrology and hydraulic model is the direct rainfall or rainfall-on-grid methods (refer Book 6, Chapter 4, Section 7).

Empirical relationships can be utilised to determine peak flows from small catchments and are applied to the design of roof gutters, downpipes, and infrastructure to manage stormwater runoff from properties in accordance with standards such as AS/NZS 3500.3. These approximate methods include nominal “deemed to comply” infrastructure specifications or generally require information about catchment area and slope, and utilise assumed runoff coefficients, time of concentration and design rainfall intensity in a lumped catchment design process.

The probabilistic or the urban Rational Method is a more detailed approximate method that is utilised to generate peak flowrates for use in the design of pipe networks within small properties and for small sub-catchments. This framework of analysis differs from simple empirical relationships by including equivalent or effective impervious areas, accumulation of flow rates and the areas of different land uses. The method uses rainfall intensity derived from Intensity Frequency Duration (IFD) data, assumed runoff coefficients and time of concentration to derive stormwater peak flows.

The design approach associated with urban Rational Method is often based on lumped sub-catchment inputs to inlet structures which require the resolution of partial area effects on the timing of cumulative peak discharges throughout a conveyance network. A lumped sub-catchment process combines all land uses, including the area of pervious and impervious surfaces (full area), with an estimated time of concentration to derive peak flows at the outlet of a sub-catchment which is the inlet to a conveyance network. A partial area effect is, for example, where the runoff from impervious surfaces (partial area) arrives at the outlet before runoff from pervious surfaces reach the outlet at less than the full area travel time. These methods may be used to analyse the capacity of individual pipes or peak flows from small catchments but cannot simulate actual flow behaviour throughout conveyance networks and urban stormwater management systems (Pilgrim, 1987).

The simple nature of the urban Rational Method cannot account for the complexity of contemporary urban catchments and modern stormwater management approaches, the temporal and spatial variability of storm events, and variations in antecedent or between storm event processes. Approximate methods, such as Rational Method, should only be applied within a catchment where more detailed analysis of rainfall runoff observations have defined the parameters (for example, runoff coefficient and time of concentration) for use in the method (Phillips et al., 2014; Coombes et al., 2015a). However, Goyen (2000) established that derivation of runoff parameters at the regional scale or bottom of a catchment may not necessarily describe local processes in sub-catchments. Local information is also needed to determine urban runoff parameters.

Runoff or hydrograph routing methods are commonly associated with computer models that include internal processes that incorporate different land uses with separate pervious and impervious surfaces. The process includes depression storages and losses with lag times to generate separate hydrographs of runoff for each land surface. These runoff routing methods typically employ event based rainfall inputs (Book 4, Chapter 3, Section 2) of selected Annual Exceedance Probability (AEP) and duration of peak burst rainfall (refer to Book 9, Chapter 6, Section 4). An objective of this process is to achieve probability neutrality between rainfall inputs and generated runoff for urban catchments.

These runoff routing methods may utilise single or multiple design storms and associated temporal patterns to determine regimes of excess rainfall that is then routed through hydraulic models that range from simple pipe hydraulics to full two-dimensional hydrodynamic processes. A key limitation of event based modelling approaches is the need for assumptions about joint probability of antecedent conditions (such as soil moisture and available storage in volume management solutions) and the characteristics of storm events (Kuczera et al., 2006). In addition, event based methods have traditionally only simulated runoff from burst rainfall and have not considered that runoff is also generated by pre-burst and post-burst rainfall (refer to Book 9, Chapter 6, Section 4). The magnitude of rainfall runoff in urban catchment may be under-estimated by event based processes unless pre-burst rainfall is also counted in rainfall event based models.

The limitations of rainfall event based models, and dramatic increases in the capacity and utilisation of computers has fostered the use of continuous simulation (Book 4, Chapter 3, Section 3 and Book 9, Chapter 3) models that can account for continuous physical, conceptual and statistical processes in urban catchments. These methods have traditionally utilised real or synthetically generated rainfall sequences to understand the yield from water supply catchments and the behaviour of water and wastewater distribution networks. These methods are also used to estimate the behaviour of stormwater quality solutions in urban catchments (Fletcher et al., 2001). However, continuous simulation can also be employed to account for the interactions between climate processes, human interventions or behaviours and stormwater runoff from urban catchments (Coombes and Barry, 2015). Pluviograph rainfall records with intervals of less than an hour (often 6 minute intervals) are used in continuous simulation of rainfall runoff from urban catchments.

The continuous simulation method involves simulation of a rainfall runoff model over a time period of sufficient length to account for all of the important interactions between rainfall and catchment processes to produce an urban flood frequency analysis. Sufficient lengths of observed rainfall are usually not available to provide adequate information about rare runoff events and synthetic rainfall sequences are often required for continuous simulation models (Book 4, Chapter 3, Section 3; Book 2, Chapter 7,). Use of continuous simulation with synthetic rainfall inputs may require calibration of the rainfall model and the continuous runoff routing model (Book 4, Chapter 3, Section 3). However, all models require calibration and verification.

An alternative use of continuous simulation is to derive the probability distribution of initial conditions prior to storm events such soil moisture storage, and available storage in rainwater tanks and bioretention facilities (Coombes and Barry, 2008a; Hardy et al., 2004). These probability distributions of initial conditions are then utilised in event based runoff routing models to determine runoff from urban catchments. Note that these types of probabilistic inputs are associated with complete storm events and will need to be applied in event based models using complete storm events or combinations of pre-burst and burst rainfall.

Direct rainfall or rainfall-on-grid models combine hydrological and hydraulic processes to generate rainfall runoff and hydraulic routing in a single model. Rainfall is applied to each grid in a two-dimensional hydraulic model to generate overland flows and discharges in conduits (Book 6, Chapter 4, Section 7). This method can provide more realistic representation of catchment storages and surface runoff processes including cross catchment flows. A fine grid of good quality topographic, losses and roughness data is required, and topography information will need to be edited to include key infrastructure such as street gutters, hydraulic structure, conveyance networks and road crowns (Hall, 2015). Rainfall-on-grid models should be calibrated to local historical spatial flood levels or flow data. Use of regional rainfall runoff parameters is not suitable for direct rainfall methods that are driven by local processes.

There may also be a need to vary roughness parameters (such as Manning's n) with flow depth (for example, Zahidi et al. (2017); Khrapov et al. (2015); Muglera et al. (2011)) and carefully assign loss parameters in each grid (Babister and Barton, 2012). The results at local and sub-catchment scales may be unexpected as all flow paths are identified. The method is subject to a range of potential challenges including mathematical instabilities, unrealistic flows and large errors created by losses, variable roughness, long runtimes and shallow flow depths. These powerful direct rain methods are subject to ongoing research and model results should be interpreted with caution. It is imperative that designers check that catchment response with an alternative model and volume of runoff is consistent with loss model used (refer to Book 9, Chapter 6, Section 4). If a rainfall excess model is used this represents the volume of runoff that appears at the catchment outlet not rainfall applied to the model so depression storage needs to factored into losses.

6.4.2. Choice of Rainfall

Most hydrology and hydraulic models require rainfall inputs to estimate stormwater runoff and associated flood responses. The investigations underpinning the this guideline incorporated 30 years of additional data and science (Book 2, Chapter 1) to develop improved design rainfall frameworks. There was also a need to incorporate climate change processes into design rainfall frameworks (Book 2, Chapter 2, Section 4). Design rainfalls are simpler and different to real or observed rainfall. More advanced design rainfalls that assume storm bursts and spatial uniform temporal patterns cannot capture that actual variability of observed rainfall. This insight motivated a change in practice from simple average rainfall intensity or single rainfall burst approaches to ensemble and Monte Carlo methods to better capture the natural variability of rainfall.

The design of stormwater infrastructure and understanding of runoff for urban areas involves decisions at multiple scales. This insight can be combined with ensembles of design rainfall patterns to determine the appropriate rainfall inputs as shown (for example) by the Box and Whisker plot of peak runoff (discharge) to the catchment outlet in Figure 9.6.3.

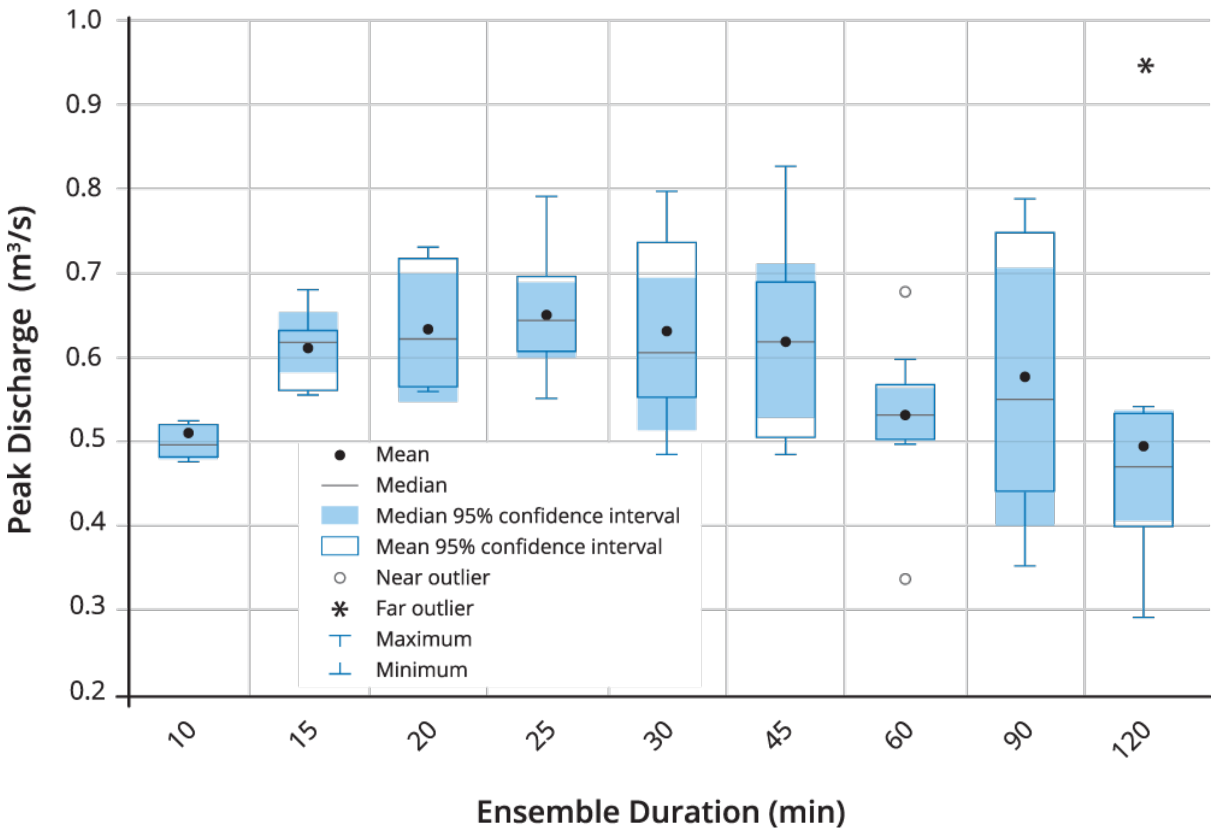

Figure 9.6.3. Example of a Box and Whisker Plot of Peak Stormwater Runoff Utilised to Select the Critical Storm Burst Ensemble and Other Design Information

Figure 9.6.3 indicates highest average and median peak discharge is generated by the ensemble of 10 storm bursts of 25 minute duration at the catchment outlet. A small number of higher values of peak runoff also occur in the 45 minute (maximum value) and 120 minute (far outlier value) durations which could be used to test the potential maximum hazard of surface flows. Conveyance infrastructure within the catchment should be designed using ensembles of storms with durations up to and including 25 minutes to account for impacts of smaller duration storms upstream of the outlet. Different design ensembles may apply in situations that incorporate within catchment storage solutions and at different locations in the urban catchment.

This improved approach to design rainfall inputs to models is particularly important for urban catchments that are significantly different to rural catchments because they generate runoff from majority of rainfall as shown in Figure 9.6.4.

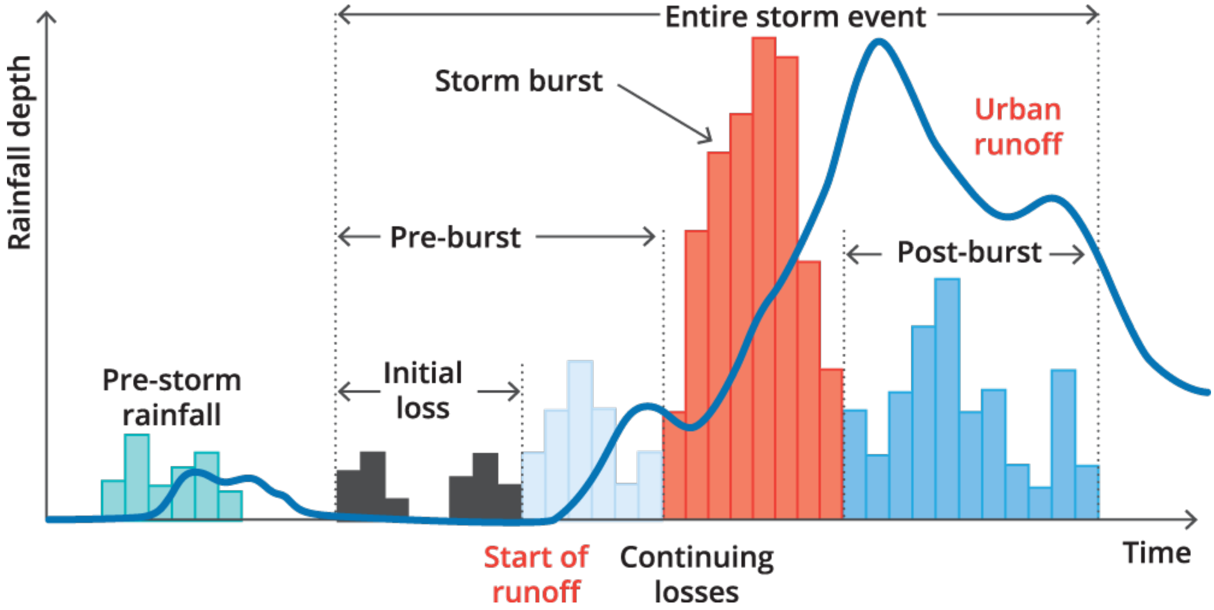

Figure 9.6.4. Rainfall Runoff Processes in Urban Catchments

Figure 9.6.4 demonstrates that urban runoff can be generated by pre-burst, burst and post-burst proportions of complete storms (entire storm event). There are many different configurations of pre-burst, burst and post-burst rainfall in real rainfall events that should be considered in analysis of urban hydrology. Urban designs based on a single burst pattern of rainfall or peak rainfall assumptions can overlook substantial runoff rates and volumes which may adversely impact on the performance of inlet structures in conveyance networks, volume management measures, roads and overland flow paths (Coombes et al., 2015b).

A range of updated rainfall products are available from the ARR Data Hub (Babister et al., 2016), including new spatially distributed IFD, Areal Reduction factors (ARF), design temporal patterns for burst rainfall, hydrological losses, and pre-burst rainfall – as summarised in Table 9.6.5.

Table 9.6.5. Summary of Updated Design Rainfall Processes

Input	ARR 1987	Pre Update	ARR 2016
IFD	Paper maps	BoM web page	Updated BoM web page. Book 2, Chapter 3.
ARF	Figure 2.7 from US data	FORGE work (except NSW)	New equations derived using Australian data. Book 2, Chapter 4.
Temporal patterns	Single temporal pattern of design burst rainfall based on Average Variability Method (AVM)	AVM, filtered for embedded burst	Ensemble of real storms. Book 2, Chapter 5.
Spatial pattern	Centroid	Spatially distributed IFD	Spatially distributed IFD

Input	ARR 1987	Pre Update	ARR 2016
Climate change			Factors available from Book 1, Chapter 6 and the ARR Data Hub.
Losses	State based advice, sometimes based on data	Calibrated in the hydrologic Model.	Calibrated losses. Uncalibrated models use losses available from Book 5, Chapter 5 and the ARR Data Hub.
Pre-burst	Allegedly incorporated into advice	Mixed	Estimates provided on ARR Data Hub. Use 60 minute pre-burst rainfall with burst rainfall ensembles of durations less than 60 minutes

The different rainfall inputs to hydrology and hydraulic models are discussed in Book 2. The updated IFD design rainfall data is available from the BoM website. Derivation of the IFD data using the additional rainfall records is outlined in Book 2, Chapter 3, Section 4 and the application of the updated IFD design rainfalls is presented in Book 2, Chapter 3, Section 9.

ARF are available from the ARR Data Hub and is discussed in Book 2, Chapter 4. Design rainfalls (IFD) only apply at a point in a catchment. When estimates of rainfall runoff are required for catchments with areas greater than 10 km^2, the design rainfall intensities at a point are not representative of the areal average rainfall intensity for the entire catchment. The ARF is the ratio between the design values of areal average rainfall and point rainfall, for same duration and Annual Exceedance Probability (AEP). Application of ARF is outlined in Book 2, Chapter 4, Section 3.

Most runoff-routing methods utilise design temporal patterns to determine the timing of rainfall falling on catchment and generate hydrographs of runoff. The traditional use of a single average temporal pattern has been found to be inadequate for hydrological analysis due to the variability of natural rainfall patterns (Book 2, Chapter 5) and of the characteristics of urban catchments (Book 9, Chapter 3). The application of design temporal patterns as outlined in Book 2, Chapter 5, Section 9. Ensembles of design temporal patterns that are more likely to capture these natural and human variabilities are available from the ARR Data Hub. It is noted that two different ensemble patterns are provided, point rainfall patterns for catchments with areas up to 75 km^2 and areal rainfall patterns for catchments with areas greater than 75 km^2.

Climate change has the potential to alter the frequency and severity of rainfall events, storm surge and floods by altering rainfall IFD relationships, rainfall temporal patterns, continuous rainfall sequences, antecedent conditions and baseflow regimes (Book 1, Chapter 6; Book 2, Chapter 2, Section 4). Climate change factors are presented as changes in average temperature and associated increases in rainfall intensity and losses for selected global emission pathways in the ARR Data Hub[1].

The ARR Data Hub provides regional rural losses for complete storms and pre-burst rainfall. In urban areas, the median values of local losses should be utilised wherever possible. The average initial losses from urban impervious surfaces is less than 1 mm (Book 4, Chapter 2, Section 7) and ranges from 1 mm to 4 mm for urban effective impervious areas (Book 5, Chapter 3, Section 4). In most cases, storm burst loss is equal to median storm loss less pre-burst rainfall.

Rural and regional loss assumptions should not be a default assumption for urban areas and a hierarchy for selecting urban losses is highlighted as follows:

- Use local losses based on GIS investigations, local knowledge and observations. Losses derived at a regional scale are not local losses- use local losses in small scale models. Note that a well-constructed model with adequate spatial scale should account for effective impervious area and connectivity effects

- Regional losses (Book 5, Chapter 3, Section 4 and Book 5, Chapter 3, Section 5): Impervious area losses: IL: <1 mm, CL: 0 mm/hr; Effective Impervious Area: IL: 1-2 mm, CL: 0 mm/hr; Pervious area ≈ rural losses

- Rural losses: Urban losses are some proportion of rural losses

Continuous simulation of rainfall runoff processes is aided by the increased availability of continuous (also known as pluviograph or instantaneous) rainfall from the Australian Bureau of Meteorology (BOM). However, as discussed in Book 9, Chapter 6, Section 4, longer synthetic continuous rainfall records are usually required to understand the impacts of rarer runoff events. Development and availability of synthetic continuous rainfall sequences are discussed in Book 2, Chapter 7. Additional discussion of synthetic continuous rainfall records that incorporate regional layers (surfaces) of spatial observed climate observations is also provided by Coombes and Barry (2015) and Coombes and Barry (2018). This guideline also provides software to generate multi-site continuous synthetic rainfall (Multi-site Rainfall Simulator) at http://arr.ga.gov.au/.

Radar rainfall (refer Cecinati et al. (2017)) can be used to interpolate between point rainfall observations for use in hydrology and 2D hydraulic models. There have been many studies that have developed methods to correct errors in radar rainfall but some residual errors are intrinsic to radar rainfall that should be resolved by spatial and temporal comparison to point rainfall observations.

6.4.3. Runoff From Properties

Stormwater runoff from roofs and properties, at the lot scale, is the basic building block of urban stormwater catchment behaviour ((Goyen and O'Loughlin, 1999a; Stephens and Kuczera, 1999) and Book 9, Chapter 3). Runoff from properties involves a complex interaction of roofs, yards, paved areas, gardens, and adjoining roads and footpaths as shown for a residential property in Figure 9.6.5.

[1] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

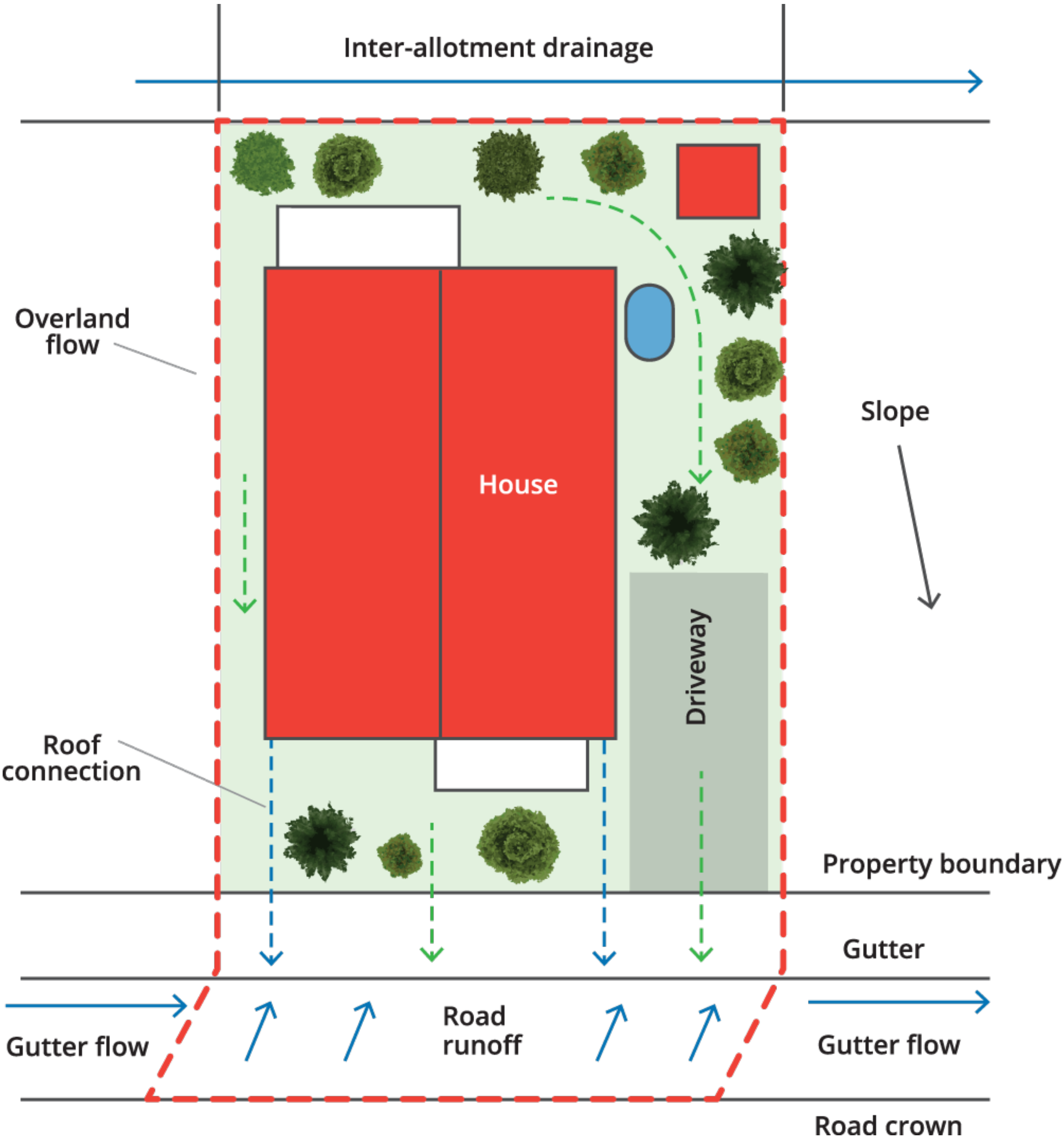

Figure 9.6.5. Stormwater Runoff from Roofs and Properties – Lot Scale Effects

Figure 9.6.5 demonstrates the pathways of stormwater runoff from different surfaces within a property. These runoff processes are dominated by directly connected impervious surfaces, indirectly connected surfaces and pervious surfaces. Rain falling on impervious roof surfaces flow into roof gutter storages which discharge via downpipes into pipes connected to the street gutter or pipe network. Runoff from impervious driveway surfaces and adjacent road surfaces discharge to street gutters. These impervious surfaces facilitate highly efficient translation of rainfall into runoff, are subject to small depression storage losses, and are mostly directly connected to street gutters. Rain falling on pervious yard areas is partially retained in depression storages and infiltrates into soil profiles prior to generation of runoff from residual rainfall. These types of pervious surfaces are relatively inefficient at generating runoff and are often indirectly connected to street gutters or pipe networks. Urban properties can also include impervious areas that discharge stormwater to pervious surfaces or storages (for example, rainwater tanks, onsite detention and raingardens) that partially disconnect these surfaces from street gutters.

Runoff from impervious surfaces may also arrive at street gutters more rapidly than runoff from pervious surfaces. In many situations, pervious surfaces may not generate runoff for frequent rainfall events. These runoff behaviours are influenced by the configuration of property assets (including building form), topography and stormwater management measures. In situations where allotments slope away from roads, runoff from roofs and impervious surfaces may be directed to an inter-allotment conveyance (easement drainage) network. Local authorities will often specify locations of stormwater discharges from properties – this is known as a legal discharge point. Subsoil drains are sometimes used on properties to lower water tables around buildings or in waterlogged areas and discharge stormwater from properties.

Property scale influences are fundamental to urban stormwater runoff. However, there has been limited testing at this scale (Stephens and Kuczera, 1999), and designs of roof and property drainage are not clearly defined (Jones et al., 1999). A major challenge for simulation of urban stormwater runoff is the behaviour of individual properties and accumulation of these property behaviours throughout urban catchments (Goyen (2000), Coombes (2015); Book 9, Chapter 3). The cumulative impacts of properties on the behaviour of catchments are defined by the timing, volume and rate of stormwater runoff from each property. The runoff behaviour of properties can also be altered by a range of onsite stormwater management approaches including disconnection of roof downpipes from street gutters, raingardens, landscaping, rainwater tanks, infiltration measures, onsite detention and green spaces (refer Book 9, Chapter 3 and Book 9, Chapter 4). Local authorities can apply restrictions on the flow rate, quantity and quality of stormwater that discharges from a property to encourage onsite management of stormwater to avoid or reduce downstream impacts (Chocat et al., 2001; Patouillard and Forest, 2011; Walsh et al., 2012; Everard and McInnes, 2013).

Calibration or verification of urban stormwater modelling frameworks at the catchment scale does not imply that the sub-catchment or local behaviours in models are also correctly described (Goyen and O'Loughlin, 1999a; Stephens and Kuczera, 1999; Kuczera et al., 2006; Coombes, 2015). Attention to local detail in stormwater design is required to ensure that potentially overlooked local processes do not generate local failures or excessive infrastructure or unexpected downstream consequences. The problems generated by approximated local behaviours can become worse in areas subject to increasing urban density and infill development. Kemp and Myers (2015), for example, found that increases in urban density of 18% generated 16% increase in runoff volumes and a 300% increase in expected flood damages for 20% AEP storm events.

Simple methods for design of roof gutters, downpipes and property drainage are provided in Australian Standards (for example, AS/NZS 3500.3), by suppliers of roofing materials, government authorities and the Plumbing Code of Australia. These approaches include nominal and general methods. Nominal methods apply to single dwellings on properties with land areas up to 1,000 m^2 by providing “deemed to comply” specifications of infrastructure (configuration, minimum pipe sizes, depth of cover over pipes and slopes).

Design calculations are provided for more complex land uses and larger properties. These guidelines highlight the need to avoid ponding against buildings, flows into buildings and management of overland flows from adjoining properties. Large residential, commercial and industrial properties and car parks include more complex and dendritic stormwater management systems (for example Figure 9.6.6).

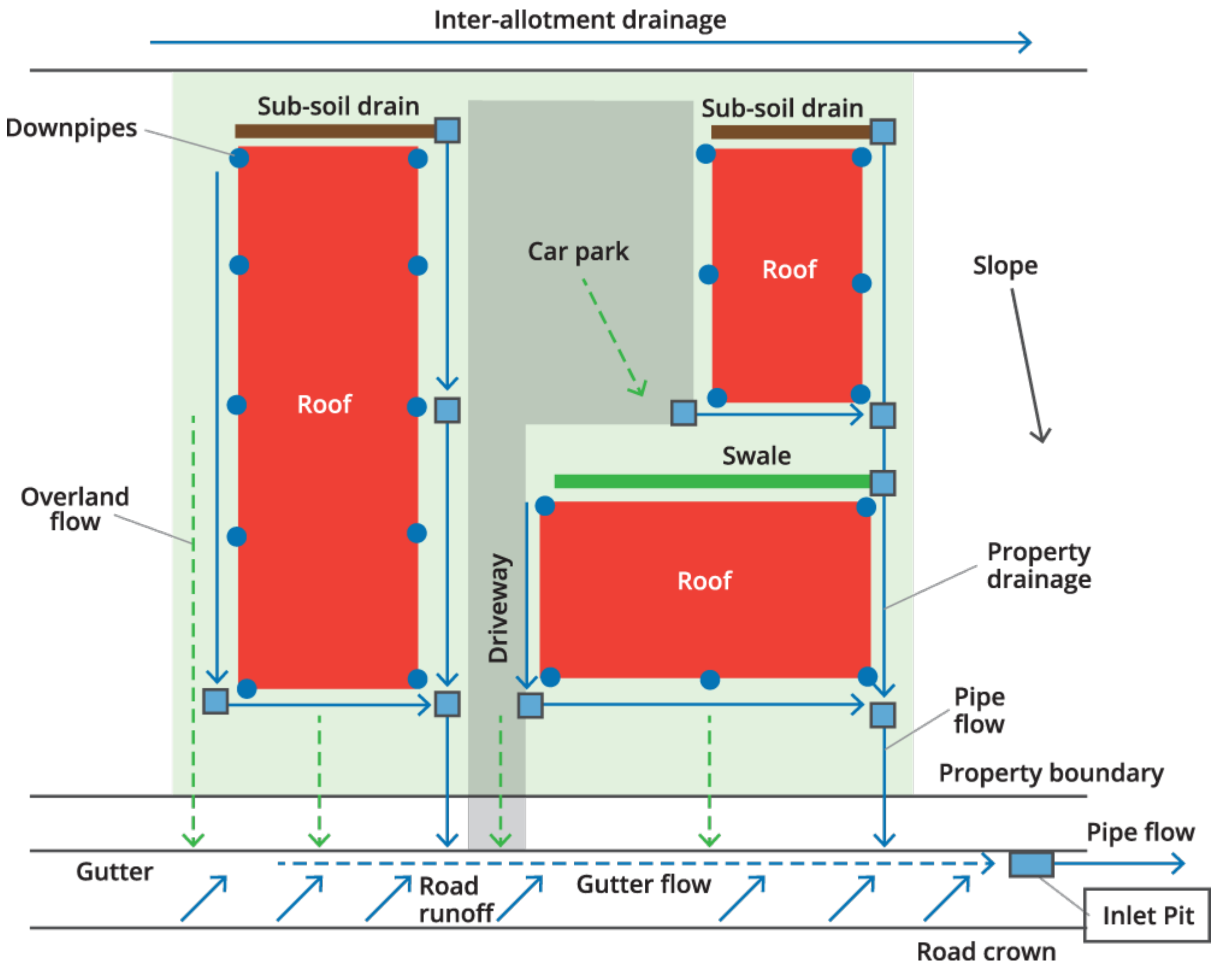

Figure 9.6.6. Stormwater Management System for Larger Properties with Complex Land Uses

Figure 9.6.6 shows that stormwater management schemes within properties may combine multiple pathways of stormwater runoff from different surfaces that have variable levels of connection to the street gutter or inlet pit in the street conveyance network. The performance of these networks may be affected by in-pipe attenuation effects, volume management measures, and substantial variations in the timing and magnitude of runoff to sub-catchment outlets. These outflows from properties are surface flows, or direct inflows to pipe networks in streets or inter-allotment conveyance networks.

Approximate or general design methods are based in rules derived from simple Rational Method assumptions and utilise catchment areas (roofs, paved surfaces and gardens), proportions of imperviousness, slopes, assumed times of concentration with associated average rainfall intensities and runoff coefficients to generate maximum or peak flow rates. A five minute time of concentration and associated rainfall intensity was commonly assumed in design processes for roof and property drainage. Performance standards for roofs have been defined by choice of rainfall intensity of a 5% AEP for roof gutters and of a 1% AEP for box gutters. Design of conveyance networks within properties aim to avoid surcharges and overland flows for 1 EY in low density areas and up to 5% AEP for important land uses (such as hospitals and aged care facilities) that may be vulnerable to greater risk or inconvenience. The volume, pattern and timing of stormwater runoff are not considered in these approaches which may lead to under-performance of stormwater management measures included unexpected surface flows on properties.

Field measurements suggest that travel time to street gutters from residential properties is two minutes or less (Stephens and Kuczera, 1999; Coombes, 2002). The assumption of five minute time of concentration in ARR 1987 (Pilgrim, 1987) was based on the lowest available time interval of IFD rainfall at the time. Revised IFDs available from the BoM provide values for rainfall intensity that commence at a one minute duration which permits use of finer detail in design and to account for shorter flow times to outlets. Observations by Stephens and Kuczera (1999), Goyen (2000) and Coombes (2002) indicate that initial losses from roof gutter systems range from 0 mm to 1 mm and continuing losses range from 0% for metal roofs to 20% for dry tile roofs. Average depression storage losses of impervious surfaces can range from 1 mm to 10 mm and average losses from pervious surfaces range from 2 mm to 20 mm.

Goyen and O'Loughlin (1999b) highlighted that spatial and temporal patterns of rainfall losses and their magnitude have significant impacts on peak stormwater runoff. Larger scale and more general estimates of losses are provided in Book 3, Chapter 3. Wherever possible, local information on losses should be incorporated in analysis of stormwater runoff and associated designs of infrastructure.

More detailed hydrograph routing methods may be required for larger properties with complex land uses to design infrastructure for given performance standards, and to understand the behaviour of the stormwater management system. The need to manage inflows of groundwater and surface runoff to basements on some properties will also require volume based analysis to understand the extent of flooding and to design pump out infrastructure. Argue (2004) provides a range of simple methods for including volumes in the small scale design processes that are known as “regime in balance” and accounting for “emptying times” of storages.

Stormwater management strategies for larger or more complex properties should be designed or analysed using event based hydrograph routing methods that utilise storm burst patterns and pre-burst rainfall as inputs. The pre-burst rainfall, rainfall intensities and patterns of storm bursts for a given location can be downloaded from the ARR Data Hub http://data.arr-software.org/ and included in models of stormwater runoff. These rainfall inputs are provided in most proprietary software packages.

This modelling process includes details of different surfaces within sub-catchments that influence stormwater runoff to inlet structures within the property stormwater management network. The analysis should include the characteristics of pervious and impervious surfaces – such as initial and continuing losses, sub-catchment areas, slopes and details of overland flow paths. This approach is similar to the design and analysis process for public stormwater conveyance (street drainage) networks.

Use of ensembles of storm burst rainfall will ensure that the stormwater management system for a property is tested by a range of equally likely storm patterns and volumes of rainfall. This will permit a more complete understanding of potential surface flow paths within the property and in the adjacent street gutter, and the impacts on downstream infrastructure. However, use of complete storms or inclusion of pre-burst rainfall with the burst rainfall patterns will assist with defining the likely magnitude of overland flow behaviours at the property. Initial losses in the analysis may need to be set to zero if the magnitude of pre-burst rainfall is greater than the capacity of depression storages on the property. At some locations, the residual pre-burst rainfall may also contribute to additional runoff and overland flows within the property. These approaches can be combined in a range of computer modelling packages.

The ability of peak flow or event based models to describe runoff behaviours are limited in situations where the joint probability of antecedent conditions and storm events is not well defined (refer to Book 4, Chapter 3, Section 3) and there are continuous responses to complete storm events. These limitations apply to stormwater strategies that include volume storage measures, rainwater or stormwater harvesting, and water quality solutions.

In these situations, continuous simulation using real local rainfall or synthetic rainfall sequences can be utilised to test the continuous interactions between key components of the stormwater management. The results from continuous simulation can be directly interrogated to understand key performance criteria such as annual average reduction in water demand, stormwater runoff and nitrogen loads created by rainwater harvesting and raingardens. Alternatively, continuous simulation can provide distributions of available storage in volume management measures (such as rainwater tanks, infiltration measures and bioretention devices) or soil profiles prior to storm events versus frequency of storm events that can be used in event based analysis (Coombes and Barry, 2008b; Hardy et al., 2004) as shown, for example, in Figure 9.6.7.

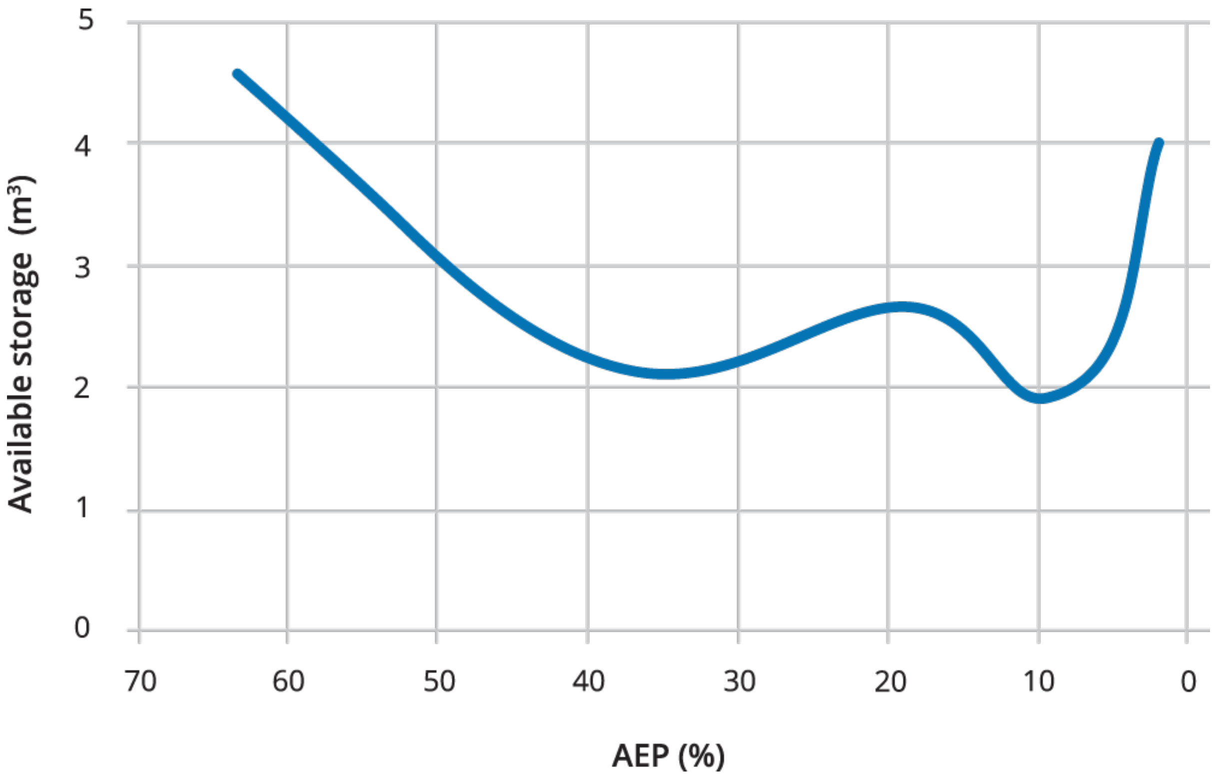

Figure 9.6.7. Example Distribution of Available Storage Prior to Storm Events versus Annual Exceedance Probability (AEP) of Storm Events

Figure 9.6.7 (for example,) demonstrates the average retention storage available in rainwater tank (capacity of 5 m^3 collecting runoff from a 100 m^2 roof area and supplying household indoor and outdoor uses) prior to storm events of a given AEP that was derived using continuous simulation. This type of information can be used in event based models to determine stormwater peak flows and runoff volumes. These results will vary significantly with different land uses, building form and throughout Australia.

6.4.4. Sub-Catchment Runoff to Inlet Structures

Sub-catchments define an urban area that discharges stormwater runoff to an inlet structure within a stormwater management network. There is further discussion of conveyance networks in Book 9, Chapter 5, Section 1 to Book 9, Chapter 5, Section 3 and of inlet structures in Book 9, Chapter 5, Section 5. The configuration and characteristics of the urban area within a sub-catchment will define the hydrological response that produces stormwater inflows to a stormwater network. These surface flows define the performance of an inlet structure as inflows to a conveyance network and as surface bypass flows. An example of a simple urban sub-catchment is provided in Figure 9.6.8.

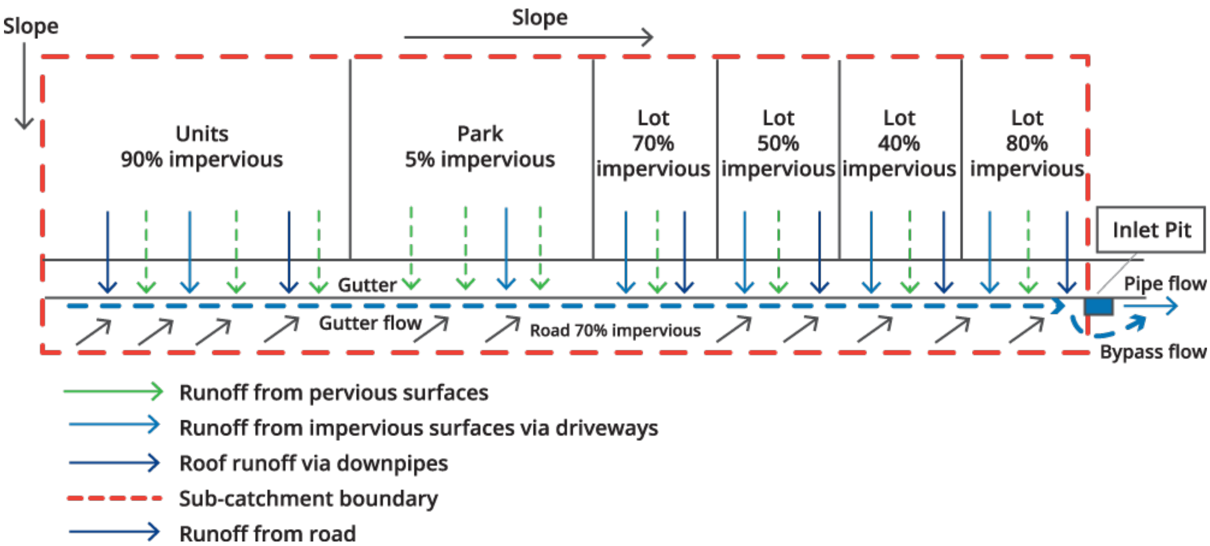

Figure 9.6.8. Example of a Simple Urban Stormwater Sub-Catchment

Figure 9.6.8 highlights that an urban sub-catchment may contain a range of different land uses, including (for example) unit and detached residential dwellings on properties, a park and part of a road. These land uses incorporate different surfaces, including roofs, paved areas (impervious), garden and grassed areas (pervious) that produce different regimes of stormwater runoff.

The behaviour of the sub-catchment surfaces can be estimated using lumped catchment approximations which are based on sub-catchment area, the total impervious area (TIA) and a travel time (or time of concentration) for a critical rainfall duration to the inlet structure (refer Book 5, Chapter 2, Section 2 and Book 9, Chapter 5, Section 5). For example, a sub-catchment area of 4,250 m^2 with an impervious proportion of 56% and time of concentration which depends on rainfall intensity, slope and distance to inlet structure. In the absence of other data, these types of approximations could be used in simple calculations or in computer models. However, it is preferable to construct analysis of urban sub-catchments using local details which can be sourced from site inspection, survey plans and inquiry using GIS.

Impervious or pervious surfaces can be directly connected or disconnected to inlet structures in conveyance networks. These surfaces may be also distant from the inlet structure or near the inlet. Thus the level of connectedness and distance of impervious areas from inlet structures should also be considered in analysis of stormwater runoff in urban areas. In addition, the analysis should account for surfaces that discharge to inlets via rapid conveyance mechanisms, such as street gutters, and for other surfaces that may discharge to inlets via slower conveyance processes such as across pervious surfaces (green spaces) or via storages.

An urban sub-catchment often includes depression storages, and a mosaic of different surfaces, runoff rates, storages and cumulative connectivity. The order of actions in the connectivity of different types of surfaces with storages to the inlet can dramatically change travel times and peak flows. Roofs may discharge via pipes to street gutters that facilitate rapid transfer of runoff volume to inlets. Runoff from road surfaces to the gutter may arrive at a similar or earlier time (refer Figure 9.6.8).

It is unlikely that lumped catchment approximations will provide reliable estimates of stormwater runoff from urban sub-catchments that include a range of different land uses and catchment storages with variable connectivity to inlets. Use of lumped catchment with TIA approximations may generate over-estimations of stormwater peak flows. Distributed methods of analysis may be more appropriate for ungauged catchments, where there are storages within catchments or for analysis using more robust runoff routing in computer models.

Urban sub-catchments include Directly Connected Impervious Areas (DCIA), Indirectly Connected Impervious Areas (ICIA) and pervious areas as described in Book 5, Chapter 3, Section 4. Limited regional investigations suggest that a combination of these effects produced Effective Impervious Areas (EIA) which are 55%-65% of the TIA of urban sub-catchments. Estimates of indirectly connected areas are further impacted by interactions between impervious and pervious areas, by storage in sub-catchments with Water Sensitive Urban Design (WSUD) measures and are influenced by Antecedent soil Moisture Conditions (AMC).

Other impervious surfaces may discharge via driveways to the street gutter which produces a different time for stormwater runoff to reach the inlet structure. Pervious surfaces also discharge to the street gutter, partially via impervious surfaces, to the inlet. Thus the timing of the arrival of runoff volumes to the inlet is dependent on these many different configurations and characteristics within the sub-catchment. So the performance of the inlet structure and the magnitude of surface bypass flows are dramatically affected by these considerations. These complex processes can be better described by semi-distributed (link-node) and distributed (grid) computer models (Book 5, Chapter 2, Section 4; Book 6, Chapter 4, Section 7) that explicitly combine these details with pre-burst rainfall and ensembles on burst rainfall patterns.

Regional analysis of a small number of urban catchments provides estimated initial losses of 1 – 3 mm for EIA and 20 – 30 mm for indirectly connected areas in sub-catchments (Book 5, Chapter 3, Section 5). Estimated median continuing losses were 2.5 mm/hour in South East Australia and 1 – 4 mm/hour elsewhere. These event based regional values should only be used in the absence of local data. It is essential that assumptions about losses in stormwater models are based on assessment of local conditions. The magnitude of losses is also impacted by AMC which is altered by garden watering in urban areas and by available storage in volume management measures throughout the sub-catchment. It is unlikely that event based models can fully account for these effects. Sensitivity checks, Monte Carlo processes and continuous simulation can be utilised to include the variation in AMC and available storage within urban sub-catchments.

Urban drainage was historically designed using peak flows derived using peak rainfall intensity or peak rainfall bursts in accordance with the assumption that peak flowrates only affect conveyance infrastructure. Many urban drainage networks are operating below anticipated service levels due to a range of impacts including increased density of urban areas. Analysis by Coombes et al. (2015a) indicates that the absence of stormwater runoff volumes in design processes based on peak runoff assumptions may partially explain under-performance of some urban drainage networks. The performance of inlet structures and therefore drainage networks can also be affected by the volume of stormwater arriving at the structure, variations in rainfall temporal patterns and by pre-burst rainfall that was not included in the design process. The uncounted volumes of stormwater runoff in peak flow and storm burst assumptions can become additional and unexpected overland or bypass flows in urban systems.

6.4.5. From the Inlet to the Outlet

Rainfall runoff from sub-catchments accumulates as inflows to conveyance networks or as surface flows throughout urban catchments that discharge towards an outlet (Book 9, Chapter 5). A network of conveyance infrastructure may incorporate pipes, open channels, roadways and open space. These networks often include water quality, volume management and flow control infrastructure (refer to Book 9, Chapter 4) that are incorporated in sub-catchment scale processes (such as source and neighbourhood controls: see Figure 9.5.1) or as regional controls at the outlet.

Analysis and design of stormwater management and flooding in urban areas was historically based on separate hydrology and hydraulic processes, and is focused at the network scale. A key objective of these processes was determination of flows in conveyance infrastructure such as pipes and open channels to avoid surcharges and bypass flows at inlets (refer to Book 9, Chapter 5, Section 5) to avoid nuisance, property damage and risk to life (refer to Book 9, Chapter 5, Section 2 and Book 6, Chapter 7). These urban conveyance networks include significant surface flows, usually along roads and through open spaces, from sub-catchments into and throughout conveyance networks. These flows from sub-catchments to inlets and within conveyance networks were determined as a hydrological process as an input to hydraulic models of conveyance networks (see Book 9, Chapter 5, Section 6and Book 5, Chapter 2).

The conveyance network is a framework of sub-catchment inputs. Urban stormwater design typically employed pipe network hydraulic models that utilise peak inflows or hydrographs as inputs (refer to Figure 9.5.18). More advanced one-dimensional models were also available that can be applied to simulation of conveyance networks (refer to Book 6, Chapter 4, Section 6; Book 5, Chapter 6 and Book 9, Chapter 5).

Overland or surface flows are a key consideration in analysis and design of urban stormwater management infrastructure. The dominant urban hydraulic response to rare rainfall events (such as 1% AEP) is often overland flows on roads and across open space. Emerging methods of analysis and design of urban stormwater involve combined hydrology and hydraulic models to better understand surface flows throughout urban catchments. These methods include coupled one and two-dimensional models, and direct rainfall (rainfall-on-grid) models. Book 6, Chapter 4, Section 7, and Babister and Barton (2012) provide detailed discussion about these approaches.

The flowrates, depth and area of surface flows in urban catchments are highly sensitive to different temporal patterns and volumes of rainfall (Babister and Barton, 2012). Similarly, Goyen (1981), Goyen (2000) and Coombes et al. (2015a) found that the performance of conveyance infrastructure also varies with temporal patterns and volumes of rainfall. It is recommended that ensembles of ten temporal patterns of design rainfall are used for investigation of the hydrology and hydraulic processes in urban areas. The separation of hydrologic and hydraulic routing is often blurred in analysis of urban areas which fosters complicated decisions around the use of hydrologic inputs and their interaction with hydraulic models. An overview of the difference approaches to rainfall inputs provided by this guideline is compared to ARR 1987 approach in Figure 9.6.9.

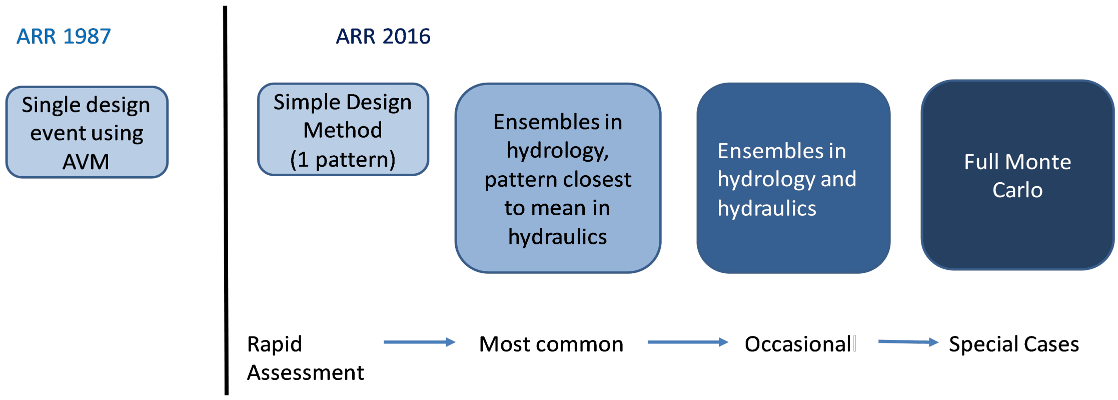

Figure 9.6.9. Changes in Design Modelling Techniques for Urban Areas

Figure 9.6.9 highlights that this guideline provides ensembles of 10 temporal patterns for each region that is a departure from the single event process supported by ARR 1987. These rainfall inputs can be used in hydrology and hydraulic modelling as required for different design and assessment tasks (refer to Book 2, Chapter 4 for further detail). The rapid assessment approach is not recommended for design of urban conveyance networks and the Monte Carlo processes can be used in special cases. It is expected that rainfall ensembles in hydrologic simulations, and in hydrologic and hydraulic simulations would be commonly utilised in urban conveyance networks. The process of using rainfall ensembles in hydrology is outlined in Figure 9.6.10.

Figure 9.6.10 shows that the inputs to analysis of the conveyance network include IFD information from the BOM, ensembles of rainfall temporal patterns, regional losses, pre-burst rainfall and Areal reduction factors from the ARR Data Hub. Wherever possible, local losses derived in accordance with Book 9, Chapter 6, Section 4 and Book 9, Chapter 6, Section 4 should be used in preference to regional losses for urban areas. These inputs are used in a hydrology model to generate ensembles of peak flows throughout the urban catchment for various storm durations and the required quantiles or AEPs of storm events. Mean peak flows are derived for key locations in the catchment and the rainfall temporal pattern that produces peak flows closest to the mean peak flows are utilised in the hydraulic model. This approach may be better suited to models with longer run times as considerable time can be expended determining critical durations in both hydrology and hydraulic models.

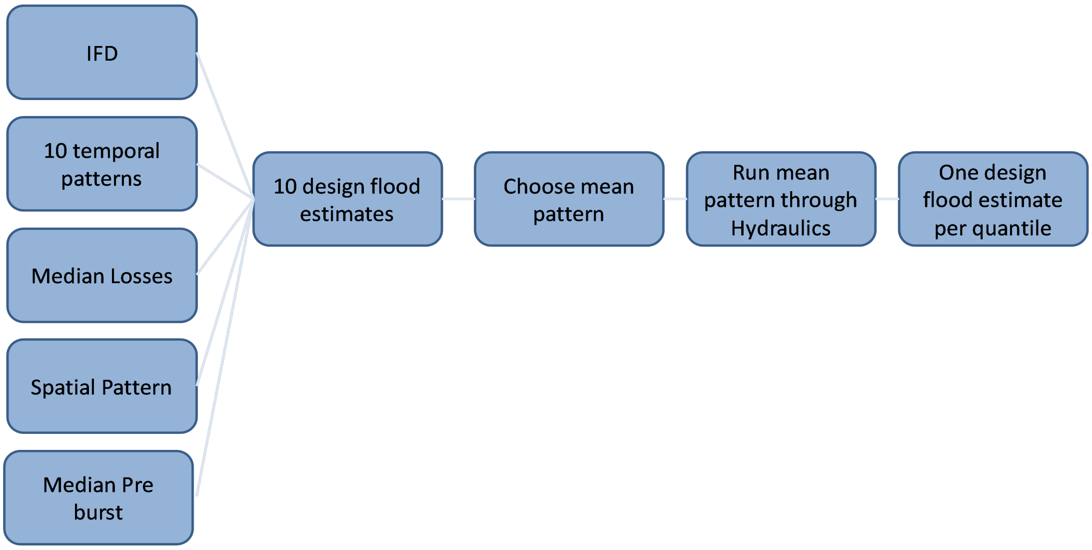

Figure 9.6.10. Design Process that Utilises Rainfall Ensembles in Hydrology to Select the Rainfall Pattern Closest to Mean Peak Flows for use in Hydraulic Analysis

The processes outlined in Figure 9.6.10 produce a single estimate of flood depth for each selected quantile or AEP. It is important to highlight that the critical rainfall duration and temporal pattern estimated using the hydrology model is likely to be different to the critical rainfall and temporal pattern relevant to the hydraulic simulations. These differences between critical hydrology and hydraulic inputs can have substantial impacts on the design of infrastructure and understanding of surface flows.

In situations where the hydraulic impacts of the design processes are significant, rainfall ensembles can be used in the hydrologic and hydraulic simulations as outlined in Figure 9.6.11.

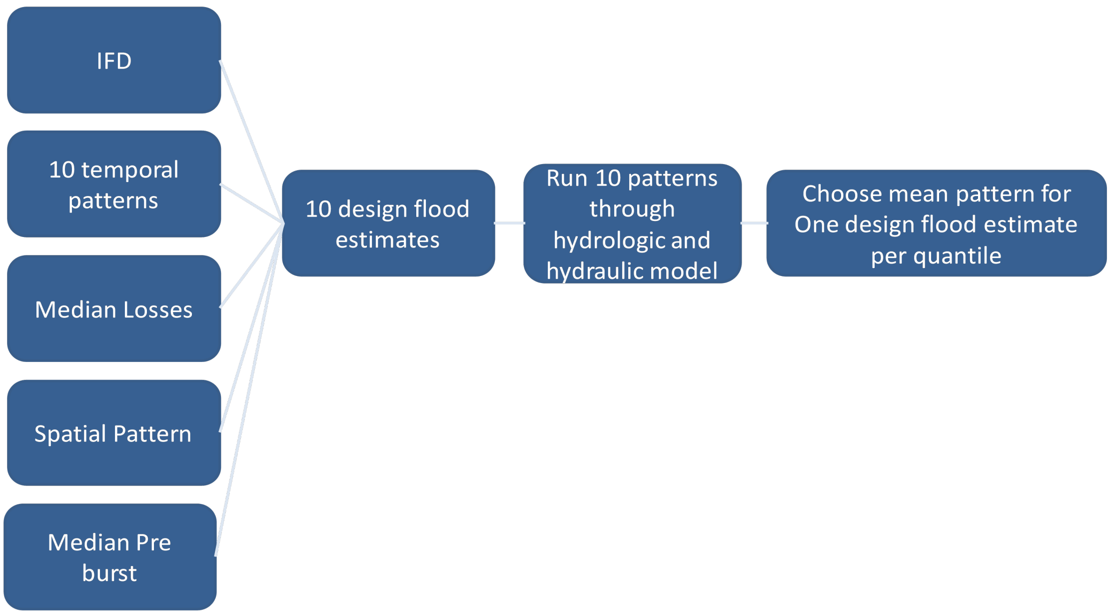

Figure 9.6.11. Design Process that Utilises Rainfall Ensembles in Hydrology and Hydraulic Simulations to Select the Mean Pattern for Analysis of Flooding

Figure 9.6.11 outlines that process for utilising ensembles of rainfall patterns in hydrology and hydraulic models. This process is better suited to situations where there are shorter model run times, critical flooding considerations and for coupled hydrology and hydraulic models. The processes outlined in Figure 9.6.10 and Figure 9.6.11 may also need to be applied to understand critical rainfall durations and patterns at key internal locations within catchments.

A brownfield case study based on the Woolloomooloo catchment in Sydney demonstrates the use of ensemble temporal patterns of rainfall and effects on the performance of hydraulic models used for design or assessment of conveyance networks (see Ward et al. (2018)). The catchment area is approximately 1.6 km^2 and has been heavily urbanised with limited open spaces or pervious areas. The catchment is also characterized by undulating terrain and contains known depression storages. The catchment drains to the harbour through a pit and pipe network, with the streets acting as overland flow paths (Figure 9.6.12).

Figure 9.6.12. The Woolloomooloo Catchment in Sydney

This guideline supports a number of modelling techniques and Table 9.6.1 and Table 9.6.2 provide guidance on selection of modelling approaches. Use of coupled 1D/2D and direct rainfall models were necessary to understand within catchment surface flows and flooding. The potentially short model run times and need to understand local flooding supports use of rainfall ensembles in both hydrology and hydraulics models. This study combined a well-known hydrology model with a popular 2D and 1D hydraulic model that relies on second order finite-difference schemes to simulate the hydrodynamics of floodplains and waterways.

This case study discusses three modelling options that were designed to account for within catchment overland flows and flooding:

- Use of a hydrology model to generate overland flows from small sub-catchments for use in a coupled 1D/2D hydraulic model. Individual properties, roofs and small area land surfaces were assigned as sub-catchments (refer to Book 9, Chapter 6, Section 4) in the hydrologic model to capture the rainfall concentration phase of stormwater runoff into the hydraulic model. This approach is necessary to understand within catchment flooding.

- A concentrated direct rainfall model where rainfall is applied to polygons of different land surfaces separated by perviousness and connectivity to the hydraulic 1D/2D model. These concentrated land surfaces also account for rainfall losses.

- Direct rainfall-on-grid where rainfall, after accounting for initial and continuing losses, was applied to all active grid cells. A fixed grid of $2m^2$ was employed in the hydraulic model.

Direct rainfall methods are known to trap volumes of rainfall in depressions and in areas with high roughness throughout 2D hydraulic models. The value of a carefully constructed direct rainfall model is the ability to identify sub-catchment flow paths, contributing areas and storage. However, the designer must ensure that catchment storages or initial losses are not doubled counted in simulations by the addition of regional loss assumptions. Given that there is a paucity of research into the accuracy of direct rainfall models, it is recommended that results of direct rainfall methods are compared with traditional methods by examination of the characteristics of hydrograph produced by both methods (Babister and Barton, 2012). A suitable method of representing buildings and good quality topography data is also required to produce accurate urban stormwater runoff behaviours. A mass balance or volume error check is also recommended.

The historical process of determining rainfall loss parameters using ARR 1987 assumptions, including soil type and antecedent moisture content parameters (AMC) from the ILSAX model, is provided in Table 9.6.6 for comparison.

Table 9.6.6. ARR 1987 Rainfall Loss Parameters

Parameter	Value
Paved Area Depression Storage (Initial Loss)	1.0 mm
Grassed Area Depression Storage (Initial Loss)	5.0 mm
SOIL TYPE	3
Slow infiltration rates. This parameter, in conjunction with the AMC, determines the continuing loss	
AMC	3
Description	Rather wet
Total Rainfall in 5 Days Preceding the Storm	12.5 to 25mm

This guideline provides a range of up-to-date parameters for use in analysis. The catchment is located within the East Coast South temporal pattern region. Temporal patterns for the East Coast South region and Intensity Frequency Duration (IFD) rainfall depths were downloaded from the ARR Data Hub website. This information is combined to construct ensembles of 10 rainfall patterns for the required flood quantiles (AEP). This case study focuses on the 1% AEP storm. The initial and continuing storm losses of 28 mm and 1.6 mm/hour for rural areas, and median pre-burst rainfall of 1.1 mm associated with a one hour 1% AEP storm event can also be downloaded from the ARR Data Hub.

It is recommended that varied rainfall losses are applied to different types of surfaces in the catchment. These surfaces include urban pervious areas such as parks, and impervious areas such as roads, median strips and building roofs. The identified impervious areas were split up into Effective Impervious Area (EIA) and Indirectly Connected Impervious Area (ICIA).

Effective Impervious Area represents the portion of a catchment area that has an impervious response. Due to the highly urbanised nature of the catchment this portion was identified as 75% of the total impervious area. The remaining area that is not classified as Effective Impervious Area is Indirectly Connected Impervious Area (25%). Building roofs were identified separately as Indirectly Connected Impervious Area as the down pipes were not assumed to directly discharge into the storm water pipes. The information from the ARR Data Hub is modified by loss values for urban catchments that are provided in Book 5, Chapter 3 and in Book 9, Chapter 6, Section 4 as summarised in Table 9.6.7.

Table 9.6.7. ARR 2016 Rainfall Loss Parameters for Urban Areas

Urban Area	Storm Initial Loss (mm)	Continuing Loss (mm/hr)
Effective Impervious Area	1 – 2 mm	0
Indirectly Connected Area	60 to 80% of rural catchment losses	For south eastern Australia, a typical value of 2.5mm/h, with a range of 1 to 3 mm/h, would be appropriate. This value should be adjusted based on engineering judgement and reviewing the catchment characteristics such as soil types, interaction of indirectly connected impervious areas with pervious areas. For other areas, adopt a range of 1 to 4 mm/h.
Urban Pervious Area	Traditionally, designers have adopted similar loss values for these areas as for those they would adopt in rural areas.	

In event based modelling approaches, it is important to subtract pre-burst rainfall from local losses associated with impervious and pervious surfaces as follows:

Burst initial loss = Storm initial losses – Pre-burst rainfall (for Burst initialloss ≥ 0)

For example, the burst initial loss for effective impervious area is 1.5 – 1.1 = 0.4 mm. The adopted burst losses for the urban surfaces are presented in Table 9.6.8. Note that in a situation where pre-burst rainfall is greater than the storm initial losses, the residual pre-burst rainfall should be included in the analysis.

Table 9.6.8. Adopted ARR 2016 Rainfall Loss Parameters

Urban Surface	Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Effective Impervious Area	0.4	0
Indirectly Connected Area	16.1	1.6

Urban Surface	Burst Initial Loss (mm)	Continuing Loss (mm/hr)
Urban Pervious Area	26.9	1.6

Hydraulic and associated flood behaviour is influenced by the hydraulic resistance due to topography and urban form. The selection of appropriate roughness coefficients is critical to the success of this approach (see Book 6, Chapter 4). Depth varying Manning's "n" roughness parameters were selected for each land use to account for shallow overland flow depths across urban surfaces. Some hydraulic modelling packages provide this capability in accordance with emerging research into depth varying roughness (for example, Zahidi et al. (2017), Khrapov et al. (2015), Muglera et al. (2011)).

Analysis of the performance of urban conveyance networks is critically dependent on potential blockage of inlet structures (Book 6, Chapter 6; Book 9, Chapter 5, Section 5) and the need to address safety design criteria (see Book 6, Chapter 7; Book 9, Chapter 5, Section 3). Assessment of potential blockage of inlet structures should also consider data from local authorities about maintenance programs and local flooding (Weeks et al., 2013). The assumed blockage factors for inlet pits subject to runoff from 1% AEP rainfall events were derived from Book 9, Chapter 5, Section 5, from local government historical records, and maintenance programs (see Table 6.6.1), from Weeks et al. (2013) and are presented in Table 9.6.9.

Table 9.6.9. Assumed Capacity of Inlet Pits for 1% AEP Rain Events

Sag Inlet Pit	
Kerb Inlet	80%
Grated Inlet	50%
Combination	Assume Grate 100% blocked
On-grade Inlet Pit	
Kerb Inlet	80%
Grated Inlet	60%
Combination	90%

The critical rainfall duration for the catchment was derived using the ensembles of rainfall temporal patterns in the combined hydrology and 2D hydraulic model to reveal the highest mean and median flood elevations at key locations as shown for in Figure 9.6.13.

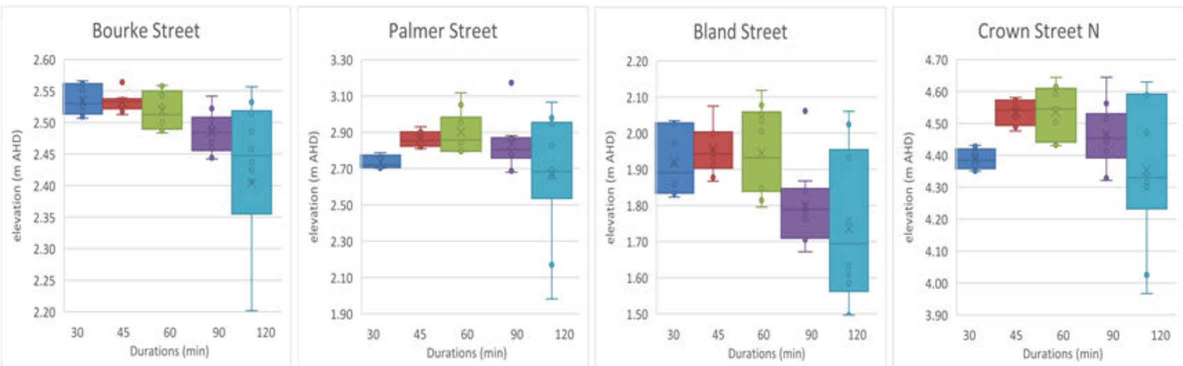

Figure 9.6.13. Use of Ensembles of Storm Bursts (1% AEP) in the Hydraulic Model to Select Critical Duration

Figure 9.6.13 reveals that the use of ensembles of rainfall in the hydraulic model indicates that different critical rainfall durations apply throughout the catchment. The results from Figure 9.6.13 were used with consideration of the characteristics of the catchment to select the critical storm duration of 60 minutes. The impact on stormwater runoff from using the single storm burst pattern from ARR 1987 is compared to use of an ensemble of ten storm burst patterns (1% AEP) from this guideline for part of the Wooloomooloo catchment in Figure 9.6.14. This graph presents ten hydrographs of stormwater runoff in the trunk drainage system at Bourke Street confluence.

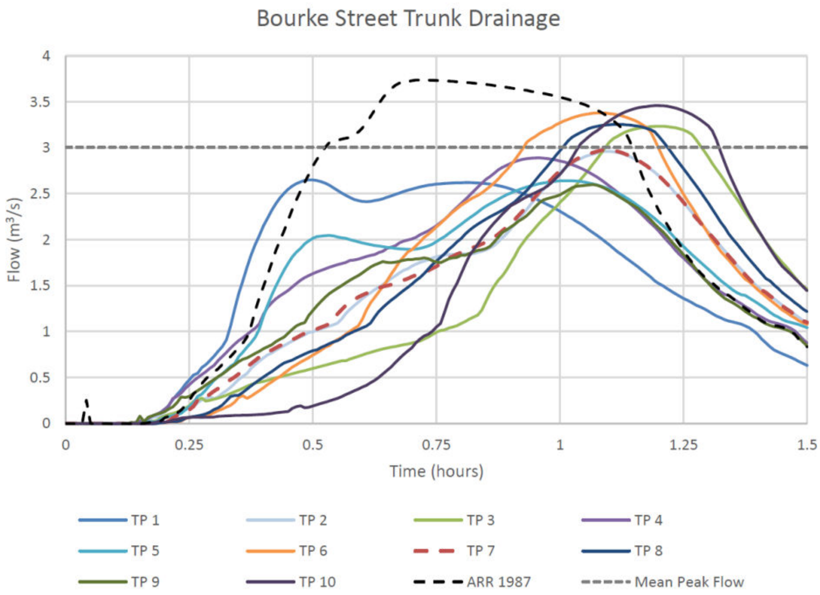

Figure 9.6.14. Example of Runoff from ARR 1987 Single Storm Burst and Ensembles of Storm Bursts from this guideline (1% AEP)

Figure 9.6.14 demonstrates that a single pattern of burst rainfall from ARR 1987 produces a different hydrograph shape, volume and peak runoff at the catchment outlet to the ensemble of storm burst patterns from this guideline. This difference is driven by the 30 years of additional data and science available to this guideline that has allowed the derivations of more spatially relevant rainfall and temporal patterns. The variability of the equally likely storm burst patterns from the ARR ensembles facilitates testing of catchment characteristics for generation of maximum runoff.

The direct rainfall method applies rainfall directly to all grid cells and the scale of routing is at every 2 m by 2 m grid cell. In this approach the depth of flow is shallow and rainfall can get stuck on the model grid. To maintain the area of rainfall applied to the grid, the buildings were nulled (removed) from the actual grid and rainfall was scaled up to account for the lost building areas.

The concentrated direct rainfall method applied rainfall to polygons of different local surfaces such as buildings and parks. This process permits the specification of the area, initial and continuing losses that are applied to each land use polygon. Separate attributes are applied to roofs to account for the different connectivity to concentrated stormwater flows.

A manual volume check should be undertaken on all direct rainfall model configurations. The volume of water leaving the model through the downstream boundary should be equal to the amount of water that was applied (via direct rainfall and inflows across external boundaries), less losses and storages within the model. The upper portion of the catchment (area of 52.8

Ha) was assessed to maximize the volume of water that drains from the catchment at the last time step. The characteristics of the upper catchment are shown in Table 9.6.10.

Table 9.6.10. Characteristics of the Upper Catchment used in the Volume Check

Type	Catchment Area (m^2)	IL (mm)	CL (mm/hr)
100% Pervious	22,508	26.9	1.6
100% Impervious	102,607	0.4	0.0
EIA	133,909	0.4	0.0
ICIA	34,549	16.14	1.6
ICIA (Buildings)	234,630	16.14	1.6
AVERAGE LOSS		**11.4**	**0.9**
TOTAL (m^2)	**528,202**		

The model run was extended to allow all stormwater to drain from the catchment by extrapolating the outflow curve towards zero. Inflow volume was calculated as the cumulative depth of rainfall less initial and continuing losses multiplied by the area of the catchment. Flows extracted from the hydraulic model 1D results can also be converted into volumes. A flow line along the upstream catchment divide together with outflow boundaries were used in the 2D hydraulic model to also account for the volume of overland flows leaving the catchment. These results can be presented as a cumulative depth graph or as a pie chart (refer to Figure 9.6.15)

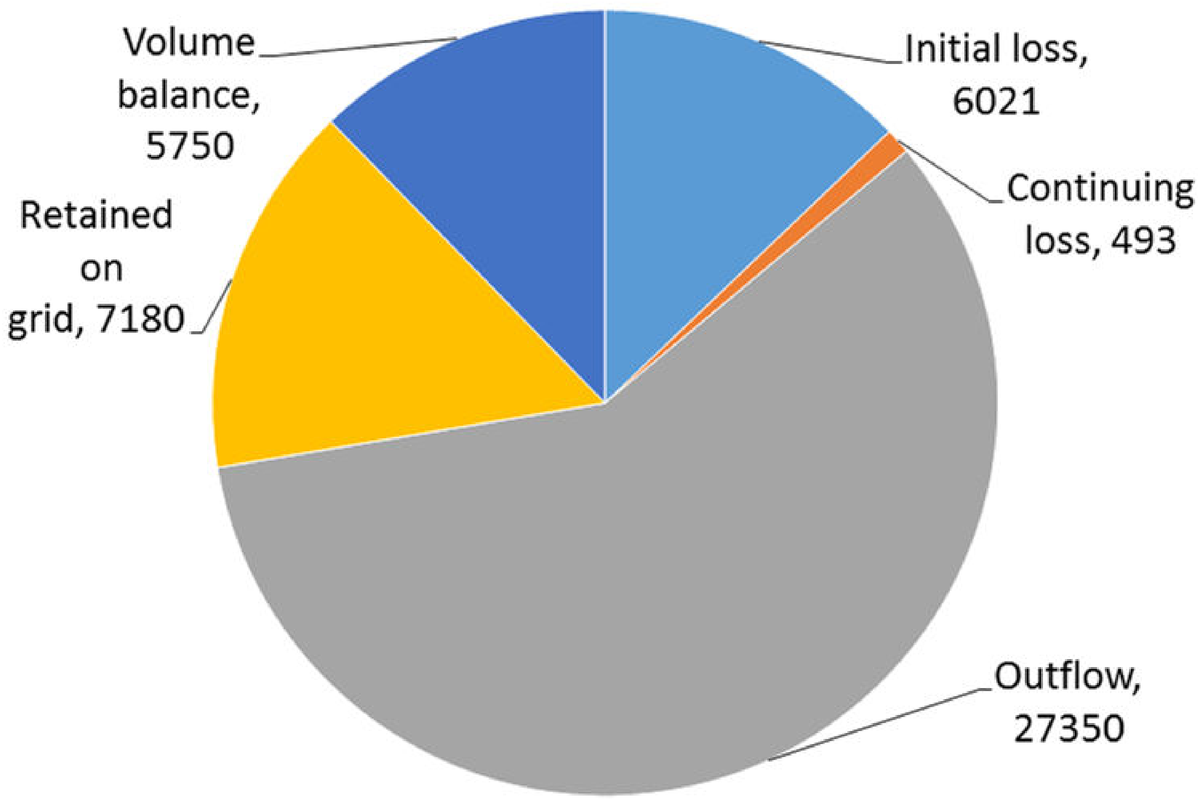

Figure 9.6.15. Upper Catchment Volume Check for Direct Rainfall Model (Prior to Corrections)

Figure 9.6.15 shows 5,750 m^3 (14%) of rainfall was retained in the model (11 mm) which is described as the volume balance. An acceptable error or additional retention of stormwater is less than 5% which indicates a need to reduce initial losses used in the direct rain model. Accounting for volumes of depression storage in the catchment topography by decreasing initial rainfall losses will increase in overall pipe and overland outflows. These results indicate that the catchment topography includes depression storages that capture 23.1 mm of rainfall. The results from coupled hydrology and 1D/2D hydraulic model with traditional loss assumptions revealed rainfall losses of 24.3 mm. The concentrated direct rainfall and direct rainfall methods can also be evaluated using sensitivity testing of initial conditions as follows:

- No accounting for rainfall lost to depression storage;
- Accounting for depression storage loss by reducing the initial loss. Apply direct rainfall with initial loss, less the average depth on grid;
- Accounting for depression storage using a restart file, which reapplied the conditions from the last time step to the model. Direct rainfall applied with the initial conditions adopted from the final time step of the initial simulation.

The outflow depths in the standard direct rainfall simulations changed from 52 mm to 61 mm by using a restart file and in the standard direct rainfall simulations changed from 52 mm to 56 mm by reducing the assumed initial losses in the models.

The hydrograph outputs of overland flows at selected locations (refer to Figure 9.6.12) at Riley Street near the park (top pane) and at Crown Street North (bottom pane) are shown in Figure 9.6.16. It is clear that overland flow is under-represented in the uncorrected direct rainfall models as compared to traditional coupled 1D/2D models.

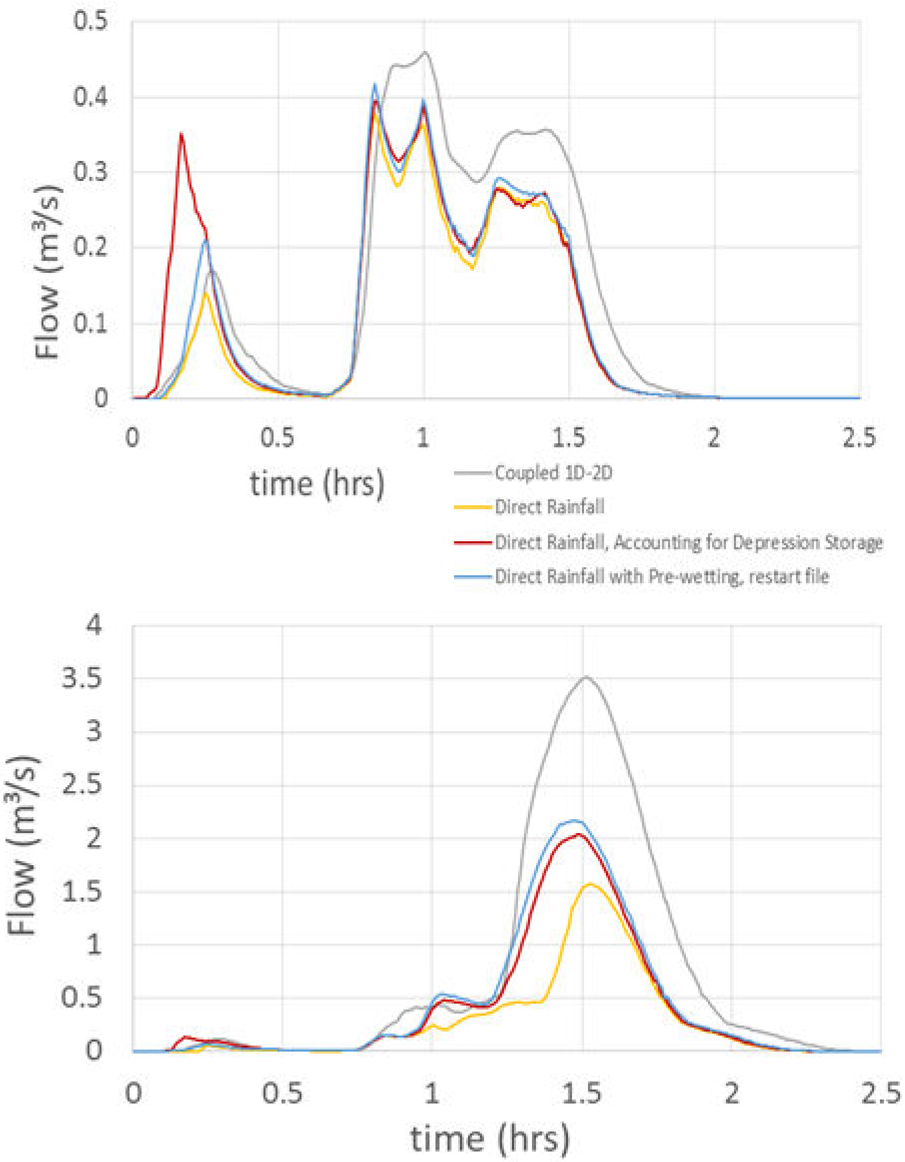

Figure 9.6.16. Comparison of Treatment of Initial Conditions in Overland Flows Generated by Coupled Direct Rainfall Models Near the Top of the Catchment (Top Pane: Riley Street) and Near the Bottom of the Catchment

Figure 9.6.16 demonstrates that the uncorrected direct rainfall models produce variable under-estimation of surface flows, as compared to a traditional coupled 1D/2D model, that is dependent on location and attributes of sub-catchments. Techniques that account for depression storage or pre-wetting of the catchment surfaces using restart files can improve the comparative performance of direct rainfall models. However, the residual differences in surface flows highlight that 2D models and in particular direct rainfall models should also be verified using historical records of local flood depths. Surface flows are a significant proportion of the responses from urban catchment as shown in Figure 9.6.17.

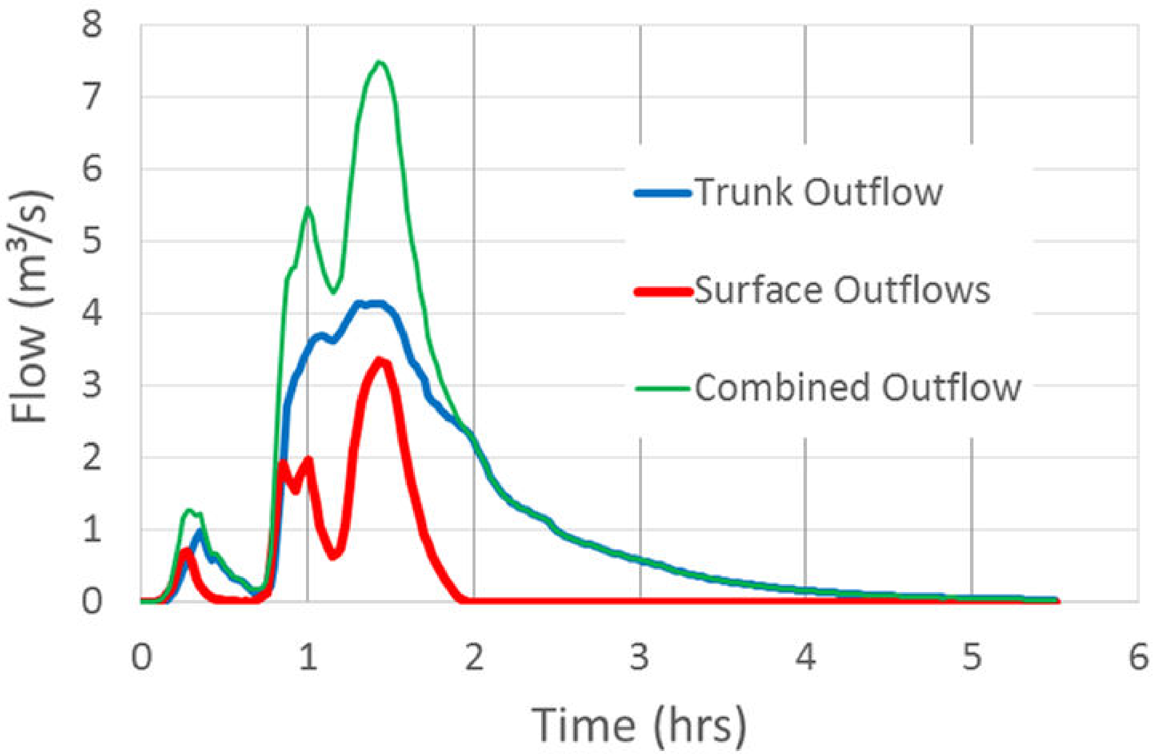

Figure 9.6.17. Outflow Hydrographs Catchment Showing the Significance of Surface Flows

This case study demonstrates practical application of the ARR ensemble temporal patterns on an urban catchment that is dominated by overland flow. The pattern that best represents the mean response has been selected based on flood elevation rather than flow. It is clear from the results of this analysis that a volume check of direct rainfall approaches should be undertaken in accordance with recommendations of Babister and Barton (2012) and the results should be verified using historical records of spatial flooding. A significant amount the rainfall excess is not generating runoff because rainfall is trapped on the terrain grid. This trapped rainfall excess represents an effective overestimation of the catchment loses with associated underestimation of surface flow and should be factored into the losses so that the correct amount of rainfall excess is generated. This can be carried out by either pre-wetting parts of the catchment or adjusting the assumed initial losses or a combination of both.

6.4.6. Downstream

Outflows from urban sub-catchments and conveyance networks interact with regional storage controls and water quality measures (refer to Book 9, Chapter 4 and Book 9, Chapter 5), discharge to urban waterways (See Book 9, Chapter 2and Book 9, Chapter 3) and to receiving waters such as estuaries, rivers, bays and oceans. The methods outlined in Book 6, Chapter 5 may need to be applied to interactions of rainfall and storm surge processes in estuaries, bays and oceans to account for combined impacts on urban flooding.

The complexity of urban areas also fosters the need to consider the joint probability of the different factors such intersection of urban runoff with regional flows in rivers or water levels in regional storages and water quality measures, which may be correlated or independent of each other. Methods to account for joint probability are provided in Book 4, Chapter 4. The urban designer should also consider climate change impacts on urban flooding as outlined in Book 8, Chapter 7, Section 7 and Book 1, Chapter 6.

The connectivity between design of urban conveyance and a volume management facility, setting the rural base case for design targets, application of climate change and assessment of downstream impacts on a sensitive waterways is combined in a greenfield example (Coombes and Barry, 2018). This conceptual design example is located near Ballarat in Victoria and includes an objective of no increase in peak flows in the downstream natural waterway to mitigate impacts of the urban development on erosion of the stream. The pre-development catchment is shown in Figure 9.6.18 and the proposed development is presented in Figure 9.6.19.

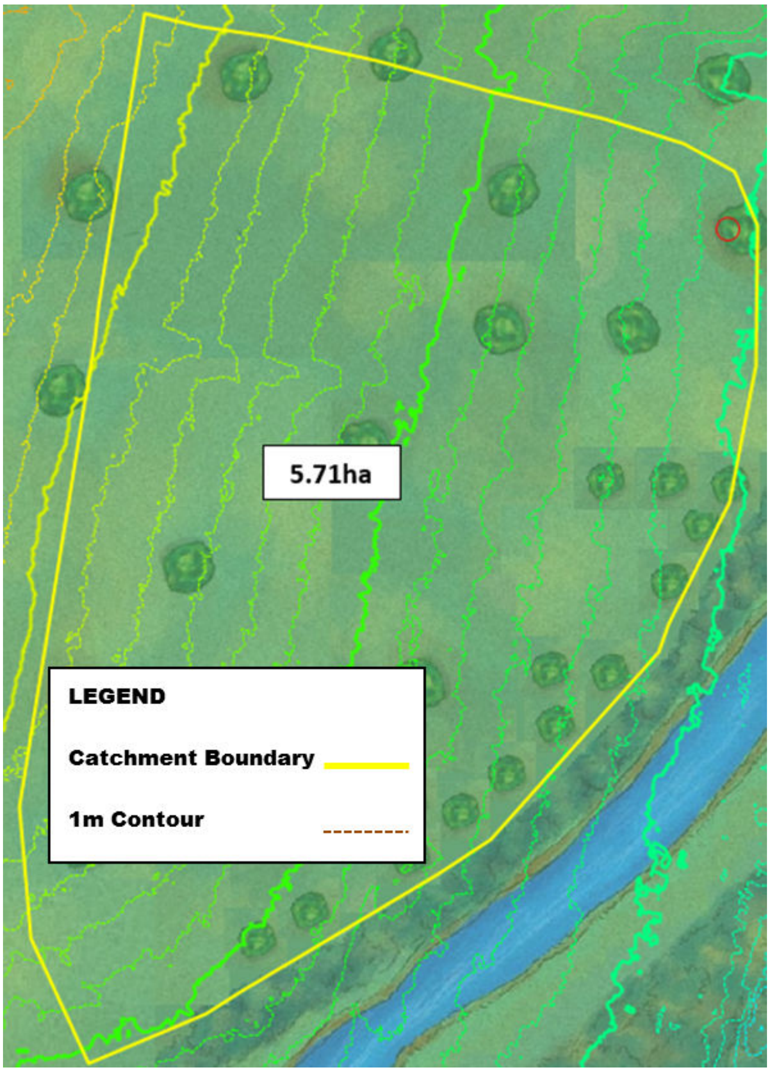

Figure 9.6.18. Catchment prior to development

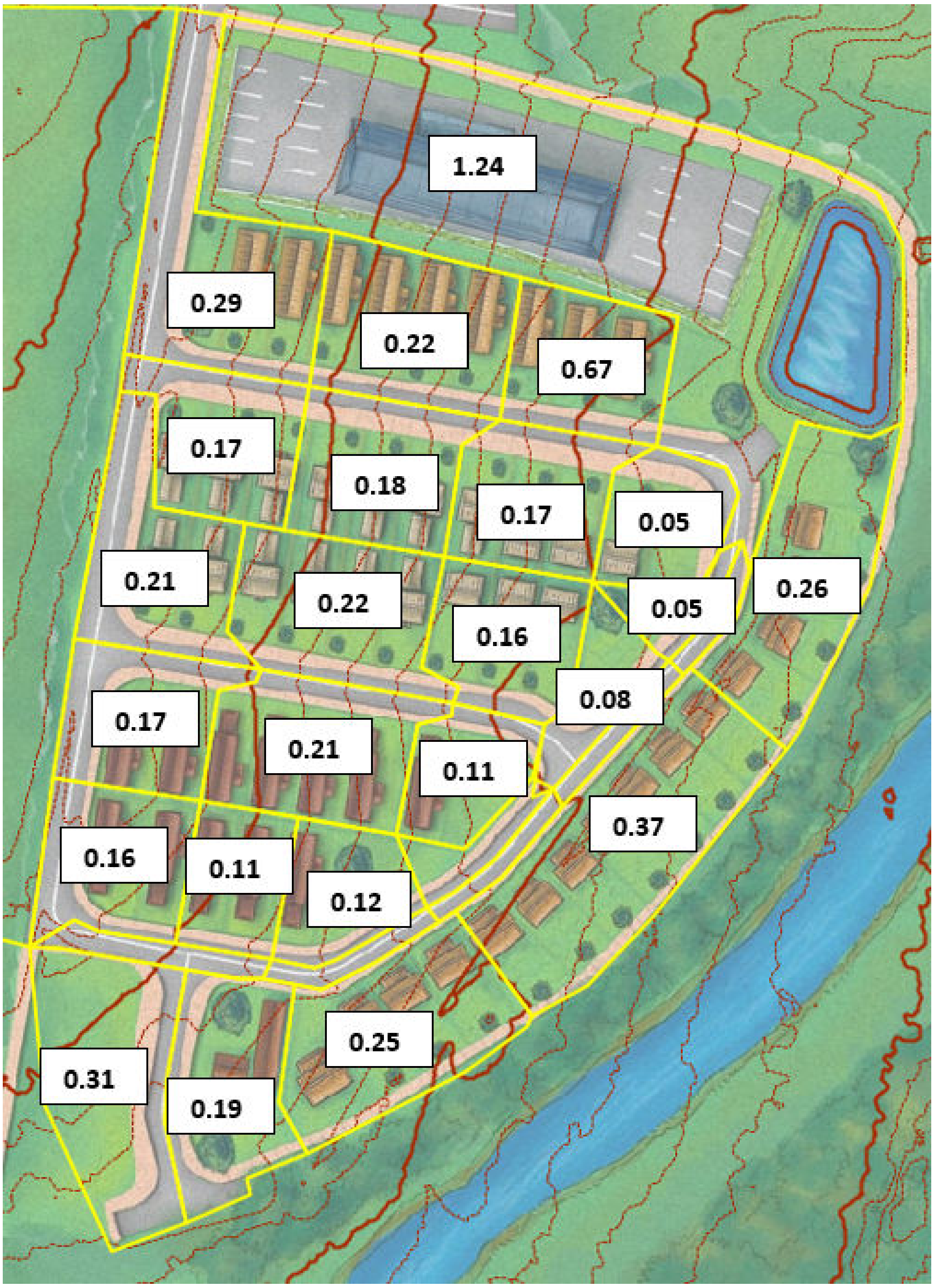

Figure 9.6.19. Developed Catchment

An estimate of pre-development peak flows was required to set the design peak flow targets for the proposed urban development. The Regional Flood Frequency Estimation Model (RFFE) available from http://rffe.arr-software.org/ was utilised to estimate rural peak flows with uncertainty as shown in Figure 9.6.20 which is based on gauged flows from multiple regional gauges (Figure 9.6.21). The use and limitations of the RFFE is described in Book 3, Chapter 3. Whilst the example catchment size is less than the currently recommended minimum and the RFFE is subject to improvement, this process provides a good starting point for defining the rural flow target. The rural flows from the RFFE might also be combined with statistical analysis of observed flows in a nearby catchment using FLIKE (refer to Book 3, Chapter 2, Section 8) to improve regional flow estimates. These improved regional peak flow results from the nearby catchment can be used to calibrate a hydrology model and the parameters transferred to the design catchment as explained by Coombes et al. (2016). Patil and Stieglitz (2012), for example, outline methods of transferring parameters from gauged catchments to ungauged catchments.

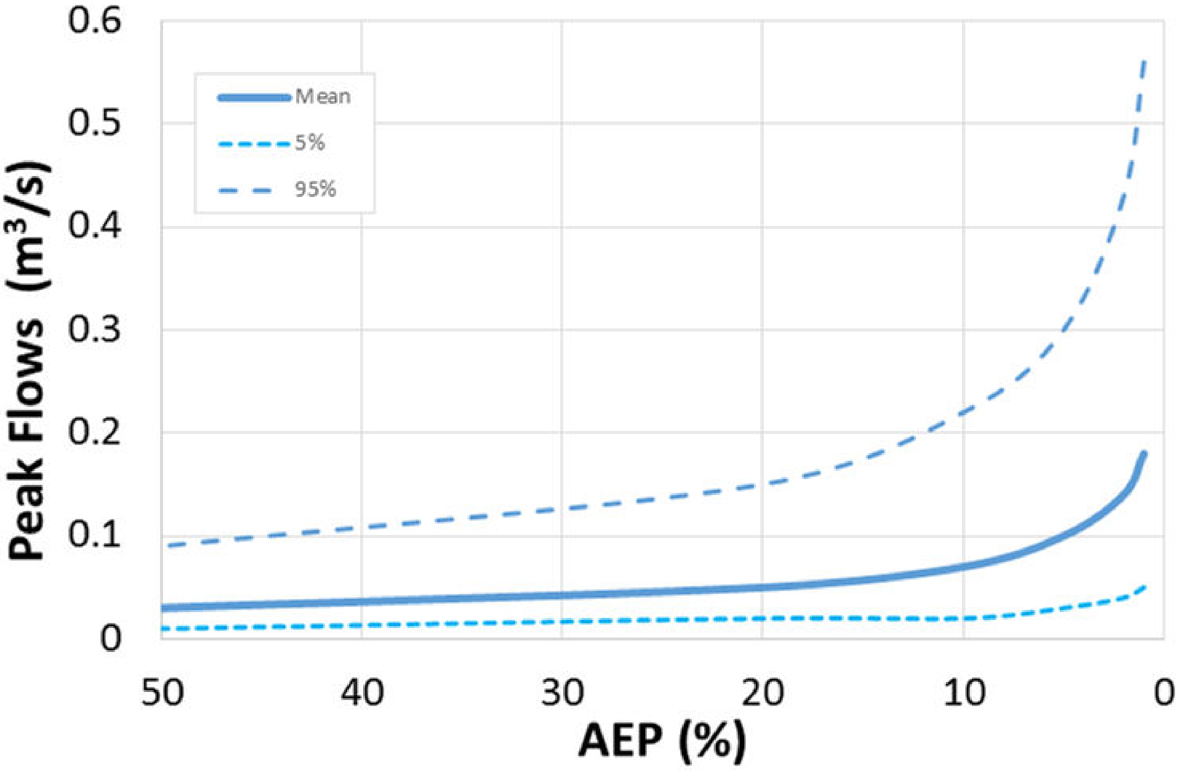

Figure 9.6.20. Estimated Rural Peak Flows using the RFFE

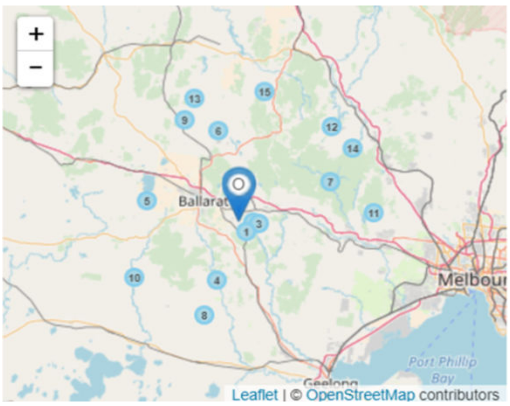

Figure 9.6.21. Regional Flow Gauges used in the RFFE Estimate of Rural Peak Flows

The next step in the design process involved selecting the project location in the ARR Data Hub and downloading hydrology and rainfall information, including local design rainfall IFD and ensembles of temporal patterns. Most proprietary models will download this information and set up the ensembles of rainfall inputs. Estimated regional rural losses for initial losses (IL) of 25 mm and continuing losses (CL) of 4.3 mm/hr were also downloaded from the ARR Data Hub.

A model with combined hydrology and hydraulic capacity was used with initial estimates of IL = 25 mm and CL = 4.3 mm/hr, design burst rainfall ensembles and pre-burst rainfall to estimate local rural losses that were calibrated to rural flows sourced from the RFFE as shown in Figure 9.6.22. The critical duration was found to be 1.5 hours as defined by highest mean peak flows for 50%, 10% and 1% AEP events as shown in Figure 9.6.23, Figure 9.6.24 and Figure 9.6.25. Median pre-burst rainfall for 90 minute storm durations were also selected from the ARR Data Hub for 50% AEP: 4.1 mm; 10% AEP: 3.3 mm and 1% AEP: 1.1 mm. The pre-burst rainfall was included in the hydrology model and spread over the hour prior to burst rainfall and the calibration processes aimed to find values of IL and CL that produced simulated rural peak flows that were similar to RFFE peak flows for the 10% AEP events. This process enabled an estimate of local rural initial losses of 16 mm and continuing loss of 5 mm/hr for an assumed Mannings roughness coefficient (n = 0.075).

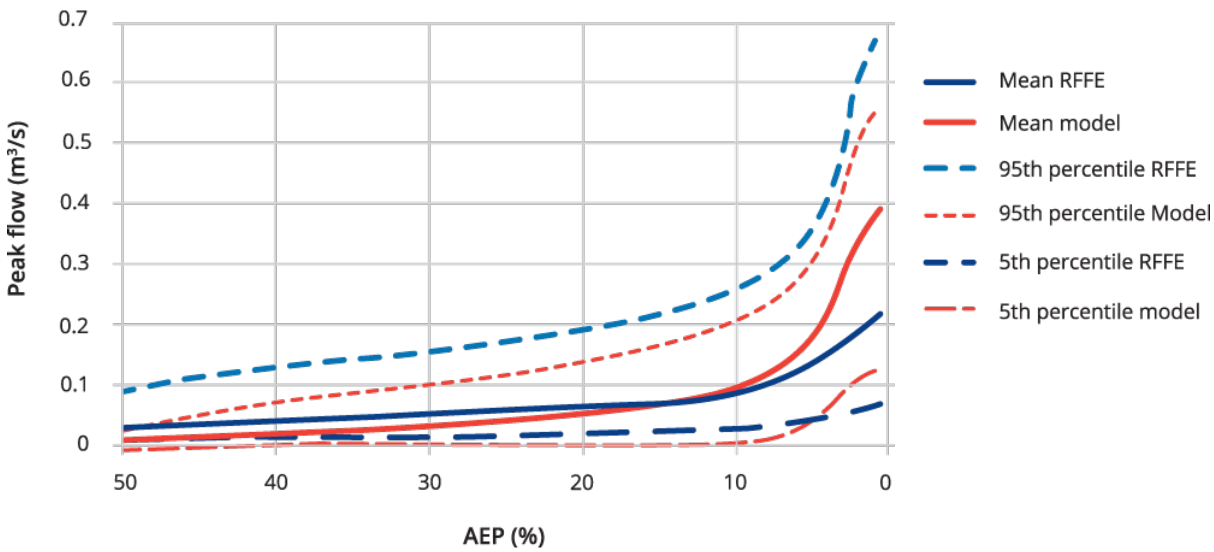

Figure 9.6.22. Calibration of Rural Flows to RFFE Flow Estimates

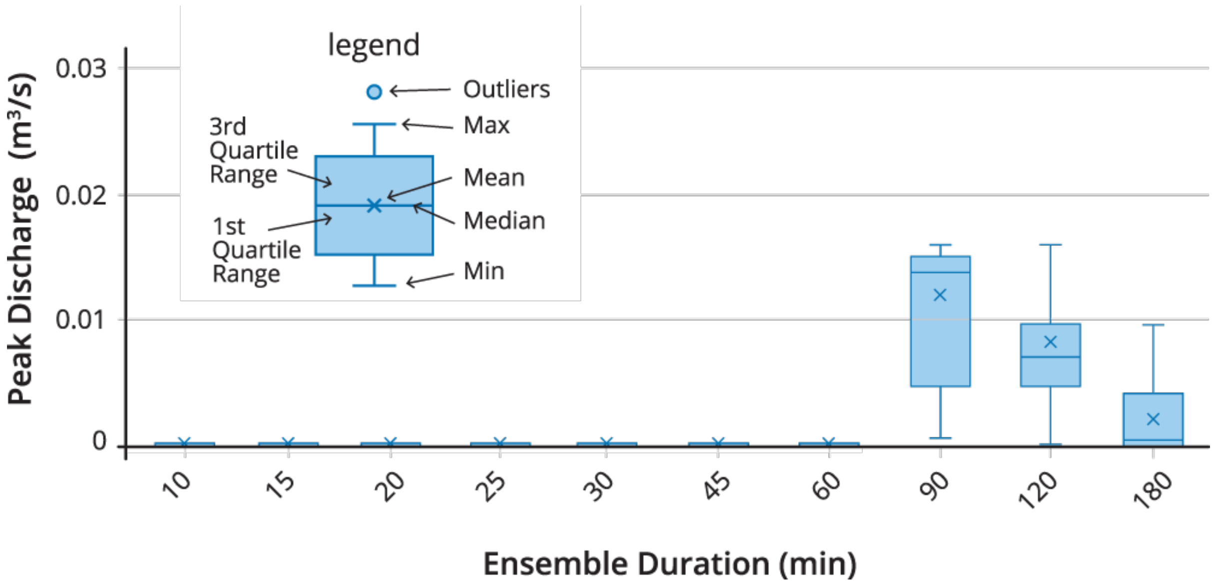

Figure 9.6.23. Pre-Development Peaks Flows for 50% AEP Events

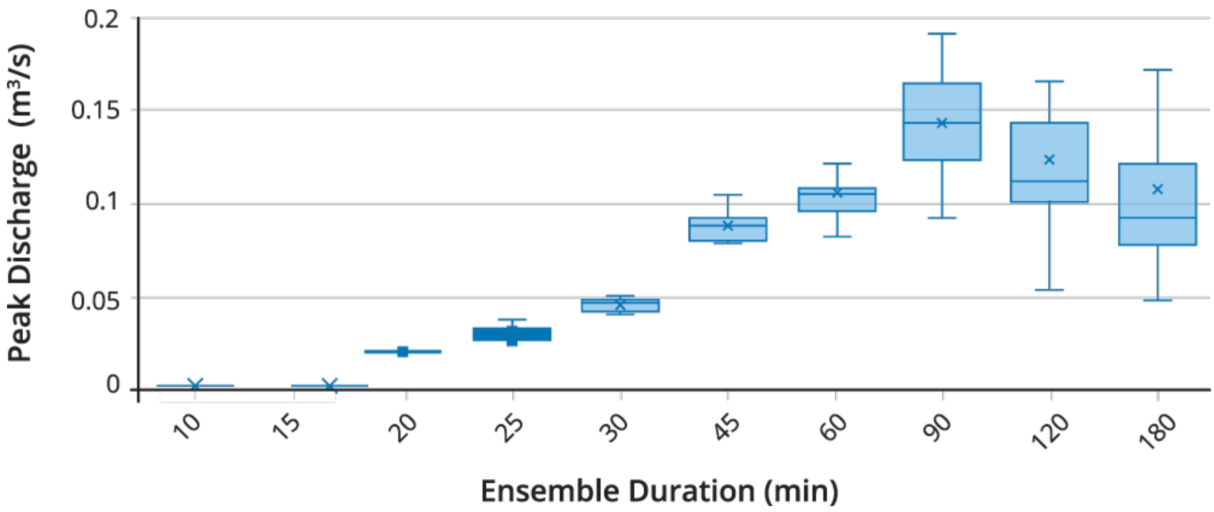

Figure 9.6.24. Pre-Development Peak Flows for 10% AEP Events

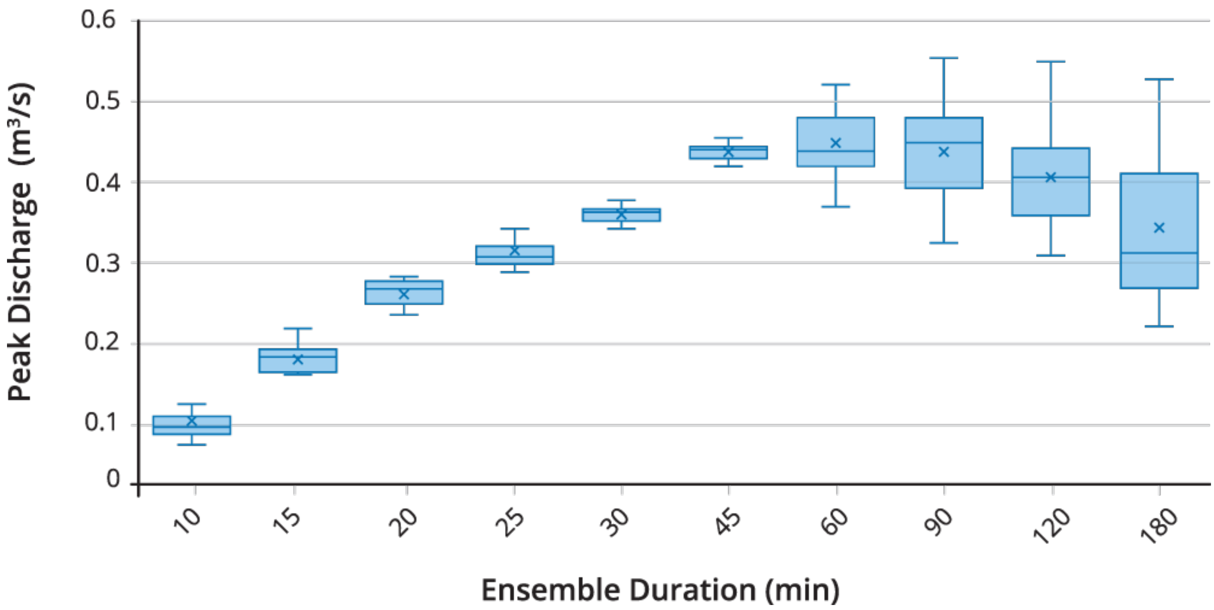

Figure 9.6.25. Pre-Development Peak flows for 1% AEP Events

The mean maximum pre-development peak flows for the 50%, 10% and 1% AEP were found to be 0.011 m^3/s, 0.14 m^3/s and 0.45 m^3/s respectively. These values were used as the peak flow targets for the urban development. The altered land surfaces (impervious and pervious areas of roads and properties) associated with the urban development was included in the hydrology model. The loss values for the urban catchment from Book 5, Chapter 3 and Book 9, Chapter 6, Section 4 were assigned as follows:

- Effective Impervious Area: IL =1.5 mm, CL = 0 mm/hr
- Pervious Area = rural losses

Indirectly connected impervious area assumptions were not required because the spatial detail of land uses with associated connectivity were included in the hydrology/hydraulics model. The hydrology of the urban catchment was simulated for all design rainfall ensembles to determine a critical duration of 10 minutes for the 10% AEP flows relevant to the design of pit and pipe conveyance infrastructure (refer to Book 9, Chapter 5). These simulations were completed prior to design of infrastructure to determine the relevant critical duration and design storm for use in the design process. Pre-burst rainfall for a one hour duration was selected from the ARR Data Hub (50% AEP: 2.2 mm, 10% AEP: 2.2 mm, 1% AEP: 0.8 mm) for use with the 10 minute duration design rainfall ensembles relevant to the design of the pit and pipe conveyance infrastructure. The pre-burst rainfall was distributed across an hour prior to the burst rainfall.

The hydrographs from the simulation using ensembles of 10% AEP design burst rainfall with pre-burst rainfall was examined to select the design storm closest to mean peak flow for design of conveyance infrastructure as shown in Figure 9.6.26. Urban peak flows from all design rainfall durations for 50%, 10% and 1% AEP events are presented in Figure 9.6.27, Figure 9.6.28 and Figure 9.6.29 respectively.

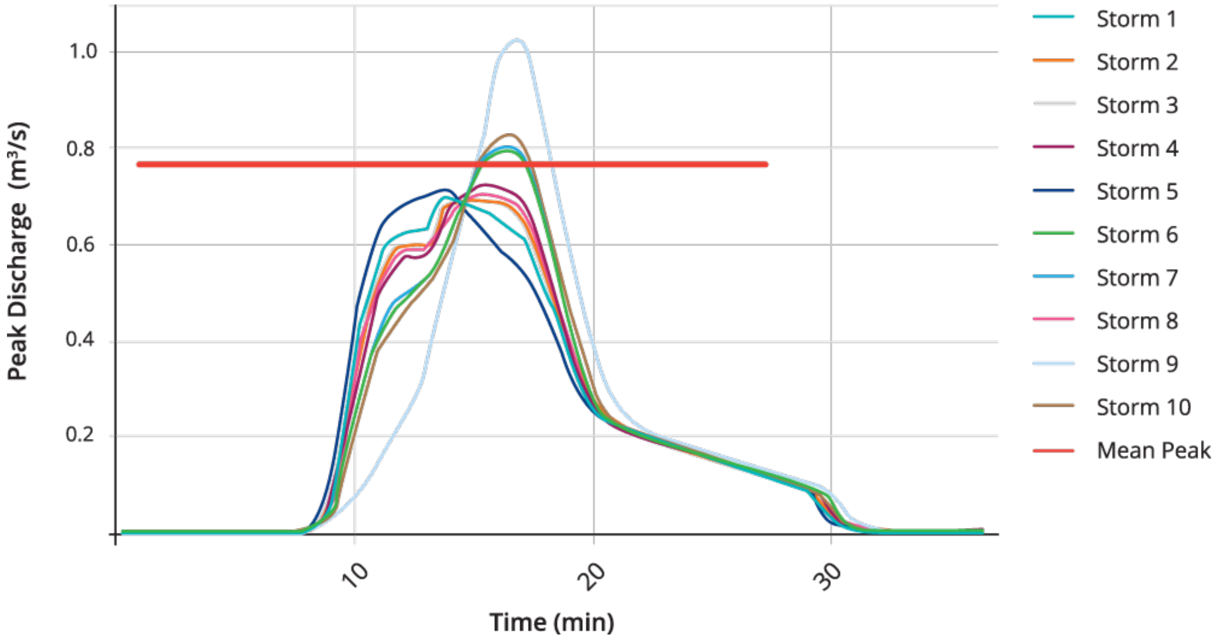

Figure 9.6.26. Selection of the 10% AEP Design Storm for Preliminary Infrastructure Design

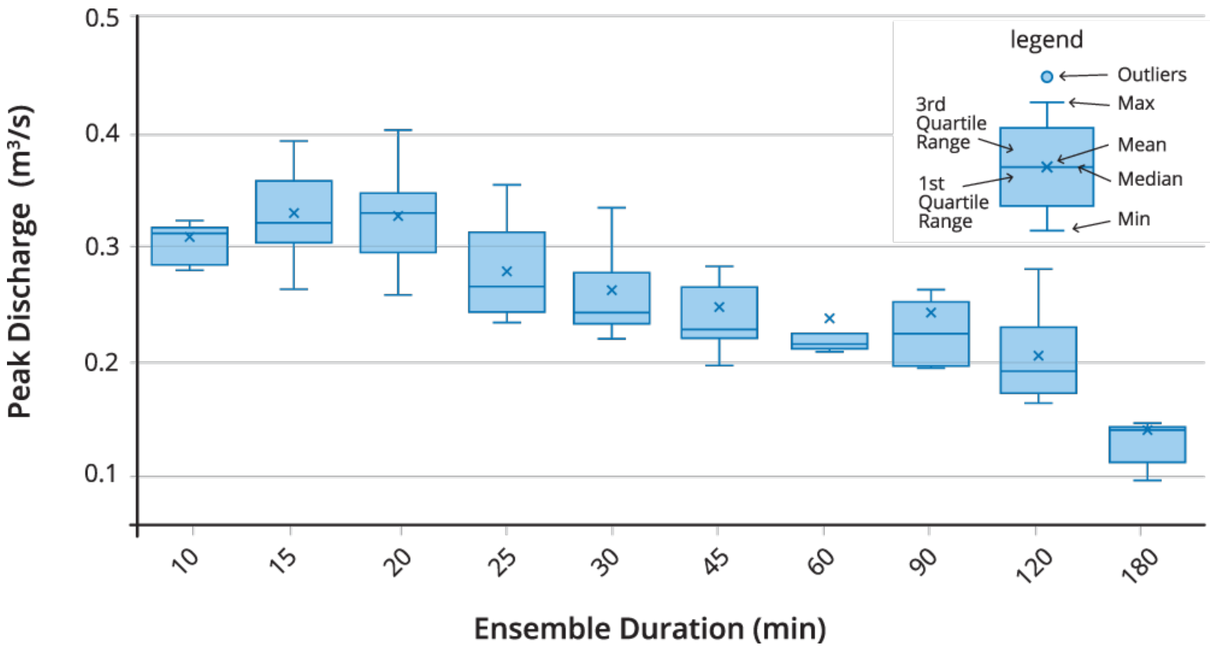

Figure 9.6.27. Development Peak Flows for 50% AEP Events

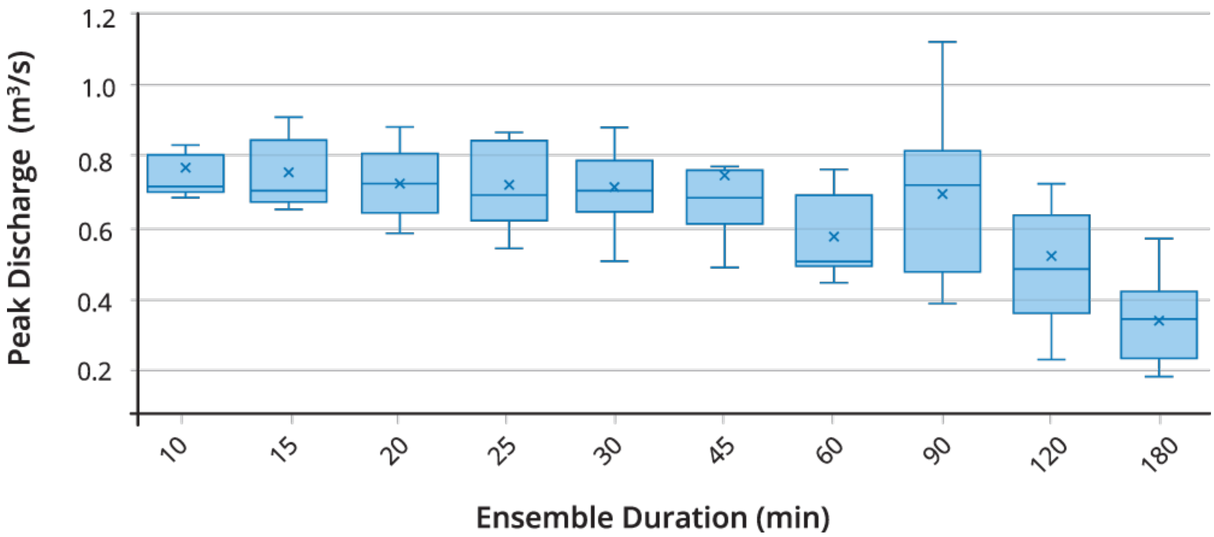

Figure 9.6.28. Development Peaks Flows for 10% AEP Events

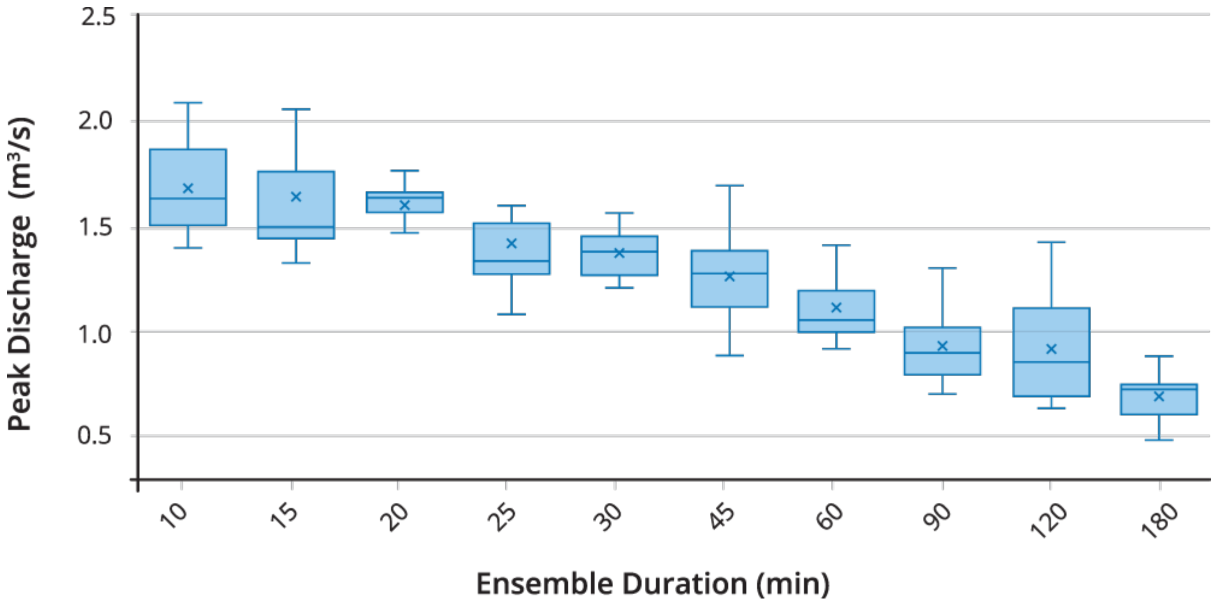

Figure 9.6.29. Development Peak Flows for 1% AEP Events

The preliminary design of the conveyance network (Book 9, Chapter 5) shown in Figure 9.6.30 was sized using storm 7 (Figure 9.6.26) for the 10% AEP event with a design pre-burst rainfall of 2.2 mm. Inlet pit relationships from Book 9, Chapter 5, Section 5 were applied to the design process. Pit inlet capacities for on grade pits (Figure 9.5.12) and sag pits (Equation (9.5.1) to Equation (9.5.4)) were derived using Book 9, Chapter 5, Section 5. Design blockage of on grade and sag pits was derived from Table 9.5.2 and pit energy losses were defined using Book 9, Chapter 5, Section 5.

A hydrology/hydraulics model was use to sized pipes in the conveyance network with objectives of maintaining 150 mm freeboard to grates of inlet pits and less than two metre flow width on roads. The design of the conveyance network was then checked using ensembles for 10% AEP design storm events with pre-burst rainfall for 10, 15, 20 and 30 minute durations.

The safety of surface flows were also checked by simulating the performance of the conveyance network using design rainfall ensembles for 1% AEP burst events with pre-burst rainfall for 10, 15, 20 and 30 minute durations. In accordance with Book 9, Chapter 3, Section 4 and Book 9, Chapter 5, Section 6 (also see Book 7, Chapter 6), the design aimed to limit surface water depths to less than 200 mm and less than 50 mm at road crowns. These objectives also included limiting depth velocity product to less than 0.4 and aimed for freeboard to floor levels of greater than 300 mm.

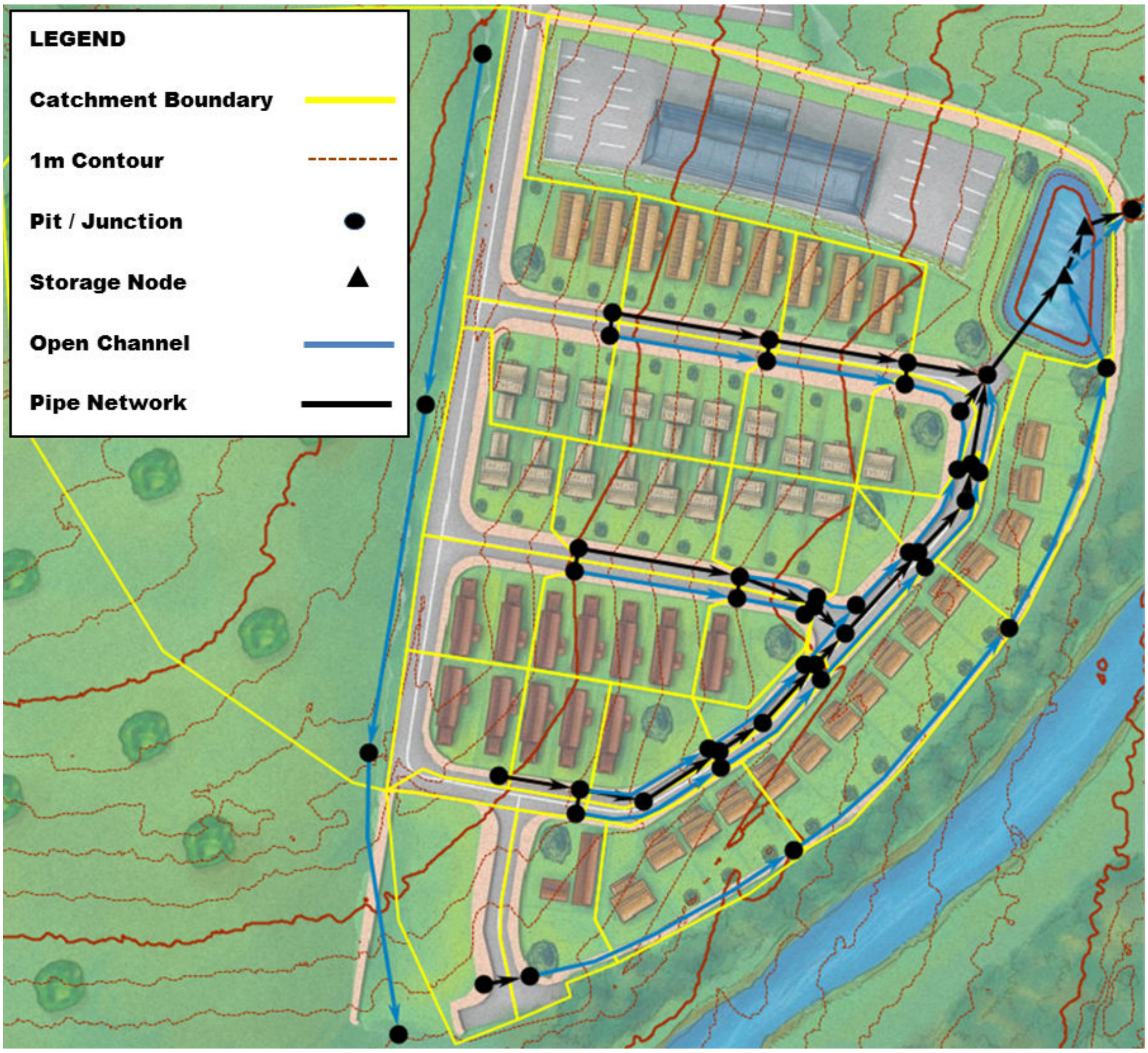

Figure 9.6.30. Overview of the Planned Conveyance Network in the Urban Development

A storage basin was then designed to manage flooding and impacts on downstream waterway (refer to Book 9, Chapter 4) by mitigating the 50%, 10%, 1% AEP peak flows to meet the rural target defined above. Storage volume and outflow arrangements were utilised to achieve this (refer to Figure 9.6.31). The design of the basin included a freeboard of 300 mm from 1% AEP maximum depth and an emergency spillway designed for full blockage of 1% AEP rainfall events (refer to Book 6, Chapter 6 for blockage discussions).

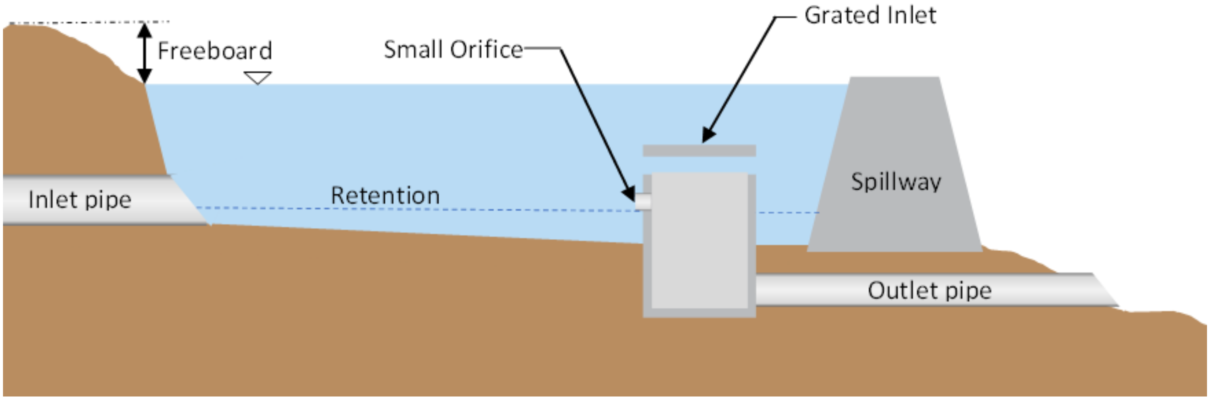

Figure 9.6.31. Overview of the Planned Storage Basin Below the Urban Development

A trial basin design was undertaken using the hydrology/hydraulics models and ensembles of 1.5 hour duration design rainfall with pre-burst rainfall. The design of the basin was then tested and modified using ensembles of design rainfall with pre-burst rainfall for all durations to ensure the rural peak flow targets were met and the maximum basin depth was not exceeded. The final results for peak flows discharging from the development via the basin are shown in Figure 9.6.32, Figure 9.6.33 and Figure 9.6.34 for 50%, 10% and 1% AEP rainfall events. Water levels in the basin for all 1% AEP rainfall durations are provided in Figure 9.6.35.

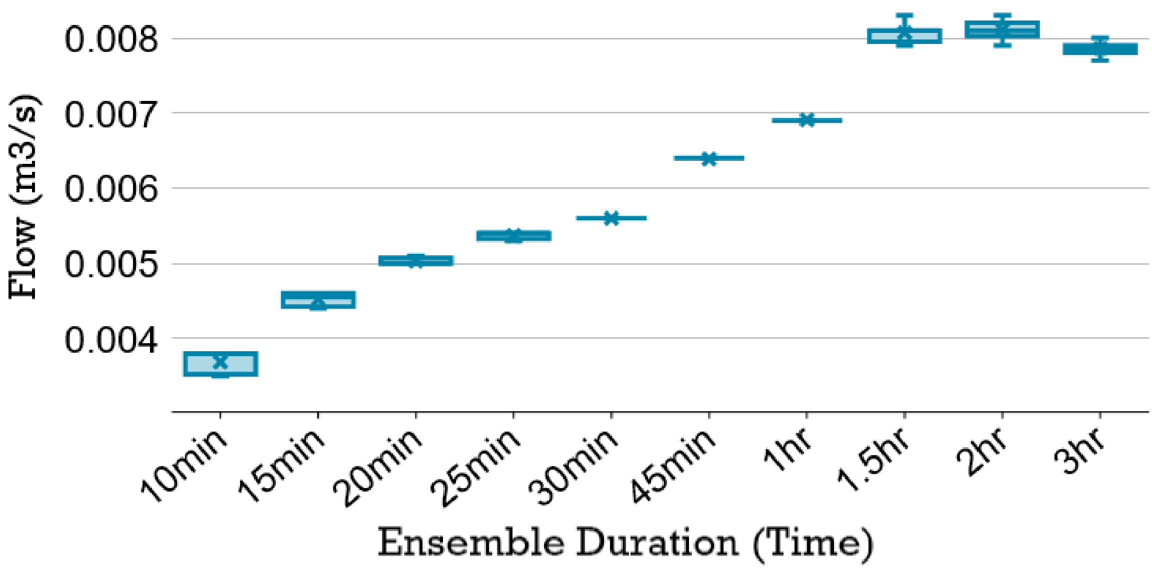

Figure 9.6.32. Peak flows from the Basin for 50% AEP Events

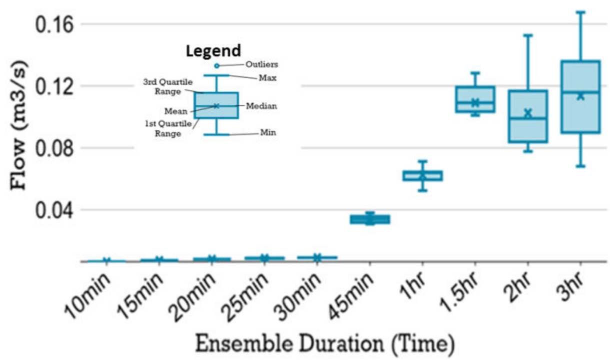

Figure 9.6.33. Peak Flows from the Basin for 10% AEP Events

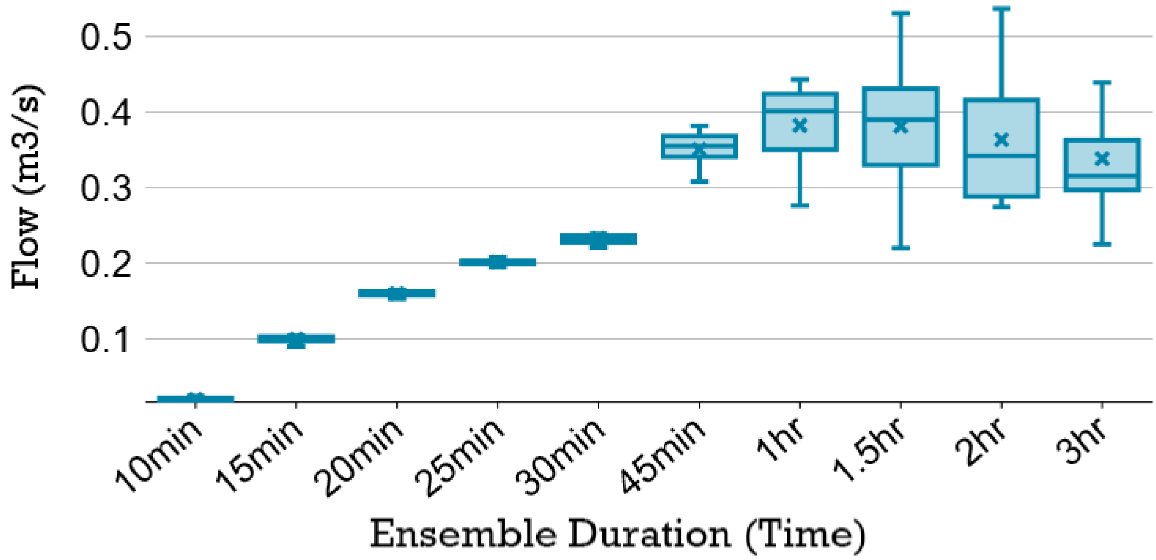

Figure 9.6.34. Peak Flows from the Basin for 1% AEP Events

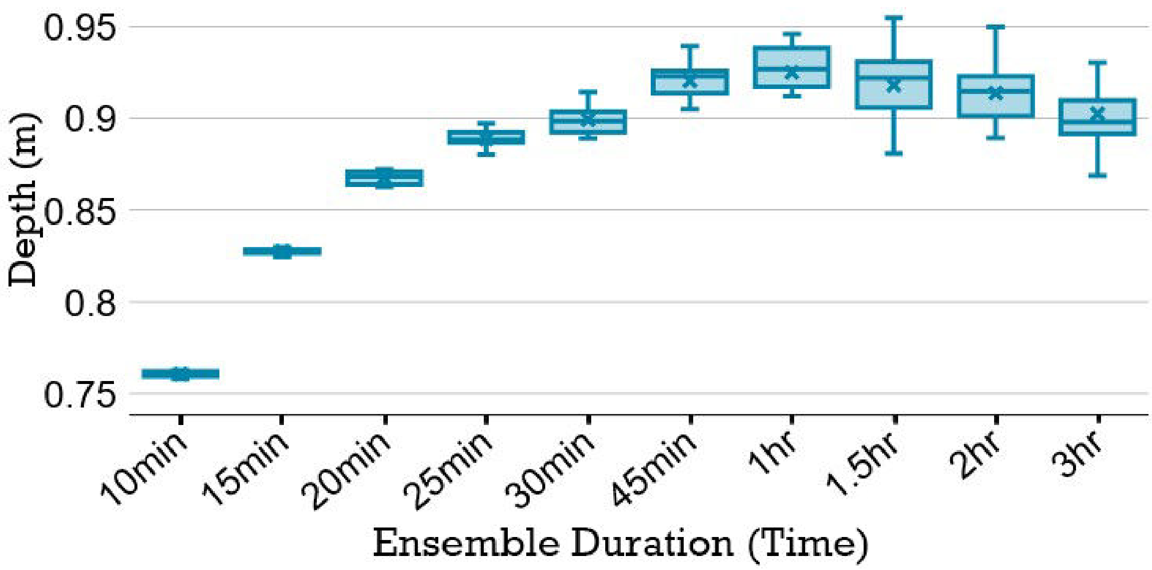

Figure 9.6.35. Peak Water Levels the Basin for 1% AEP Events

Figure 9.6.32 to Figure 9.6.34 show that the mean peak flows from the basin were less than the rural flows with critical durations ranging from one to three hours. A one hour critical duration of peak water levels in the basin was also observed from the analysis (refer to Figure 9.6.35). These result highlight that critical durations of stormwater runoff can vary throughout catchments and across different types of the infrastructure.

This example was developed using the methods in Book 1, Chapter 6 (2019). The methods have evolved since time of writing but the principles are the same[2]. A design life for the infrastructure and consequence level for climate change impacts was selected. A design life of 100 years was assumed for the basin with medium consequences of failure due to impacts on the waterway and surrounding rural properties.

This assessment was utilised to extract data from ARR Data Hub for the RCP 8.5 value for 2090 which indicated an expected 16.1% increase in peak rainfall[3]. This expected increase in peak flows was used to alter the increase in peak rainfall. This expected increase in peak flows was used to alter the increase in peak rainfall (Please note the Data hub value for Ballarat has changed as of May 2019 to 16.3% to reflect changes to the predicted temperatures from Climate Change Australia). This expected increase in peak flows was used to alter the relevant design rainfall ensembles and the hydrology/hydraulic model was rerun to test the impact of climate change on peak water levels in the basin and on roads. Designers should also utilise emerging research to incorporate that most up to date climate change assessments. For example, Wasko and Sharma (2015) outline greater potential for increased rainfall intensities in urban areas.

The impact of applying the expected 2090 climate change effects on design rainfall on peak water levels in the basin and at a critical location on the road is shown in Figure 9.6.36. Increases in peak water depths are experienced in the basin and on the road. The increased runoff into the basin is managed by the emergency spillway and peak water levels are

[2] This section was written before the latest climate change guidance in Book 1, Chapter 6 (2024). A minor change to the text has been made to reflect the change in guidance.

[3]Please note the Data hub value for Ballarat has changed as of May 2019 to 16.3%. This reflects changes to the predicted temperatures from Climate Change Australia

acceptable. However, peak water levels on the road exceed the design objectives and the designer should highlight this situation to the consent authority for further consideration.

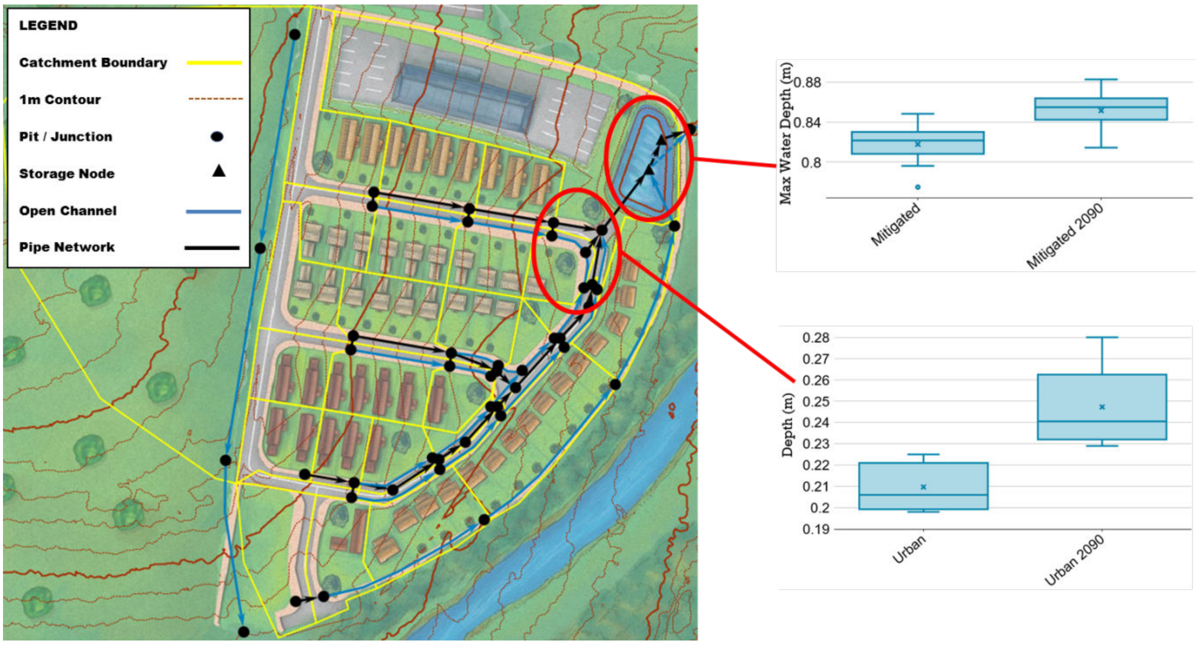

Figure 9.6.36. Peak Water Levels in the Basin and on Roads for 1% AEP Events Subject to Climate Change

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