Chapter 1
Introduction

Isabelle Testoni, Monique Retallick

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Examples	N/A
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1.1 Objectives and Scope

There is a need to provide guidance on issues related directly to urban catchments. The approach and constraints for urban catchments is significantly different to rural catchments in some aspects of flood hydrology. It is therefore necessary to outline specific approaches and philosophies applied to the urban environment in a separate book.

The considerations in managing runoff in urban catchments is varied and complex. Some considerations explored in this book are:

- How urbanisation effects catchment characteristics
- Drainage systems
- Overland flooding versus riverine (or channel) flooding
- Storage of runoff in detention and retention systems
- Design of pipe systems

This book also contains discussion on safety of people and vehicles. This equally applies to rural catchments.

As part of the ARR revision projects a search was undertaken to uncover long term streamflow gauges in urban areas. Insufficient data was uncovered to allow the development of an urban flood method. The existing urban streamflow gauges should be given special recognition for their importance to the development of future techniques. This recognition could be used to help justify the ongoing support and maintenance of these gauges.

It is highly desirable to identify a set of high quality urban catchments to allow new methods to be tested against observed data. Such catchments should have long term gauged records, good quality rating curves, and a reasonably stationary level of development.

While theory hydraulic structures is covered in Book 6, practical guidance on how it applies to an urban context is covered in this book. Although ARR aims to cover best practice in flood estimation, new methods, data and software is constantly being developed. Careful consideration should be taken in using the most up the date and appropriate methods.

Chapter 2
Aspects of Urban Hydrology

Tony Ladson

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2.1 The Urban Hydrologic Cycle

In both urban and non-urban situations, the starting point of hydrologic analysis is the water cycle. In rural areas, hydrologists are concerned with catchment inputs - mainly precipitation; outputs - evaporation and runoff, and the storage of water in the catchment. In urban catchments, the fundamental processes are the same but the results of development profoundly changes the catchment water balance (Figure 2.1):

- Inputs are increased because mains water is supplied to urban catchments along with rainfall.
- The water stored in the catchment changes. Much of the soil is paved over so there is less water infiltration into soil from rain. Drainage networks rapidly remove surface water. Imported water may contribute to groundwater storage if there is leakage from water supply and sewage pipes; or water may leak into pipes, or enter the gravel filled trenches surrounding pipes, depleting groundwater.
- The way water leaves a catchment changes. Runoff volumes are often substantially increased and are disposed of through hydraulically efficient networks. Wastewater systems provide an alternative flow path that can interact with groundwater. There may be less opportunity for water to evaporate if it has quickly drained from a catchment.

The change in the rate and volume of inputs, outputs and storage explains the hydrologic behaviour we see in urban areas: the rapid response to rainfall and increased flood magnitude and frequency that co-occur with development. This chapter explores aspects of urban hydrology, the impact of development and urban drainage systems, focussing on the areas where the effects of urbanisation needs to be considered in flood estimation.

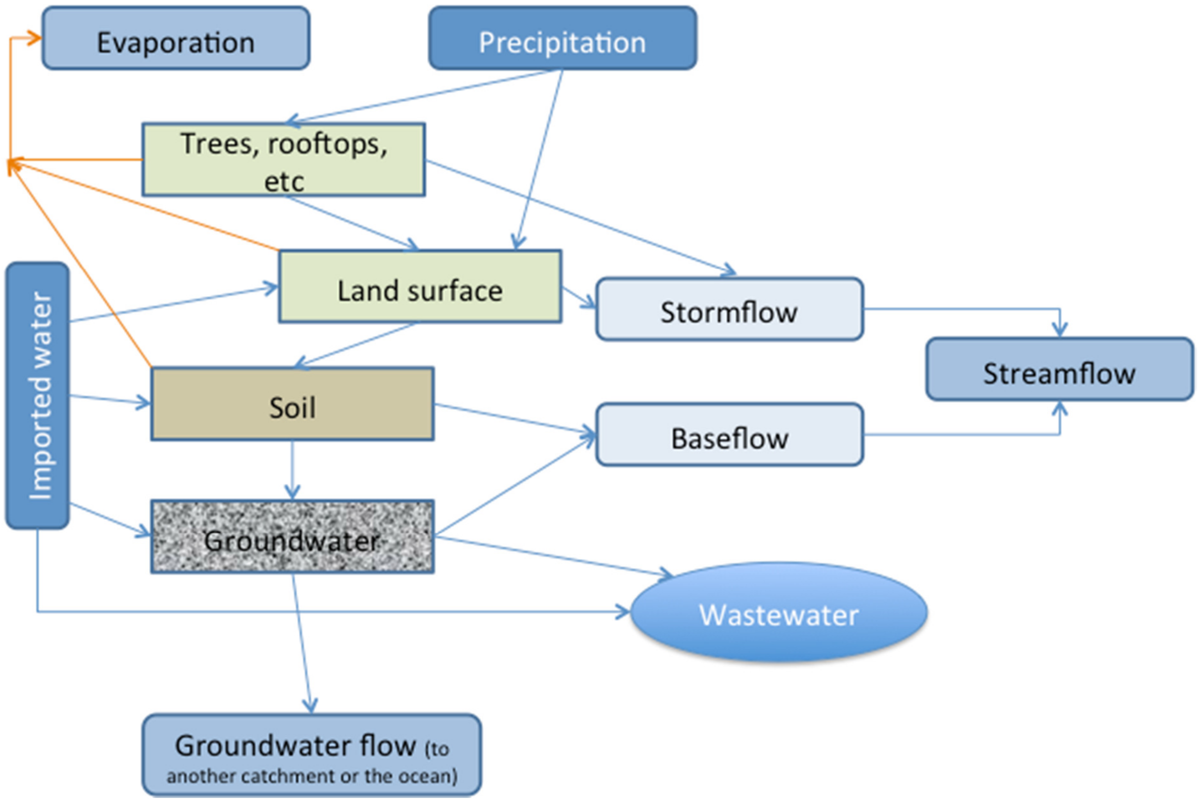

Figure 2.1 Simple Model of Water Inputs, Storage and Flows in an Urban Catchment

2.2 Human Impact on the Hydrologic Cycle

2.2.1 Urban Water Balance

To gain an insight into urban hydrology we need to consider the hydrological cycle at different temporal and spatial scales. At the spatial scale of a suburb or city, a water balance can identify the influence of imported water on catchment hydrology.

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

$$\Delta S = (P + I) - (E_a + R_s + R_w) \quad (1)$$

Where: ΔS is the change in catchment storage
P is precipitation
I is imported water
E_a is actual evapotranspiration
R_s 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). Although there are substantial differences in climate of these study areas, and the number of examples is small, we can make some generalisations.

- Wastewater leaving a catchment is less than (59% to 86% of) the amount of water that is imported. This means that imported water contributes to stormwater and/or evapotranspiration. As a result, stormwater plus evaporation exceeds precipitation in all case studies.
- Imported water is about 30% (30% to 39%) of precipitation. That is, 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 2.1 Annual Water Balance Data From Suburbs of Australian Cities[1]. Units are mm.

Location	Input			Output				Wastewater /imported water
	Precip.	Imported water	Imported water as a percentage of precipitation	Actual evapo-transpiration	Storm water runoff	Waste water runoff	Change in store (miss-close)[d]	
Curtin, ACT (Mitchell, et al. 2003) (1979-1996)	630	200	32%	508	203	118	1	59%
Sydney (Bell, 1972) (1962-1971)	1150	349[a]	30%	736	501	262	0	75%
Sydney (Kenway et al., 2011) (2004-2005)	952	370	39%	766	281	319	-44	86%
Subiaco-Shenton Park Perth (McFarlane, 1985)	788	285 + 96[b]	36%	766	104	154	117[c]	54%
Melbourne (Kenway et al., 2011) (2004-2005)	763	237	31%	688	165	190	-43	80%
South East Queensland (Kenway et al., 2011)[e] (2004-2005)	1021	374	37%	814	390	179	12	49%

[a]Includes imported water and use of groundwater
[b]Inflow of stormwater from upstream area
[c]Adjusted for change in groundwater storage
[d]See original studies for details
[e]Kenway et al. (2011) also estimate a water balance for Perth but this was not accurate and is not considered further.

[1] 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. However, these accounts also include substantial rural water use in surrounding areas so are less useful for isolating urban influences.

2.2.2 Lessons From a Detailed Water Balance Study, Curtin ACT

The most detailed information on an urban water balance undertaken in Australia is available for Curtin, ACT where Mitchell et al., (2003) obtained sufficient information to construct an annual water balance for the period Jan 1978 to June 1996. This study provides information on the variability in the urban water balance over time and the influence of climate (Table 2.).

Table 2.2 Water Balance for Curtin Catchment, Canberra for the Period 1979 – 1995. Units are mm. (Adapted from Mitchell et al., 2003)

Year	Precipitation	Imported Water	Actual evapotranspiration	Stormwater runoff	Wastewater runoff	Change in storage
Driest	247	269	347	74	107	-12
Average	630	200	508	203	118	1
Wettest	914	141	605	290	126	34

On average, annual input and output was about 830 mm. Approximately 24% (200 mm) of water was imported to the catchment via the supply system. The remaining 630 mm, was contributed by precipitation. Outputs where divided between actual evapotranspiration (61%, 508 mm), stormwater runoff (24%, 203 mm) and wastewater runoff (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 stormwater than was input via precipitation. Also, in all but the driest years wastewater plus stormwater were greater than imported water indicating the potential for harvest of suburban discharges to meet water demands but also highlighting 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 is three times as much rainfall as water imports but in the driest year, more water was imported to the catchment than fell as rain. In the wettest year, imported water made up only 13% of water input (Figure).

Considering outputs, the largest term is evapotranspiration which represents 59% or more in each year. Although the total evapotranspiration varies between 347 mm and 605 mm between dry and wet years, the proportion of water lost as evapotranspiration is reasonably constant (59% to 66%) (Figure). 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 determined by climate, changing by a factor of about 4 from 74 mm in the driest year to 290 mm in the wettest. Woolmington and Burgess (1982) demonstrated the direct link between garden watering and augmentation of low flows in Canberra urban streams, although this is no doubt moderated by water restrictions.

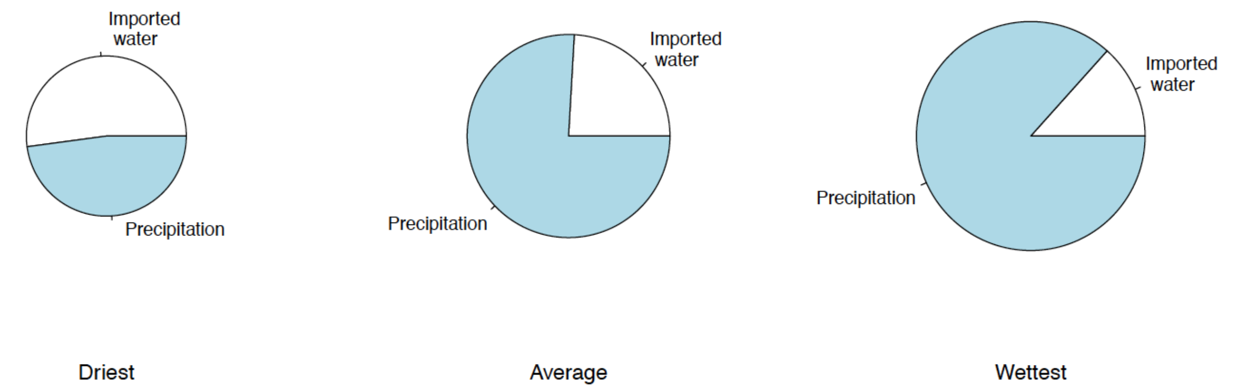

Figure 2.2 Total Water Input to Curtin, ACT: relative amounts of precipitation and imported water for the driest, average and wettest year (area of pie chart is proportional to total input). The proportion of imported water increases in drier years.

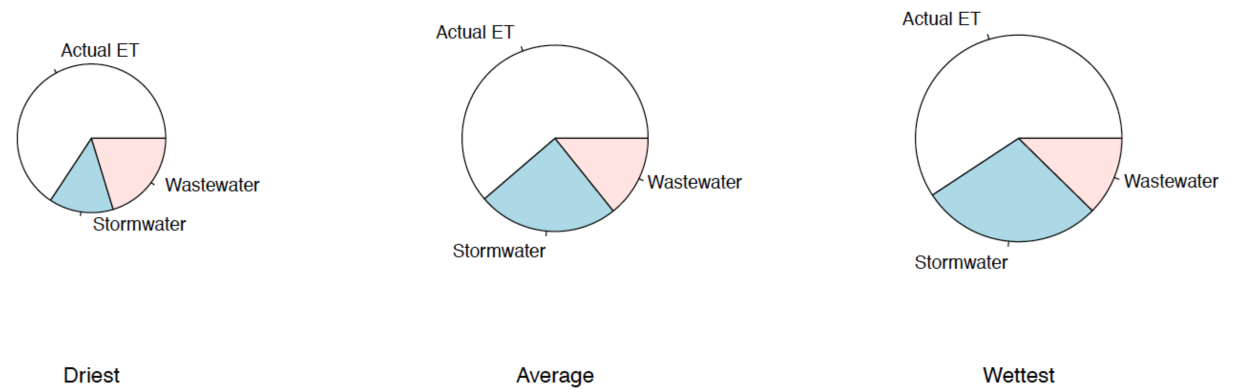

Figure 2.3 Total Water Output from Curtin, ACT: relative amounts of actual evapotranspiration, stormwater and wastewater for the driest, average and wettest year (area of pie chart is proportional to total output). The proportion of stormwater increases in wetter years

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

2.2.2 Comparison of Rural and Urban Water Balances

There are 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, drainage 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 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.

2.3 Aspects of Urban Drainage Systems

2.3.1 Impervious Areas

An water balance provides the overall context for hydrologic changes caused by urbanisation but to identify the impacts on flood flows we need to consider changes at shorter time periods and two key effects of develop:

1) the effect of the expansion of impervious areas; and
2) efficient drainage systems (Hollis, 1988; Schueler, 1994; Jacobson, 2011).

First, urbanisation results in impervious surfaces replacing vegetated soils. This:

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

Second, the natural stream network is augmented by piped drainage that directly collects water from roofs and roads throughout the urban catchment. The expanded 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
- 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 flow volume
- Increased frequency of high flow events
- Increased magnitude of high flow events
- Increased rates of change (both rising and falling limb of the hydrograph)
- 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
- 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. 2005a).
Key hydrologic changes are considered in more detail in the following sections.

Increased Flow Volumes

More rainfall is converted to runoff in urban catchments both because of the increased impervious areas and because of increased runoff from pervious areas which are commonly compacted and/or irrigated by imported water (Harris and Frantz, 1964; Cordery, 1976; Hollis and Ovenden, 1988a; 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 flood magnitude as a consequence of urbanisation has been recognised for many decades (e.g. Leopold, 1968). Urbanisation causes up to a 10 fold increase in peak flows of floods in the range 3 months to 1 year 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.

In Australia, increased flood magnitudes have been confirmed by analysis of paired catchment data, for example the comparison of urban Giralang and rural Gungahlin catchments in Canberra (Codner et al. 1988) as well as numerous modelling studies e.g. 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 compared to rural catchments and recedes more quickly. The quick response means there are more flow peaks in urban streams (Mein and Goyen, 1988; McMahon, 2003; Baker et al., 2004; Heejun, 2007). In work in Canberra, urbanisation was found to reduce the volume of channel storage by a factor of 30 (Codner, et al., 1988). This contributed 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 – has been found to decrease by 1.5 to 10 times with urbanisation (Packman, 1981; Bufill and Boyd, 1989).

Increased Runoff Frequency

Runoff occurs more frequently as the amount of impervious area increases. 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; 2014). This means that runoff frequency can increase by a factor of 10 or more.

The increased responsiveness to rainfall means the seasonality of flows in urban streams is changed. In many areas, rural catchments only produce runoff after they have wet-up following a long period where rainfall exceeds evapotranspiration. As a result, flows occur seasonally in many rural catchments with little runoff when the catchment is dry even when there is heavy rainfall (Western and Grayson, 2000). In urban streams, flow occurs anytime there is rain. In temperate urban catchments, the largest urban runoff often occurs following the intense rain of a thunderstorm during summer when, in the equivalent rural catchment, there is little flow (Codner et al., 1988; Smith et al., 2013).

Changed Base Flows

The influence of urbanization on groundwater, and hence stream baseflow, is complex. Various features of urbanization have confounding effects and their relative magnitude will determine the overall influence on baseflow. These features include:

- reduced vegetation cover
- increased in impervious surfaces which means less infiltration but also reduced evaporation of shallow groundwater
- infiltration from garden irrigation
- water leaking from pipes which contributes to ground water
- drainage of groundwater into pipes or the gravel filled trenches that surround pipes.

The most common response to urbanisation is that base flow is decreased. More impervious areas means less opportunity for water to infiltrate so groundwater storage and discharge is 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 water conveyance. The urban drainage network is denser and more extensive then the natural stream system it replaces. This means that water is conveyed rapidly from both pervious and impervious surfaces throughout an urban catchment. Flow resistance is lower in the straight, smooth, drainage paths of urban waterways than 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 has a major influence on peak discharge that follows urbanisation. The largest impacts occur when urban streams are lined i.e. they are made hydraulically efficient.

This was confirmed of catchments with similar imperviousness but with and without conventional urban drainage. The hydrologic alteration was much reduced in those suburbs with less efficient informal drainage 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 (Walsh et al., 2005b).

Conveyance of Flood Flows

Understanding the conveyance of water in urban areas during times of overland flooding is a critical part of urban drainage analysis and design. The major/minor drainage principle requires that flow paths must be considered once water can on longer be contained in pipes, but behaviour can be complex. In many areas, modelling of overland flow paths is being used to guide land zoning to control development and so reduce flood risk (Baker et al., 2005).

The catchment boundary for overland flow will often differ from that for piped flow which means that the behaviour of large floods may differ substantially from smaller events so has the potential to take people by surprize. An example is shown in Figure below for a suburb protected from riverine flooding by a levee. Stormwater is usually discharged under the levee into the river. Overland stormwater flooding cannot reach the river because of the levee and instead backs up and floods houses. The catchment draining to the houses near the levee becomes the whole of the whole suburb during overland flooding whereas, under low flows, the catchment drained to the creek via a pipe. This type of unexpected and rapid flooding can be dangerous as people are unlikely to be prepared.

Figure 2.4 Normal Conditions: a suburb is protected from flooding from a creek by a levee. Stormwater drains to the creek. Overland Stormwater Flooding: water ponds against the levee potentially houses near the levee

2.3.3 Receivers

Many urban areas are situated on estuaries or bays, which will provide a downstream boundary for stream water levels. 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 use as part of the development of flooding scenarios for planning (e.g. Melbourne Water, 2012).

Major rivers, which flow through urban centres, will also often be the receivers of urban stormwater. These rivers will determine the base level to be used for modelling so separate analysis 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 drainage 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 reduced flooding in these 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 these impacts.

2.3.4 More Complex Than Rural

Many aspects of urban flooding are more complex then 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 flood magnitude
- Floods can occur at any time i.e. there is not requirement for antecedent rainfall to prime the catchment for 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 and stormwater overflows.

In reviewing the components of average annual flood damage, Ronen (2009) suggested that, in general, risks form riverine flooding had been reasonable 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 drainage systems. This is generally true for Australian towns and cities. There are, however, two areas that 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: Wolloomooloo, Blackwattle Bay, Hay Street, Tank Stream and Bennelong. These discharged to Sydney Harbour. With the construction of the Bondi Ocean Outfall Sewer in 1889, most of these original sewers were converted to carry stormwater only, with wastewater being 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).

2.4 References

ASCE (American Society of Civil Engineers) (1975) Aspects of hydrological effects of urbanization. ASCE Task Committee on the effects of urbanization on low flow, total runoff, infiltration, and groundwater recharge of the Committee on surface water hydrology of the Hydraulics Division. *Journal of the Hydraulics Division* **HY5**: 449-468.

Al-Rashed, M. F. and M. M. Sherif (2001) Hydrogeological aspects of groundwater drainage of the urban areas in Kuwait City. *Hydrological Processes* **15**: 777-795.

Askarizadeh, A., M. A. Rippy, T. D. Fletcher, D. L. Feldman, J. Peng, P. Bowler, A. S. Mehring, B. K. Winfrey, J. A. Vrugt, A. AghaKouchak, S. C. Jiang, B. F. Sanders, L. A. Levin, S. Taylor and S. B. Grant (2015) From rain tanks to catchments: use of low-impact development to address hydrologic symptoms of the urban stream syndrome. *Environmental Science and Technology* **49**(19): 11264-11280.

Baker, A., P. Rasmussen, K. Parkyn, R. Catchlove and E. Kazazic (2005) A Case Study of the December 2003 Melbourne Storm: the Meteorology, Rainfall Intensity, and Impacts of Flash Flooding. *29th Hydrology and Water Resources Symposium*. Canberra. Engineers Australia.

Baker, D.B., Richards, R.P., Loftus, T.T., Kramer, J.W., 2004. A new flashiness index: characteristics and applications to Midwestern rivers and streams. *Journal of the American Water Resources Association* **40**: 503-522.

Barron, O. V., Barr, A. D. and Donn, M. J. (2013) Effect of urbanization on the water balance of a catchment with shallow groundwater. *Journal of Hydrology* 485:162-176.

Beighley, R. E. and G. E. Moglen (2002) Trend assessment in rainfall-runoff behaviour in urbanising watersheds. *Journal of Hydrologic Engineering* **7**(1): 27-34

Bell, F. C. (1972) The acquisition, consumption and elimination of water by the Sydney Urban System. *Proceedings of the Ecology Society of Australia* **7**:161-176.

Bhaskar, A. and C. Welty (2012) Water Balances along an Urban-to-Rural Gradient of Metropolitan Baltimore, 2001–2009. *Environmental & Engineering Geoscience* **28**(1): 37-50.

Boyd, M. J., M. C. Bufill and R. Knee (1993) Pervious and impervious runoff in urban catchments. *Hydrological Sciences Journal* **38**(6): 463-478.

Boyd, M. J., Bufill, M. C. and Knee, R. M. (1994) Predicing pervious and impervious storm runoff from urban drainage basins. Hydrological Sciences Journal **39**(4): 321-332.

Brandes, D., Cavallo, G.J. and Nilson, M.L. (2005) Base flow trends in urbanizing watersheds of the Delaware river basin. *Journal of the American Water Resources Association* **41** (6): 1377-1391.

Brath, A, Montanari, A, and Moretti, G. (2006) Assessing the effect on flood frequency of land use change via hydrological simulation (with uncertainty), *Journal of Hydrology* **324**(1-4): 141-153.

Bufill, M. C. and Boyd, M. J. (1989) Effect of urbanization on catchment lag parameters. *Hydrology and Water Resources Symposium*. Christchurch, NZ, 23-30 November. 171-178.

Bureau of Meteorology (2015) National Water Accounts. http://www.bom.gov.au/water/nwa/ (accessed 12 June 2015).

Carroll, D. G. (1995) Assessment of the effects or urbanisation on peak flow rates. *Second International Symposium on Urban Stormwater Management 1995: Integrated Management of Urban Environments*. Engineers Australia, Canberra. 201-208.

Codner, G. P., E. M. Laurenson and R. G. Mein (1988). Hydrologic Effects of Urbanisation: A case study. *Hydrology and Water Resources Symposium*, ANU Canberra, 1-3 Feb 1988, Institution of Engineers Australia. 201-205.

Cordery, I. (1976). Some effects of urbanisation on streams. *Civil Engineering Transactions, The institution of Engineers, Australia*: **CE18**(1): 7-11.

Coutts, A. M., J. Beringer, S. Jimi and N. J. Tapper (2009). The urban heat island in Melbourne: drivers, spatial and temporal variability, and the vital role of stormwater. *Stormwater 2009*. Stormwater Industry Association.

Delleur, J. W. (2003) The evolution of urban hydrology: past, present and future. *Journal of Hydraulic Engineering* **129**: 563-573.

Epsey, W. H. and Winslow, D. E. (1974) Urban flood frequency characteristics. *Journal of the Hydraulics Division*. American Society of Civil Engineers **100(HY2)**:279-293.

Ferguson, B. K. and P. W. Suckling (1990) Changing rainfall-runoff relationships in the urbanizing Peachtree Creek Watershed, Atlanta, Georgia. *Water Resources Bulletin* **26**(2): 313-322.

Grimmond, C. S. B. and T. R. Oke (1986) Urban Water Balance: 2. Results From a Suburb of Vancouver, British Columbia. *Water Resources Research* **22**(10): 1404-1412.

Harris, E. E. and Franz, S. E. (1964) Effect of urban growth on streamflow regimen of Permanente Creek Santa Clara County, California: Hydrological effects of urban growth. Geological survey water-supply paper 1591-B. US Department of the Interior.

Heejun, C. (2007). Comparative streamflow characteristics in urbanizing basins in the Portland Metropolitan Area, Oregon, USA. *Hydrological Processes*, *21*(2), 211–222.

Hill, P. I., Graszkiewicz, Z., Taylor, M. and Nathan R. (2014) Loss models for catchment simulation: Phase 4. Analysis of Rural Catchments. Australian Rainfall and Runoff Report No. P6/S3/016B. Engineers Australia.

Hill, P. I., R. G. Mein and L. Siriwardena (1998). How much rainfall becomes runoff: loss modelling for flood estimation. Cooperative Research Centre for Catchment Hydrology, Melbourne.

Hollis, G. E. (1988) Rain, Roads, Roofs and Runoff: Hydrology in Cities. *Geography* **73**(1): 9-18.

Hollis, G. E. and J. C. Ovenden (1988) One year irrigation experiment to assess losses and runoff volume relationships for a residential road in Hertfordshire, England. *Hydrological Processes* **2**: 61-74.

Hollis, G. E. and J. C. Ovenden (1988) The quantity of stormwater runoff from ten stretches of road, a car park and eight roofs in Hertfordshire, England during 1983. *Hydrological Processes* **2**: 227-243.

Hollis, G. E., (1975) The effect of urbanization on floods of different recurrence interval *Water Resources Research*, **11**:431-435.

Jacobson, C. R. (2011) Identification and quantification of the hydrological impacts of imperviousness in urban catchments: a review. *Journal of Environmental Management* **92**: 1438-1448.

Kenway, S., A. Gregory and J. McMahon (2011) Urban Water Mass Balance Analysis. *Journal of Industrial Ecology* **15**(5): 693-706.

Ku, H.F.H., Hagelin, N.W., Buxton, H.T., 1992. Effects of urban storm-runoff control on Ground-water recharge in Nassau County, New York. *Ground Water* **30** (4): 507-514.

Lerner, D. N. (2002) Identifying and quantifying urban recharge: a review. *Hydrogeology Journal* **10**: 143-152.

Leopold, L. B. (1968). Hydrology for urban land planning - A guidebook on the hydrologic effect of urban land use. *U.S Geological Survey, Circular*. **554:** 18. http://pubs.usgs.gov/circ/1968/0554/report.pdf

McFarlane, D. J. (1984) The effect of urbanisation on ground-water quality and quantity in Perth, Western Australia. PhD thesis. The University of Western Australia.

McMahon, G., Bales, J.D., Coles, J.F., Giddings, E.M.P., Zappia, H., (2003) Use of stage data to Characterize hydrologic conditions in an urbanizing environment. *Journal of the American Water Resources Association* **39** (6): 1529-1546.

Mein, R. G. and Goyen, A. G. (1988) Urban runoff. Civil Engineering Transactions. **CE30**(4): 225-238.

Melbourne Water (2012) Planning for sea level rise. 5 June 2012. http://www.melbournewater.com.au/Planning-and-building/Forms-guidelines-and-standard-drawings/Documents/Planning-for-sea-level-rise-guidelines.pdf

Mitchell, V. G., T. A. McMahon and R. G. Mein (2003) Components of the Total Water Balance of an Urban Catchment. *Environmental Management* **32**(6): 735-746.

Packman, J. C. (1981) Effects of catchment urbanization on flood flows. ICE, Flood studies report – Five Years On. Proceedings of a conference organized by the Institution of Civil Engineers, held in Manchester, 22-24 July. Thomas Telford Ltd, London, 121-129.

Price, K. (2011) Effects of watershed topography, soils, land use, and climate on baseflow hydrology in humid regions: A review. *Progress in Physical Geography* **35**(4): 465-492

Prosdocimi, I., T. R. Kjeldsen and J. D. Miller (2015) Detection and attribution or urbanization effect on flood extremes using nonstationary flood-frequency models. *Water Resources Research* **51**(6): 4244-4262.

Ronan, N. M. (2009). Future flood resilience - Victoria's next strategy. Albury, Floodplain Management Association.

Schueler, T. (1994) The importance of imperviousness. *Watershed Protection Techniques* **1**(3).

Simmons, D. L. and R. J. Reynolds (1982) Effects of urbanisation on base flow of selected south-shore streams, Long Island, New York. *Water Resources Bulletin* **18**(5): 797-805.

Smith, B. K., J. A. Smith, M. L. Baeck, G. Villarini and D. B. Wright (2013) Spectrum of storm event hydrologic response in urban watersheds. *Water Resources Research* **49**: 2649–2663.

Stephenson, D. (1994) Comparison of the water balance for an undeveloped and a surburban catchment. *Hydrological Sciences Journal* **39**(4): 295-307

Tarr, J. A. (1979) The separate vs. combined sewer problem: a case study in urban technology design choice. *Journal of Urban History* **5**(3): 308-339.

Tholin, A. L. and Keifer, C. J. (1959) The hydrology of urban runoff. *Journal of the Sanitary Engineering Division* **85**(SA2): 47-106.

Walsh, C. J., A. Roy, H., J. W. Feminella, P. Cottingham, P. M. Groffman and R. Morgan (2005a) The urban stream syndrome: current knowledge and the search for a cure. *Journal North American Benthological Society* **24**(3): 706-723.

Walsh, C. J., Fletcher, T. D. and Burns, M. J. (2012) Urban stormwater runoff: a new class of environmental flow problem. *PLOSone* DOI: 10.1371/journal.pone.0045814

Walsh, C. J., Fletcher, T. D., & Ladson, A. R. (2005b) Stream restoration in urban catchments through redesigning stormwater systems: looking to the catchment to save the stream. *Journal of the North American Benthological Society*, *24*(3), 690-705.

Western, A. W. and Grayson, R. B. (2000) Soil moisture and runoff processes at Tarrawarra. In: Grayson, R. G. and Blöschl, G. (eds) *Spatial Patterns in Catchment Hydrology: Observations and Modelling*. Cambridge University Press, 209-46.

Wong, T., P. F. Breen and S. Lloyd (2000). *Water sensitive road design - design options for improving stormwater quality of road surfaces*. Cooperative Research Centre for Catchment Hydrology. Melbourne.

Woolmington, E. and J. S. Burgess (1983) Hedonistic water use and low-flow runoff in Australia's national capital. *Urban Ecology* **7**(3): 215-227.

Chapter 3
Urban Drainage Philosophy

Peter Coombes, Steve Roso

Chapter Status	
Book	9
Chapter	3
Date	27/11/2015
Content	Working draft
Graphs and Figures	In preparation
Examples	N/A
General	Need to integrate with other chapters and check consistency

3.1 Introduction

Urban stormwater management has been described historically as the hydraulic design of urban drainage networks to convey stormwater runoff to a disposal point.

There have been many changes in our approach to urban water management in Australia since the establishment of the centralised and separate water supply, stormwater and wastewater paradigm in the 1800s. Urban water management has evolved to include protection of waterways, mitigation of stormwater quality, and the use of Water Sensitive Urban Design (WSUD) and Integrated Water Cycle Management (IWCM) approaches. Although these approaches are relatively new, they have wide adoption and support in legislation and policies for water management throughout Australia. 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 to mitigate nuisance, and to avoid damage to property and loss of life.

3.2 The Journey from 1987 to 2015

There has 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 impacts 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 ARR87 guideline focused on the collection and conveyance of peak stormwater flows in drainage networks. Advice on hydrologic and hydraulic analysis was consistent with the emerging computer age and hand calculation, programmable calculator and computer methods were discussed. The increasing complexity of the different methods and an associated requirement for use of computers was highlighted.

The 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 major objective of urban drainage at the time was to convey stormwater from streets and adjoining properties without nuisance for minor rain events, and to avoid flooding of property 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 and with streets, open space and trunk drains to transport major flows. Trunk drainage was described to include designs for open channels, detention and retention basins, and bridges. Whilst urban stormwater management was presented and interpreted as a drainage approach, Chapter 14 in ARR87 also highlighted that urban drainage solutions should also:

- Limit pollutants entering receiving waters;
- Consider water conservation;
- Integrate into 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 were solely focused on the developed catchment and were mostly designed by engineers. The simplicity of the methods for estimating stormwater runoff implied accuracy and certainty of design performance to many users. Urban water management further evolved in the mid 1990's to include protection of waterways[1], mitigation of urban stormwater quality, WSUD (Whelans et al., 1994) and IWCM approaches. 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 (EA., 2006).

The acceptance of WSUD, IWCM and related approaches has been manifested in three significant ways – the development of benchmark projects [e.g; Lynbrook Estate (Lloyd et al., 2002) and Fig Tree Place (Coombes et al., 2000)] that provided evidence that these new approaches were successful, the creation of local policies and plans for integrated water management and, in recent times, the adoption of policies for sustainable water management by State and Federal governments. Recent droughts also triggered many other changes in the urban water sector, largely associated with water conservation, harvesting, recycling and reuse (Aishett & Steinhauser, 2011).

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, liveability, 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 & 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 which need to be properly understood to develop integrated solutions.

Argue (2004) suggests that the urban designer now aims to manage the impacts 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., 2014). 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).

1 Increases flow volumes and rates from urban areas (flow regimes) contributes to degradation of riparian ecosystems and promotes geomorphological changes within stream beds

3.3 The Opportunities and Challenges of 2015

Urbanisation generates dramatic changes to the natural water cycle. Impervious surfaces and directly connected drainage infrastructure decrease 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 3.1. The impervious surfaces and hydraulically efficient infrastructure associated with urban catchments increases the magnitude and frequency of stormwater runoff whilst 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.

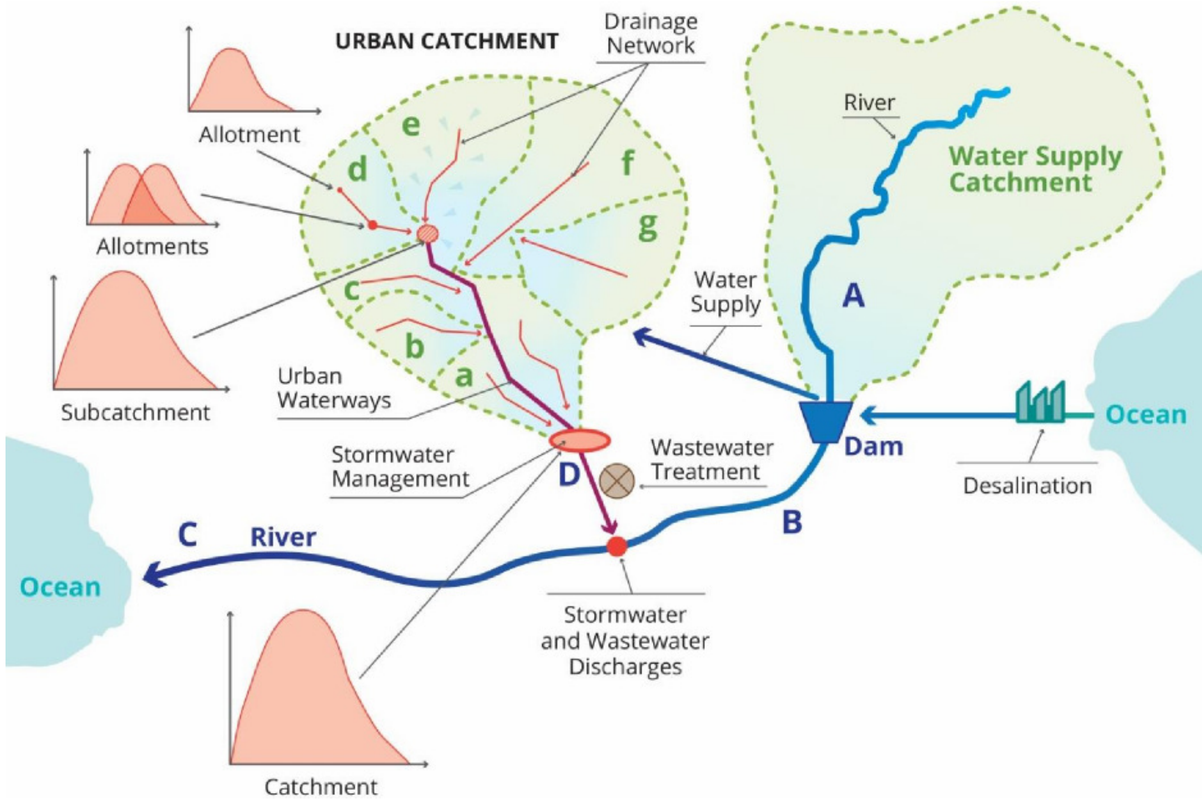

Figure 3.1 Schematic of traditional urban catchments and cumulative stormwater runoff processes

Figure 3.1 demonstrates that analysis and solutions at point D at the bottom of urban catchments 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 management of flows and water quality 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 3.1 also highlights that distributed land uses (allotments or properties) produce hydrographs of stormwater runoff into the street drainage system. The street drainage system accumulates stormwater runoff from multiple inputs that creates progressively larger volumes of stormwater runoff that ultimately flows into urban waterways or adjoining catchments (Pezzaniti et al., 2002). This process results in dramatic changes in the volume and timing of stormwater discharging to downstream environments.

There has been a progressive realisation that this issue can be solved by viewing urban stormwater as an opportunity to supplement urban water supplies and to enhance the amenity of urban areas (Mitchell et al., 2003; Coombes & Barry., 2006; Wong, 2006).[2] 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). 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 & Barry, 2014).

Changes in land uses, climate change, increased density of urban areas or decline in the hydraulic capacity of aging drainage networks can also 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, 2006). This will exacerbate the challenges of providing secure water supplies and mitigating risks of urban stormwater runoff. There may also be a need to replace stormwater networks installed during post war urban redevelopment that are nearing the end of useful life. In this situation, the capacity of an aging drainage networks or increased runoff from increasing density of development 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 discharging towards waterways (pluvial flooding) rather than from flood flows originating from waterways (fluvial flooding). There is a need to consider a more extensive range of stormwater runoff events from the frequent to the rare or extreme and the associated impacts on urban environments (Weinmann, 2007). Management of these flood related impacts requires integrated water management of the full spectrum of flood events (Figure 3.2).

Figure 3.2 highlights the evolving analysis methods, including continuous simulation and Monte Carlo simulation of full storm volumes that are likely to be required throughout the 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 about frequency and magnitude that requires clarification to allow more effective advice on design of stormwater management schemes[2], including development of green infrastructure and microclimates with reduction of urban heat island effects.

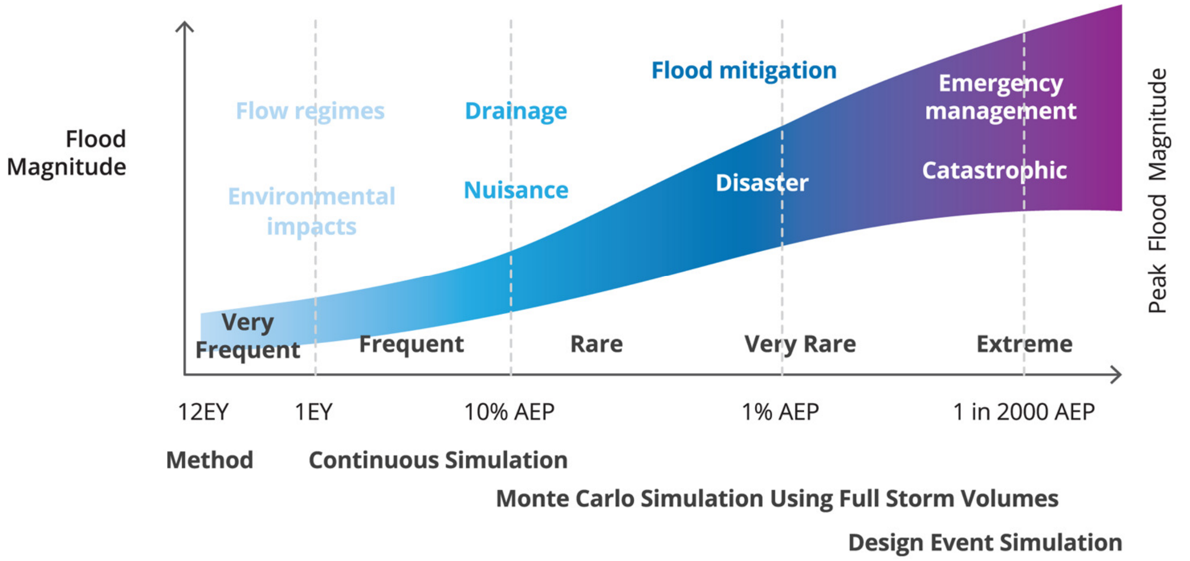

Figure 3.2 The full spectrum of flood events (adapted from Weinmann, 2007)

The 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, 2012). These integrated strategies diminish the requirement to transport water, stormwater and wastewater across regions with associated reductions in the costs of extension, renewal and operation of infrastructure (Coombes & Barry, 2014). This leads to decreased requirement to augment regional water supplies and long run economic benefits.

Strategies that focus on restoring more natural flow regimes in waterways will be beneficial in reducing remedial works in waterways and to a reduction in the 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, includes 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. As a consequence, 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 reasonably be 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 behavior from the smallest distributed scales (from the 'bottom up') (Argue, 2004; Coombes & Barry, 2014). 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 3.3.

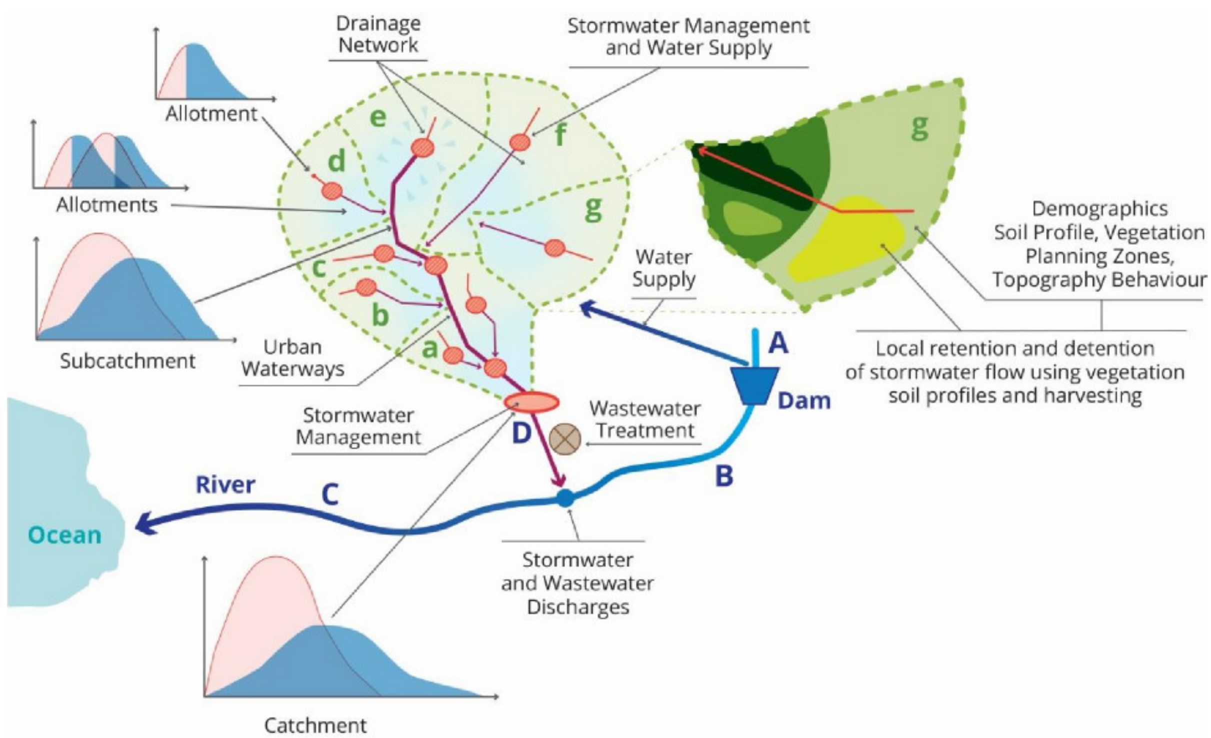

Figure 3.3 Cumulative impacts of distributed retention measures

It also follows that historical 'top down' design processes cannot 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 retention measures) in a system are smoothed or completely lost by partial use of data as averages and by the scale of analysis. As a consequence, there is no direct mechanism to capture cascading changes in behaviors throughout a system.[3] For a simple example, consider the connectivity of contemporary water cycle networks presented in Figure 3.4.

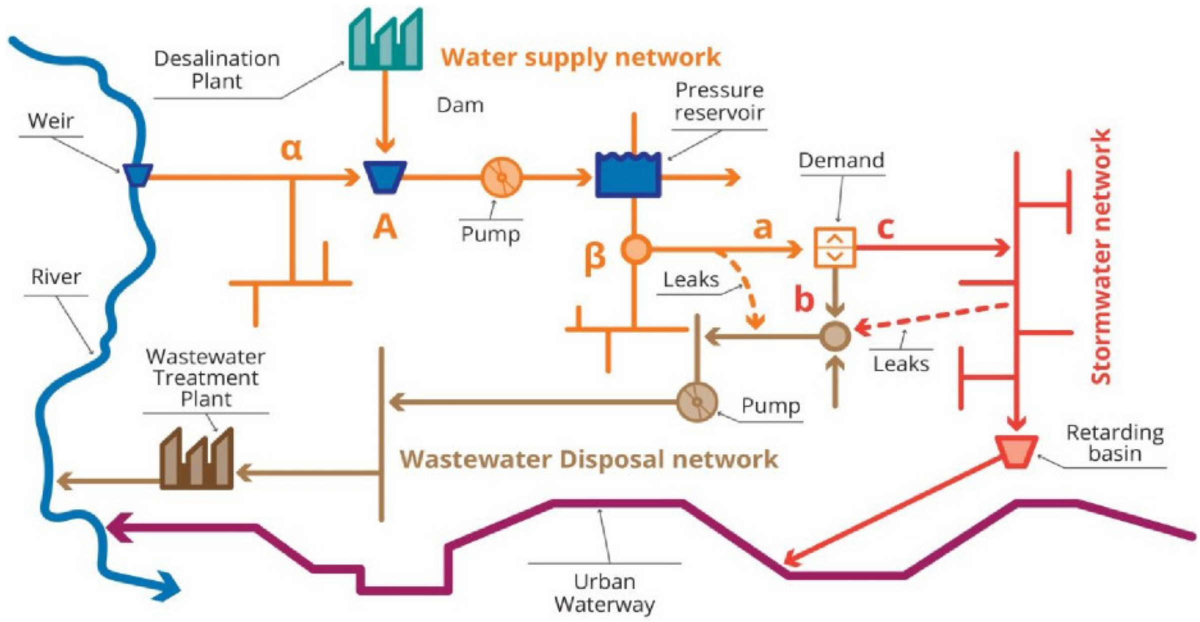

Figure 3.4 Schematic of the connectivity of urban water networks

3 This can lead to competing objectives (e.g. local versus regional) and information disparity which can only be resolved through a broader analysis framework which recognises location based principles of proportionality and efficient intervention. For example, provision of a wetland and retarding basin downstream of an urban area when management is required within the urban area to protect urban amenity and avoid local flooding.

Figure 3.4 shows that an input, or extraction at any point α or β, or an increase in water storage in a reservoir, say at A, will have some influence on flows and capacities at many other points in the system. These will, in turn, 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. Inclusion of rainwater and stormwater harvesting, and wastewater reuse further increases the level of connectivity of urban water networks.

The current practice for estimation of stormwater runoff rates and the design of drainage infrastructure is based on a methodology where all inputs, other than rainfall, are fixed variables. The fixed values of the inputs are selected to ensure that the exceedance probability of stormwater runoff is similar to that of the rainfall. 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 & 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 (such as human behaviour, rainfall and soil processes) and the ultimate stormwater runoff behaviour are unknown for each project.
- Integrated solutions often meet multiple objectives (such as water supply, stormwater drainage, management of stormwater quality, provision of amenity and protection of waterways) and are dependent on linked interactions with surrounding infrastructure.
- 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).[4] Assumptions and methods of analysis imposed by approval authorities in accordance with ARR87 can constrain the use of more appropriate analysis techniques required to better understand the behaviour of integrated solutions. Similarly, a default requirement by approval authorities for drainage 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, either directly or indirectly, may not reliably produce probabilistic design of drainage, water quality, water or wastewater infrastructure within integrated strategies. Whilst 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 to provide an authorizing guidance for more advanced methods for design of integrated solutions.

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

4 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. For example, use of a 100 year rainfall record allows analysis of about 36,500 of joint probability intersections of integrated solutions at daily time steps.

It would also seem that the definition and purpose of the minor/major drainage system is unclear in the context of modern approaches to water cycle management. Replacement of the minor or major drainage description with a definition of managing nuisance or disaster would provide a clearer focus on the relative importance of both concepts. We may be too focused on a prescriptive drainage approach to the minor system to avoid nuisance. 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 Major and Minor Drainage Systems

A typical drainage system must convey a wide range of flows within a confined corridor of land. 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 drainage system'.

A major and minor drainage system has two parts:

1. A small channel or conduit referred to as the 'minor system'. The minor system conveys stormwater runoff produced during small frequent storm events. This runoff is conveyed in a manner that maintains safety, minimises public nuisance and minimises potential maintenance problems such as ponding and saturation of normally dry ground. The minor system must have durability to withstand the effects of inundation.

2. A surface flow path referred to as the 'major system'. The major system conveys additional stormwater runoff produced during larger less probable and less frequent storm events. This runoff is potentially hazardous due to its velocity and depth and must be safely contained within the major system corridor.

3.4.1 Flood Capacity

The overall combined flood capacity of the major and minor drainage system needs to be established for design. This capacity is normally expressed in terms of the exceedance probability of the flood to be contained within the drainage system.

It is common practice to set the capacity 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 (QUDM, 2013)[1]

For example where the consequences of flooding for a particular location are high, it may be necessary to expand overall system capacity to cater for extreme events. This is not commonly required and any such decision must have regard to the overall life-cycle cost and benefits that a larger capacity system could deliver.

The threshold at which the minor system discharge capacity is exceeded and the major system begins to convey runoff is also a matter for consideration either at the design stage, or for policy makers at the time when preparing local drainage design standards. Typically the minor system capacity is set somewhere within a range between a 50% AEP and a 5% AEP flood event. Documentation of these standards can be found in drainage design guidelines prepared by local government and relevant state authorities such as the Queensland Urban Drainage Manual (QUDM, 2013). There is however no single universally appropriate minor system capacity that can be applied in practice.

Some factors that may influence the balance between major and minor system capacity are described in **Table 1** below. These factors may give rise to a number of different minor system capacity requirements for different applications within a particular state or local government jurisdiction.

Table 1 Factors influencing the balance between major and minor drainage system design capacity

Factor	Description
Land availability	Where a wide corridor of land is available for the major system it may be possible to safely convey additional flow on the surface and correspondingly reduce the proportion of flow conveyed within the minor system.
Local rainfall patterns	In some areas, such as tropical northern Australia, the runoff generated by a frequent high probability storm may be so large as to make it cost prohibitive to convey all this flow within a minor system. Major flow paths may therefore need to be expanded accordingly.
Likely level of exposure to the major flow path hazard	Where the major system is highly frequented by people or vehicles, for example in city streets or major motorways, the level of exposure to floodwaters is potentially greater and therefore also the corresponding risks. In these cases it may be appropriate that a greater proportion of the potential runoff be conveyed in the minor system.
Physical and downstream constraints	Where a system is retrofit into an existing urban area it may be impractical or cost prohibitive to achieve an ideal capacity and compromise may be required.
Erosion	For minor systems that are naturalised or otherwise unlined, erosion may occur if the flow duration and or velocity is too high. If armouring is not considered appropriate, minor system capacity expectations should be lowered and correspondingly more flow conveyed in the major system to manage these effects.
Blockage potential	Where the minor system is likely to become blocked with debris there may be merit in reducing the capacity of the minor system. Instead resources should be directed towards a safer and more durable major system flow path across the surface.
Climate change	In anticipation of predicted future increases in short duration rainfall intensity it may be appropriate to increase the capacity of the minor system to maintain current levels of service.

3.4.2 Alignment and Configuration

The overall alignment and configuration of the drainage system will influence other aspects of the urban form such as the layout of roads and the location of urban parkland. The drainage system once installed is also expected to have a long service life, and modifications are difficult to undertake at a later time.

Concept planning for the major and minor drainage system should therefore be undertaken carefully and as an early task when commencing the design of a new urban development precinct.

When calculating the dimensions of the drainage corridor, the depth and velocity of flow along any proposed surface flow paths must meet relevant standards for design, safety and maintenance. The design should at the same time ensure that operation of the drainage system during severe storms does not in itself cause unexpected or catastrophic consequences. For example the unintended diversion of flow into an adjoining catchment as a result of blockage or extreme events.

Where possible the width of the land corridor set aside for drainage purposes should be generous to improve the constructability of the system and reduce the costs of any future system renewal and maintenance activities. Co-location within urban parkland can assist in this regard.

The alignment of the drainage system should generally follow natural low points to minimise earthworks and ground disturbance. It is however expected that some re-alignment away from the natural low point will occur in order to provide for a sensible urban form and limit conflicts with other urban infrastructure.

The major system alignment should also where possible run parallel to the minor system and be continuous until it reaches a natural watercourse. Any crossings, such as roadways or footpaths, must be designed to have adequate conveyance.

The selected configuration will depend largely on the usage of the land within and alongside the surface flow path. The configuration may also vary along its course. **Figures 3.5** below shows some typical configurations deployed in Australian design practice. The most common configuration is shown in Figure 3.5 and comprises an underground pipe (minor system) beneath a roadway flow path (major system).

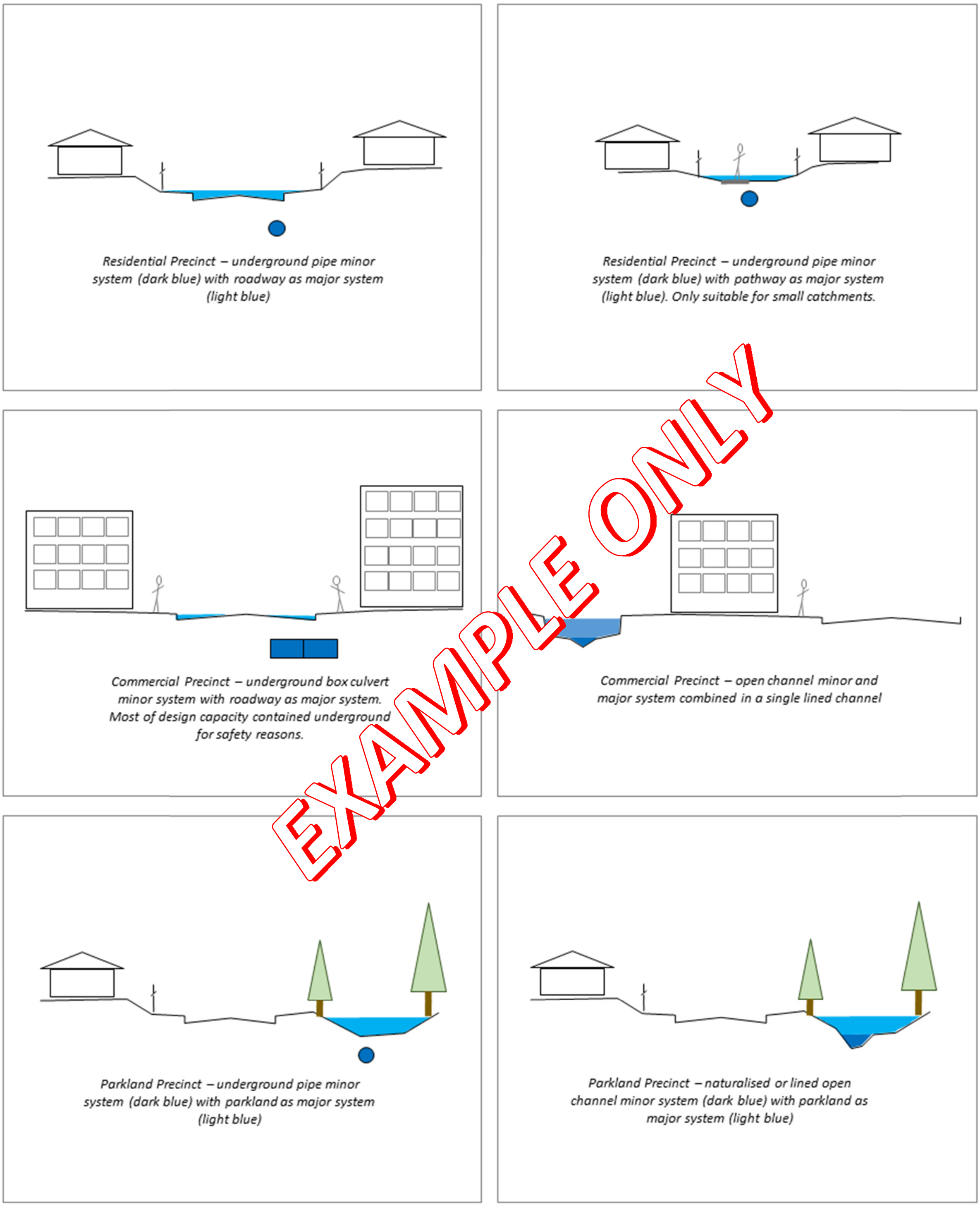

Figure 3.5 *Typical major minor drainage system configurations deployed in Australian practice*

The above figure is an example only. Graphics need improvement.

The design of the major and minor drainage system should integrate smoothly with other urban infrastructure and manage its impact on the natural landscape. In particular, when passing through urban parkland, innovative design can be used to achieve drainage objectives while also enhancing aesthetic and environmental outcomes.

There is also opportunity through innovative design to reduce drainage related construction costs and minimise land take. This opportunity should be given early consideration in the concept design phase by a suitably qualified team of designers from both parks planning and drainage design perspectives.

3.4.3 Analysis

Suitable hydrologic and hydraulic calculation methods, as further described within the following chapters, are used to estimate flood levels, depths and velocities along the course of the major and minor drainage system, allowing sizing of various components.

The methods selected must be matched to the complexity of the design problem. The design problem may include potentially complex flow behaviour, for example parallel underground and surface flow paths, multiple inflows and the effects of storage and tail water.

The methods must also be capable of predicting not only the hydraulic performance of the overall system, but also of each different component within the system. For example inlet structures, chambers, pipes, and channels. Hydraulic performance must be assessed for a range of storm scenarios.

A range of software modelling tools are available to automate aspects of these calculations. However, many do not seamlessly handle the full range of hydrologic and hydraulic calculations required, particularly where complex surface flow behaviour occurs in conjunction with complex pipe networks. For these complex scenarios the results from a hydrologic model may need to be coupled with a combination of 1d pipe network and 2d surface flow models.

3.5 Pluvial and Fluvial Flooding

In an urban flooding context, distinction is normally made between pluvial flooding (overland flow) and fluvial flooding (river and creek flooding). This distinction can be important as the two types of flooding are different in their behaviour and demand different analysis and management approaches.

3.5.1 Pluvial Flooding (Overland Flow)

Pluvial flooding is generally caused by short durations (minutes to hours) of intense rainfall, falling across a small catchment up to approximately 1 km^2 in area. This rainfall causes significant concentrations of surface runoff in low points and depressions, both natural and man-made. These concentrations of flow continue downslope and discharge into larger natural waterways with defined banks such as creeks, rivers or lakes where it becomes fluvial in character.

Pluvial flooding can be responsible for significant flood damage if an adequate major flow path has not been retained. Accordingly, pluvial flooding warrants a management program including systematic identification of pluvial flow paths and urban drainage design practices that recognize and respond to pluvial flood risk. In many cases, simple design practices such as slightly elevating the floor level above the surrounding terrain can effectively eliminate most pluvial flood risk.

Modelling approaches have been developed in recent years to assist with identification of pluvial flow paths. They involve the use of a ‘rain on grid’ hydrodynamic model, where a synthetic rainfall burst is applied across a three-dimensional surface. The surface provides an accurate representation of the ground profile and is usually captured using LiDAR techniques. The hydrodynamic model predicts the flow and accumulation of runoff across this surface. A depth and or velocity depth threshold is applied to the model results. This is then mapped spatially to allow identification of those areas containing the most significant accumulations of flow. **Figure 3.6** below shows a typical pluvial flow path map prepared by a local government using a 'rain on grid' approach.

Figure 3.6: Example pluvial flow path map (yellow shaded areas) generated using a 'rain on grid' model approach.

This approach can be complex and unreliable if not carefully applied by someone with suitable experience. However if successfully applied this method is efficient and eliminates the need for human intervention to manually pre-identify flow paths over large areas.

If the area of interest is small, a more manual approach may be more practical. This could involve capture of detailed ground survey and then having a suitably experienced person inspect the data and manually estimate the location of low points and likely flow paths. Simple hydrologic and hydraulic calculations can then be applied to estimate the depth and width of the pluvial flow path at regular intervals along its course.

Regardless of the approach used to identify pluvial flow paths, caution should be used when interpreting the mapped results, as there can be significant inherent uncertainties caused by:

- obstructions to flow paths such as buildings and fences
- rapidly changing flow conditions along the flow path's course
- survey accuracy limitations
- limited opportunity for calibration

Also, where a 'rain on grid' approach has been applied, the results inherit some additional hydrologic uncertainty due to complete reliance on the hydraulic model to simulate natural physical processes of water flow. In contrast to the empirical relationships between rainfall and runoff that are used in most traditional hydrologic modelling software packages.

Once pluvial flow paths have been identified a management program can be developed. This may include:

- public flood awareness mapping
- flood education
- investigation of drainage system upgrades
- building and development controls

In most circumstances, flood warning emergency systems are inappropriate for pluvial flooding as the potential warning times are too short.

Where building and development controls are applied, they should include provisions that, if possible, either prevent the erection of new buildings within pluvial flow paths, or set minimum levels deemed safe. Other provisions may also require measures that minimise potential blockage and obstruction to flow. To assist with applying these controls to a particular site it may be necessary to undertake a more detailed site-based flood investigation to more accurately estimate flood levels and flood behaviour.

When applying a minimum level to infrastructure such as a habitable dwelling, a freeboard allowance above the calculated flood level should be applied. Freeboard is required to account for the uncertainties that are inherent in flood calculations. A typical minimum value of 0.3m above the flood surface is suggested, however this can be increased to account for local factors such as the sensitivity of the specific infrastructure to flood damage and the expected degree of uncertainty in the flood level estimates for the site. Flood level uncertainty may vary depending on a number of 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 such as risk of blockage and climate change. If these risks are a concern for the site then they should be explicitly incorporated into the base flood level estimates before freeboard is applied.

3.5.2 Fluvial Flooding (River and Creek Flooding)

Fluvial flooding, also often referred to as river and creek flooding, is generally caused by long durations (hours to days) of intense rainfall, falling across a large catchment. Typically such catchments range in area from 1 km^2 to many thousands of km^2. Excess runoff from the catchment accumulates and concentrates in creeks, rivers and lakes that have natural features such as a main channel and defined banks. Where hydraulic conveyance is most limited, runoff escapes out of the main channel and flows onto adjoining land causing the inundation of normally dry ground. Where the natural topography is flat, this flooding can occur across vast areas. Where the natural topography is incised, the flood extent can instead be quite narrow and well defined. In general fluvial flooding is easier to analyse than pluvial flooding because the channels are easier to identify and represent using computer-based models.

This type of flooding is natural. However it can cause substantial damage to infrastructure and property if proper urban planning has not been undertaken. Indeed, fluvial flooding is recognized as one of the most significant natural hazard in Australia, responsible over the long-term for a significant proportion of total loss and damage. Consequently, fluvial flooding has been the target of significant government programs involving flood hazard mapping and implementation of measures that mitigate potential loss and damage from flood.

From an urban drainage design perspective, fluvial flooding is a constraint that needs to be understood as it may heavily influence any drainage solutions proposed. Numerical methods for the estimation of flood behaviour and identification of fluvial flood hazard are well established and tested. These methods are further described in the various books and chapters of Australian Rainfall & Runoff.

The management of fluvial flood hazard differs from pluvial flooding in so far as the quantity of floodwaters is much greater and therefore more difficult to control and contain through physical changes to the floodplain. It is preferable and more cost effective to instead avoid these hazards through careful planning. This is best achieved through carefully considered strategic plans and a suite of flood related building and development controls.

Where this opportunity has been missed, and development has already occurred within inappropriate parts of the floodplain, public flood awareness mapping, flood education, flood mitigation and flood warning emergency systems become more important.

It is noted that catchments that give rise to fluvial flooding are large, therefore the lag between rainfall and runoff is often sufficient to make flood warning and emergency management more feasible.

3.5.3 The Pluvial Fluvial Interface

Although a general distinction is drawn between pluvial and fluvial flooding, in reality there exists an interface zone within all catchment floodplains where both may exist and differentiation between the two becomes subjective. For example at the location where a small gully drains directly into a major river or creek.

Figure 3.7 Example fluvial flow path map (brown shaded areas) showing interface with pluvial flowpaths from Figure 3.6

When dealing with drainage issues in the interface zone, the problem should be assessed carefully from first principles and potential analysis and management techniques drawn from both pluvial and fluvial flood perspectives.

While both types of flooding can occur simultaneously, this is unlikely since the rainfall mechanisms that typically cause each type of flood are different. Instead it is more likely that pluvial 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, since the decision to pay a claim sometimes relies upon whether the flooding was pluvial or fluvial in nature. More recently, the insurance industry has begun to also 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.

3.6 Runoff Volume Management

Historically, urban drainage design practice has focussed on peak flow management and conveyance. Design standards have now become more stringent and require more comprehensive management of the hydrologic change that is brought about by urbanisation. This has in turn driven the need to more carefully manage the volume of runoff. Typically this is achieved through the design and installation of detention basin facilities.

There are different levels of sophistication that can be employed when implementing runoff volume management. The most basic is to only manage the peak discharge from a site for a single design flood event probability, normally the flood event used for regional flood planning (e.g. 1% AEP discharge). The basic objective here is to maintain or reduce downstream flood levels for this particular event.

A more thorough approach will also include the management of peak discharge for additional more frequent flood event probabilities. The objective of this design criteria is to manage peak discharge from frequent events that can cause nuisance flooding downstream. A typical design response to this additional criteria requires detention basins with greater storage volume and more complex outlet structures including a series of hydraulic controls each tuned to a different size of flood.

However if peak discharge is the only aspect of flood behaviour that is managed, there is a likelihood that other characteristics of the changed hydrologic response, such as peak timing and flow duration, will not be adequately addressed.

Design solutions that also manage these hydrologic changes may need to be pursued for some sites. For example an acceleration in peak timing due to urbanisation may not be acceptable for downstream communities that rely on warning time for evacuation. A substantial change in flow duration may also be unacceptable from a nuisance and environmental perspective.

Solutions that address these issues are challenging and will require complex calculation. In some respects, older calculation methods and software tools are not adequate for handling the complexities of these design tasks. Iterative testing using a hydrologic model combined with professional judgement will instead be necessary.

A common limitation of the calculation methods is a reliance on single design rainfall bursts for basin performance testing. Actual patterns of rainfall will vary from the design bursts and so too will actual basin performance. For critical structures the sensitivity of basin performance to an ensemble of rainfall temporal patterns should be assessed.

When performing these calculations, the locations of interest where performance comparisons are made, should be carefully chosen and may not simply correspond to the downstream boundary of a site. For example, if there is a flood-prone community further downstream from the site, this may become a critical benchmark location to use for hydrograph comparison.

For large sites where a number of detention basins are proposed the spatial distribution of these facilities needs careful planning. Basins positioned in series along a single valley should be avoided as the upper basins will limit the effectiveness of the lower basins. This solution may also increase risk in the event of a basin failure and or an extreme overtopping event.

3.7 Continuous Improvement

There is a need to allow changes in interpretation of the stormwater components of ARR to accommodate contemporary and integrated approaches to urban water cycle management which starts with the integration of land and water planning across time horizons and spatial scales. This guidance should encompass advances in urban water cycle management, and must be cognisant for the likely advances in science and professional practice over the next 30 years.

There is an enabling framework of guidance in ARR that encourages and permits advanced analysis techniques and innovative designs. The guidance in ARR must not 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 appropriate. 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.

An appropriate policy framework is also required that integrates 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. Appropriate design methods for integrated solutions are likely to include most of 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.

The guidance in ARR must be linked with Australian Runoff Quality (ARQ) and other quality guidelines so that urban drainage is an integrated part of the urban water cycle and avoid duplication of infrastructure. An integrated approach to stormwater management should also avoid installation of infrastructure to meet separate objectives that, in combination, create unexpected diminished performance.

This edition of ARR highlights the need to consider integrated approaches for future urban water management means that our current approaches of separate analyses of water quantity, water quality, drinking water and wastewater systems are no longer the best approach. Integrated systems have the capacity to produce solutions that respond to multiple objectives including economic, social and environmental criteria.

ARR promotes methods that bring these elements together in a combined analysis approach. It is expected that this will require strong leadership from the water industry and a recognition of the need to collaborate across science, engineering, planning and sociological sectors in order to maximise the opportunities for implementing integrated solutions.

3.8 Acknowledgements

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3.9 References

Argue, J. R., (2004). Water Sensitive Urban Design: basic procedures for ‘source control’ of storm water – a handbook for Australian practice. Urban Water Resources Centre, University of South Australia, Adelaide, South Australia, in collaboration with Stormwater Industry Association and Australian Water Association. ISBN 1 920927 18 2, Adelaide.

Armstrong, J.W., (1986). Pipelines and People, Halstead Press, Sydney.

Aishett E., and E. Steinhauser, (2011). Does anybody give a dam. The importance of public awareness for urban water conservation during drought. Submission by the Australian National University to the Productivity Commission.

Barton A. F., P. J. Coombes, A. Sharma, (2007). Impacts of innovative WSUD intervention strategies on infrastructure deterioration and evolving urban form. Rainwater and Urban Design Conference 2007, Engineers Australia, Sydney.

Brown R. R. and M. Farelly,. (2007). Institutional Impediments to Advancing Sustainable Urban Water Management: a Typology. International Rainwater Catchment Systems Conference and the 5th International Water Sensitive Urban Design Conference, Sydney, Australia.

Clarke R.D.S., (1990). Asset replacement: can we get it right? Water. 22-24

Coombes P.J., and M.E Barry (2014). A systems framework of big data driving policy making –

Melbourne's water future. OzWater14 Conference. Australian Water Association. Brisbane.

Coombes P. J., and M. E. Barry, (2008). The relative efficiency of water supply catchments and rainwater tanks in cities subject to variable climate and the potential for climate change, Australian Journal of Water Resources, 12: 85-100.

Coombes P.J. and M. E. Barry, (2008). Determination of available storage in rainwater tanks prior to storm events. Hydrology and Water Resources Symposium. Engineers Australia. Adelaide.

Coombes P. J., (2005). Integrated Water Cycle Management: Analysis of Resource Security. Water, 32: 21-26.

Coombes P.J., J. R. Argue, G.A. Kuczera, (2000). Figtree Place: a case study in water sensitive urban development (WSUD), Urban Water, 1(4): 335-343.

Daniell K.A., P.J. Coombes and I. White, (2014). Politics of innovation in multi-level water governance systems. Journal of Hydrology. 519(C): 2415-2435.

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

IEAust., (1987). Australian Rainfall and Runoff: a guide to flood estimation. Volume 1. Editor in Chief: D.H. Pilgrim. The Institution of Engineers Australia. Canberra.

IEAust., (1977). Australian Rainfall and Runoff: flood analysis and design. The Institution of Engineers Australia. Canberra.

IEAust., (1958). Australian Rainfall and Runoff: first report of the stormwater standards. The Institution of Engineers Australia. Canberra.

Kuczera G. A., M. Lambert, T. Heneker, S. Jennings, A. Frost, P.J. Coombes, (2006). Joint probability
and design storms at the crossroads. Australian Journal of Water Resources, 10: 63-79.

Lloyd, C.J., P. Troy and S. Schreiner, (1992). For the Public Health, Longman Cheshire, Melbourne

Lloyd S.D., T.H.F. Wong and B. Porter, (2002). The planning and construction of an urban stormwater management scheme. Water Science and Technology. International Water Association. Vol. 45, No. 7, pp 1-10.

McAlister T., P.J. Coombes and M.E. Barry, (2004). Recent South East Queensland Developments in Integrated Water Cycle Management - Going Beyond WSUD. Cities as Catchments:

WSU D2004, Adelaide, SA, Australia.

Mitchell, V.G., T.A. McMahon and R.G. Mein, (2003). Components of the total water balance of an urban catchment. Environmental Management, 32, 735-746.

PMSEIC., (2007). Water for Our Cities: building resilience in a climate of uncertainty. A report of the Prime Minister's Science, Engineering and Innovation Council working group. Australian Government. Canberra.

Pezzaniti D., J.R.Argue and L. Johnston, (2002). Detention/retention storages for peak flow reduction in urban catchments: effects of spatial deployment of storages. Australian Journal of Water Resources. 7:2, 1-7.

Poelsma, P.J., Fletcher, T.D., and Burns, M.J. (2013). Restoring natural flow regimes: the importance of multiple scales. Proceedings of Novatech Conference, Lyon, France, 23-27.

Victorian Government (2013). Melbourne's Water Future.

Walsh C.J. (2004) Protection of in-stream biota from urban impacts: minimise catchment imperviousness or improve drainage design? Marine and Freshwater Research 55:317-326.

Weinmann E., (2007). Hydrology and Water Resources: the challenge of finding the water balance. Australian Journal of Water Resources. 11:2, 121-132.

Whelans and Halpern Glick Maunsell (1994). Planning and Management Guidelines for Water Sensitive Urban (Residential) Design, report prepared for the Department of Planning and

Urban Development the Water Authority of Western Australia and the Environmental Protection Authority.

Wong T.H.F., (2006). Water Sensitive Urban Design: the journey thus far. Australian Journal of Water Resources. 10:3, 213-221.

Wong, T.H.F., D. Knights and S.D. Lloyd, (2008). Hydrologic, water quality and geomorphic indicators of catchment Effective Imperviousness, Australian Journal of Water Resources, 12 (2), Engineers Australia, 111-119.

Chapter 4
Modelling Approaches

Chapter Status	
Book	9
Chapter	4
Date	27/11/2015
Content	In preparation
Graphs and Figures	In preparation
Examples	In preparation
General	

Chapter 5
Detention and Retention

Brett C Phillips, Marlène van der Sterren, John Argue

Chapter Status	
Book	9
Chapter	5
Date	27/11/2015
Content	Working draft
Graphs and Figures	Working draft
Examples	Working draft
General	Need to integrate with other chapters and check consistency

5.1 Introduction

The impact of urbanisation on the frequency of runoff events, the rate and volume of runoff as described in Book 9 Chapter 2 are managed through an hierarchy of stormwater management (ANZECC, 2000). These practices include:

- Source Controls (non-structural measures): these are actions to limit changes to the quantity and quality of stormwater at the source;
- Source Controls (structural measures): these are measures constructed at or near to the source to manage stormwater quantity and quality of stormwater;
- In-system management measures: these are measures constructed within stormwater systems to manage the quantity and quality of stormwater prior to discharge to receiving waters.

The structural Source Control and In-system management measures include stormwater detention and stormwater retention which are defined by Argue (1986) as follows:

> **Detention** is defined as holding of runoff for short periods to reduce peak flowrates and releasing the stored volume in a controlled manner to the natural or artificial watercourses to continue its path in the hydrological cycle. Any reduction in the volume of surface runoff involved in this process is minimal and therefore the reduction in volume is considered to be nil;
>
> **Retention** is defined as the procedures and schemes whereby stormwater is held for considerable periods causing water to continue in the hydrological cycle via infiltration, percolation, evapotranspiration, and reuse and only the overflows are discharged directly to the natural or artificial watercourses. The volume of surface runoff is reduced.

Source control measures can include On-Site Detention (OSD) for the control of peak flowrates and On-Site Retention (OSR) for the control of peak flowrates and runoff volume. Both OSD and OSR can reduce stormwater pollutant loads discharged to receiving waters, however, OSR has been shown to be more efficient in removing the pollutants in stormwater than OSD. For further information on urban stormwater pollutant generation and control, the reader is referred to the Australian Runoff Quality guidelines (Engineers Australia, 2006). OSD and OSR are typically small stormwater storages installed on individual residential, commercial and industrial lots and are considered off-line in relation to the council or public drainage system.

In-system management measures can include community and regional detention and retention measures. Community measures are typically medium sized stormwater storage facilities constructed in public areas, including public open space. Generally the community structural detention and retention systems are combined with other community uses such as public sporting grounds, recreational areas and parks and other community facilities (e.g. libraries, community halls). Community measures can be off-line in relation to trunk drainage lines, but may be on-line in relation to local drainage lines.

Regional measures are typically large community storage facilities constructed on-line in the downstream reach of a catchment near to the receiving water.

5.2 Catchment Detention and Retention Strategies

Catchment wide planning of volume, frequency and rate of runoff should be undertaken to assist with the design and assessment of source controls and in-system management measures. The impact onto the receiving waters can be worsened if catchment–wide assessments are not conducted through compounding peak discharges from different sub-catchments and increased duration of flows in ephemeral aquatic ecosystems. This can be a challenge when a catchment drains multiple Local Government Areas (LGAs) particularly if there is a perceived or actual unwillingness by one or more Councils to adopt a consistent approach to controlling volumes, frequencies and rates of runoff across LGA boundaries.

Four management strategies for catchment-wide management which are consistent with the risk management framework discussed in Book 1 Chapter 5 are recommended and are defined as follows (Argue, 2004/2013).

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

For example, all stormwater available in the catchment is collected and conveyed to a wetland or similar facility where it is cleansed prior to injection into an aquifer as part of an Aquifer Storage Recovery (ASR) scheme. The potential for flooding in the wetland vicinity must be recognised and steps taken such as use of stormwater detention techniques to ensure acceptable performance.

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 system.

For example, each development site (including redevelopment sites) provides a stormwater retention facility with storage capacity equal to the difference between the developed site runoff volume and its pre-development equivalent in the design storm of (catchment) critical duration.

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.

For example, each development site (or redevelopment site) provides a stormwater retention facility with storage capacity equal to the runoff volume generated on that site in the design storm of (catchment) critical duration.

Infrastructure Compliant Stormwater Management (ICSM): maintain or improve the performance of existing conveyance infrastructure, through controlling flows from each development site. This strategy is most suitable for an infrastructure centred council with capacity issues related to the minor / major system in the council area.

For example, each development site (or redevelopment site) ensures that peak flows in the adopted design event do not exceed the estimated flow capacities of existing infrastructure.

When undertaking a catchment-wide assessment for detention or retention the same basic risk approach (as discussed in Book x) is adopted as follows:

1. Identify benchmark locations within a catchment and the current state of the receiving system;
2. Determine peak flows for a range of AEPs and storm bursts and/or design storms at benchmark locations, as per the guidance in Book 1;
3. Decide on a benchmark year (which equates to a benchmark catchment condition which is informed by the concerns and priorities of the wider community and the need to ensure the sustainability of receiving waters as a resource which are valued for their environmental, social and economic uses);
4. Identify the critical storm burst(s) and/or storm duration(s) spatially throughout a catchment, as per the guidance in Book 1;
5. Identify the future conditions of the catchment on which the assessment is to be based (which could represent new development in a non-urban catchment, further urbanisation in a partially developed catchment or redevelopment in an existing fully developed catchment) and its impact on the receiving system(s);
6. Undertake (iterative) assessments to identify the required sizes and configurations of source control and/or in-system management measures that mitigate the risk and flows to the acceptable level identified.

Councils will need to 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), as discussed by Phillips and Yu, 2015. If a Council or authority adopts a storm burst only approach then for a given AEP the peak flows are assessed for a range of storm burst duration and the storm burst duration which gives the highest peak flow is adopted. If a Council or authority adopts an ensemble of complete storms of a given AEP, then the authority or Council will need to decide if the benchmark condition is to be based on the 50th percentile peak flow or on a different percentile of peak flow. 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.

Catchment-wide assessments have been conducted at various scales. An example of a catchment based modelling to determine the required size and configuration of OSD on individual lots across a 110 km^2 urban catchment was undertaken to inform the fourth edition of the OSD Handbook for the upper Parramatta River catchment (UPRCT, 2005). This assessment and the resulting sizing and configuration of the OSD systems are outlined in **Section 5.9.1**.

Another example is the catchment modelling which was conducted for a selected catchment in the Hawkesbury LGA in NSW to assess the reduction in peaks and velocities which could be achieved when rainwater tanks are installed in the catchment (van der Sterren, 2012, 2014). The reduction in flows in the catchment model were shown to be significant confirming the findings from other researchers using hypothetical catchment models and other OSD design methods (Argue 1997, 1998, 2004, Argue and Scott, 2000, Coombes and Barry 2008, Coombes et al., 2001, Coombes et al., 2002a, 2002b). The model was developed with a large amount of detail, increasing the complexity of the model (Cantone et al. 2008, Dembélé et al. 2010). The catchment model was also calibrated, validated and critically analysed (van der Sterren, 2012). Overall, it was found that the implementation of rainwater tanks can assist with reducing local flooding in the selected catchment and could assist in controlling discharges from other upstream developments. This highlighted the value of a catchment wide assessment of proposed design requirements for the catchment, due to the cumulative effect of rainwater tanks.

There are four classes of source control measures that can be used to adopt the different strategies in the catchment. These four classes are identified as follows (as per *WSUD: Basic Procedures for 'Source Control' of Stormwater* (Argue, 2004/2013)):

- Category 1: Measures whose primary role is in flood control;
- Category 2: Measures whose primary role is in pollution control;
- Category 3: Measures whose primary role is in stormwater harvesting; and,
- Category 4: Measures with multi-objective roles in relation to flood control, pollution control, stormwater harvesting and/or amenity.

Category 1 measures primarily control flooding in major systems. Category 3 systems in addition to the primary aim of harvesting, can have a significant and positive impact on minor system flood control. Category 4 measures could include an integrated suite of Category 1, 2 and/or 3 measures to achieve multiple objectives. Best practice urban stormwater drainage ultimate outcome would be to adopt category 4 systems that consider all aspects of urban stormwater management in a holistic and integrated manner. This holistic approach to stormwater management in urban areas is directly linked to the concepts and ideas discussed in Chapter 2 (Ladson) and Chapter 3 (Coombes) of this Book.

Detailed step by step assessment approaches for Category 1, 2 and 3 systems are also given in *WSUD: Basic Procedures for 'Source Control' of Stormwater* (Argue, 2004/2013). The approaches detailed in this Source Control guideline can be readily adapted to continuous modelling or a selection of complete storm events..

5.3 On-Site Detention

5.3.1 Introduction

In many urban areas detention has been implemented, and in particular since 1975 the use of detention basins has been widespread in NSW (IEAust, 1985). However in urbanised areas the available sites for major community and/or regional detention basins are limited or are fully utilised over time. To avoid exacerbating what can be already substantial flooding problems in an urbanised catchment by further increasing development or redevelopment, planning and development controls are implemented to mitigate the impact of the (re) developments. OSD controls are implemented for individual development to manage this (O'Loughlin et al., 1995, UPRCT, 2005). 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).

5.3.2 Available Guidelines

There are many guidelines on the sizing and/or design of OSD, such as Department of Irrigation and Drainage (2000), Derwent Estuary Program (2012), Hobart City Council (2006) and UPRCT (2005). These guidelines can be readily used for designing and modelling OSD systems, using the modelling approaches outlined in Book 7.

There are many examples of OSD controls incorporated in Council Development Control Plans including: Bankstown City Council (2006), Banyule City Council (2012), Blacktown City Council (2005, 2006), Blue Mountains City Council (2005), City of Casey (2013), Hawkesbury City Council (2000), Hills Shire Council (2010), Holroyd City Council (2003), Kogarah City Council (2006), Ku-ring-gai Council (2005), Manningham City Council (2003), Melton Shire Council (2009), Parramatta City Council (2005), Penrith City Council (2010) and Tea Tree Gully Council (2013). These documents can assist in the design of OSD systems, however, practitioners are encouraged to determine if the method identified in the guidelines are consistent and/ or suitable for using the contemporary flood estimation techniques identified in Book 5.

Historically, the primary objective of OSD controls was to manage flooding in a 1% AEP event only. In contrast, the fourth edition of the OSD Handbook for the upper Parramatta River catchment details the sizing and design of OSD systems for lots located in a 110 km^2 urban catchment which limit peak flows in the 50% AEP and 1% AEP events (UPRCT, 2005). These OSD requirements are outlined in **Section 5.9.1**. Further implementation and development on OSD has resulted in many Councils now requiring OSD systems to reduce the post development flows to adopted benchmark peak discharges over a range of AEPs. It is expected that future catchment-wide assessments as per **Section 5.2** above, will require more detailed design requirements for OSD systems based on the selected management regime.

In the absence of local guidelines or when formulating OSD guidelines or Council requirements the following elements on an OSD system need to be considered.

- Design procedures based on control requirements - These should be determined using catchment-based assessment of lot-based measures as described in Section 5.2 and demonstrated in Section 5.9.1;

- Storage types, such as above ground storage, landscaped storage, below ground storage, modular storage, and combined above and below ground storage;
- Ponding depths in any above ground storages; and
- Provisions for maintenance.

5.3.3 Design Procedures

The primary objective of OSD is to control peak flows downstream of the subject site. It is in essence a Category 1 system and its discharge control requirements should be based on a catchment wide assessment, as shown in Section 5.2. A catchment wide assessment can be 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 many Councils now require OSD systems to reduce the post development flows to adopted benchmark peak discharges over a range of AEPs. This requires frequency stage storages and outlets and multiple PSD and SSR values.

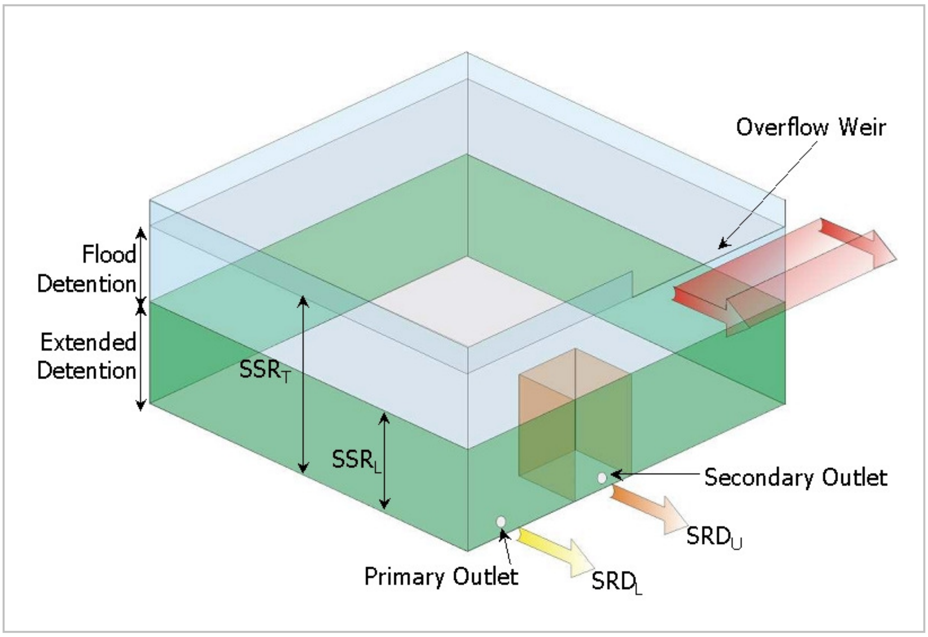


Figure 5.1 Frequency Staged Below Ground OSD System (after UPRCT, 2005)

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

1. Adopt the SRD and SSR for the Upper Parramatta River Catchment catchment (UPRCT, 2005)
2. 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 (a regime in balance strategy).
3. 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 ie. the PSD (under an Infrastructure Compliant Stormwater Management strategy).

Any 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.

Example multi-stage discharge requirements

In the case of development in the upper Parramatta River catchment, the SRD for the primary (lower) orifice outlet (SRD_L) is 40 L/s/ha. The SRD for the secondary orifice outlet (SRD_U) in the discharge control pit is 150 L/s/ha (see Figure 5.1). This is adjusted when the entire site cannot be drained to the storage as shown in the procedures set out in UPRCT (2005).

The effect of catchment scale was also investigated by determining the OSD parameters that would apply as the catchment area is progressively reduced. In all cases it was assumed that the primary SRD (SRD_L) and secondary SRD (SRD_U) remained constant at 40 L/s/ha and 150 L/s/ha respectively. The resulting values for SSR_L and SSR_T are summarised in **Table 5.1**.

In all cases it was assumed that all roof runoff is directed to the OSD storage. It was found that for catchments greater than around 2,000 ha (20 km^2) that there would be a negligible reduction in the extended detention (SSR_L) and overall detention volume (SSR_T) values.

Table 5.1 Adjusted Control requirements as a Function of Catchment Area
(adapted from URCT, 2005)

Catchment Area (ha)	SRD_L (L/s/ha)	SSR_L (m^3/ha)	SRD_U (L/s/ha)	SSR_T (m^3/ha)
100	40	190	150	334
200	40	230	150	378
400	40	260	150	413
600	40	274	150	428
800	40	283	150	437
1,000	40	288	150	443
1,200	40	292	150	447
1,500	40	295	150	451
2,000	40	299	150	454
> 2,000	40	300	150	455

In the UPRCT guideline, the overall (total) SSR for an OSD storage (SSR_T) is 455 m^3/ha. The SSR for the OSD storage is partitioned into extended detention (lower) and flood detention (upper) storages. The SSR for the extended detention storage is 300 m^3/ha. The SSR is adjusted when a rainwater tank is included in the development / redevelopment and an airspace "credit" is claimed to partially offset the SSR (refer **Section 5.9.1**) (UPRCT, 2005).

5.3.4 OSD Storage 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 Storages

The main types of above-ground storages include landscaped storages, parking and paved storages, and rainwater tanks with dedicated airspace for detention. The recommended maximum storage depths for different above-ground storage types are given in **Table 5.2**. It is recommended that a landscape architect is consulted early in the development to ensure consistent objectives and suitable plant selections for the OSD system. In any case, 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.

Table 5.2 Recommended Maximum Storage Depths for Above-Ground Storage

Storage Type	Maximum Ponding Depth
Landscaped areas	600 mm
Private courtyards	600 mm
Parking areas and driveways	150 mm
Pedestrian areas	50 mm
Paved outdoor recreation areas	100 mm

Source: 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 exceeds 600 mm or adjacent to pedestrian traffic areas.

Landscaped Storages

When surface storage is provided by landscaped areas, such as lawns and garden beds then typical design requirements for landscaped storages include:

- **Structural Adequacy -** Design of retaining walls should consider the structural safety aspects such as the need for fencing or steps or a ladder, both when the storage is full and empty;

- **Storage Configuration:**
  - Ponding depths shall not exceed the maximum storage depth requirements recommended in **Table 5.2** however where ponding occurs in vegetated system for recreational purposes (e.g. a playground) suitable velocity and depth should be selected to ensure the safety of children and the elderly (refer **Chapter8**);
  - 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 an small energy dissipation device to reduce velocity and prevent erosion of the basin floor;
- **Overflow-** An overflow should direct the flows to the legal point of discharge in a controlled and safe manner;
- **Access -** The maximum side slope should be 1(V):6(H) where possible to permit an easy access;
- **Freeboard** - There should be a freeboard to habitable floor levels.
- **Safety:**
  - Balustrades (fences) must comply with the Building Code of Australia (See 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) (ie. 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**
  - Where ponding in frequently creates 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;

Parking and Paved Storages

When surface storage is provided in car parks, driveway, and on other paved surfaces then the maximum ponding depths are as recommended in **Table 5.2**. The frequency and duration of ponding also needs to be considered. If ponding causes inconvenience to property owners or others then a frequency staged storage approach should be considered. There should be a freeboard between the maximum ponded water level and the garage floor level / habitable floor level.

Below Ground Storages

Storage Tanks

Below ground 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.

When designing below ground tanks then typical design considerations include (Department of Irrigation and Drainage, 2000, 2012):

- **Structural adequacy** – tanks 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.
- **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 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 between 1% - 10%.
- **Vegetation and soils –** the soils and their impacts on concrete structure should be assessed to ensure that the underground tank is suitable in soils of the hazard category on the site, such as saline soils.
- **Ventilation** – an important consideration for below-ground storage systems is ventilation to minimise odour problems. Ventilation may be provided through the tank 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 tank 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
- **Access Openings** - 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 tank is deeper than 1200 mm.
- **Freeboard** – a minimum freeboard of 300 mm should be provided to the habitable floor level.
- **Safety –** a suitable amount of access hatches should be provided to enable contractors to readily adopt working in confined spaces techniques and equipment.
- **Frequency – no constraints.**

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. **Figure 5.2** shows an example configuration of a below ground storage tank.

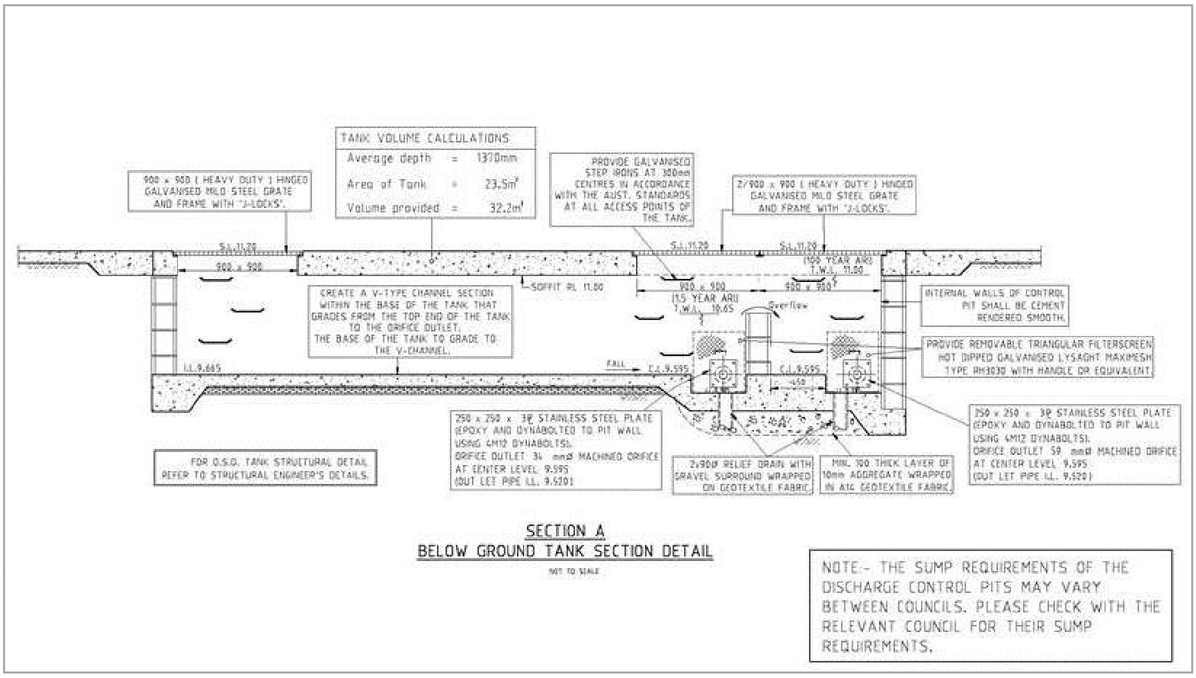


Figure 5.2 Example Below Ground Tank Layout (after UPRCT, 2005)

Underground OSD through Modular Storage

Water 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, 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.

The storage proportions for designing a composite above and below ground storage system using a frequency staged storage approach are presented in **Table 5.3**.

Table 5.3 Suggested Flood Frequency for Storage Areas (adapted from URCT, 2005)

Storage Area	Suggested Depth	Frequency of Inundation
Pedestrian areas	Beginning to pond 50 mm	5% AEP 1% AEP
Parking & driveways	Beginning to pond 100 mm 200 mm	10% AEP 5% AEP 1% AEP
Gardens	Beginning to pond 200 mm 400 mm 600 mm	1 EY 0.5 EY 10% AEP 1% AEP
Private courtyards (where area is between 25 m^2 - 60 m^2)	Beginning to pond 300 mm 600 mm	0.2 EY 5% AEP 1% AEP
Paved recreation in common areas	Beginning to pond	6 EY

5.3.5 Other Design Considerations

Outlets

The outflows from OSD systems are typically controlled by orifices. Details on the hydraulics of orifices are discussed in Book 6 and are also provided by Steward (1908); Medaugh and Johnson (1940); Lea (1942); Brater et al. (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.

Internal drainage system

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 storms up to and including the 1% AEP event.

Signage

Small OSD signs should be located in or near the OSD facility to alert future owners of their obligations to maintain the facility (refer Figure N3 in UPRCT, 2005).

OSD Warning Signs should be installed for OSD systems where deemed necessary by a Council, because of the depth and/or location of the storage (refer Figure N1 in UPRCT, 2005).

Signs are required at each entry into confined spaces, such as deep pits or below ground storages.

5.3.6 Operations and Maintenance

OSD systems are intended to regulate flows over the entire life of a development. Generally gross pollutants will be captured in an OSD system, because of the screen protecting the orifice outlet from blocking or due to the size of the outlet (Goyen, et al., 2002, Nicholas, 1995, Nicholas and Cooper, 1984). The pollutants need to be removed and therefore creates the need for continuous maintenance (Finnemore and Lynard, 1982, Urbonas and Glidden, 1983, O'Loughlin et al. 1995, van der Sterren, 2014).

Attempts have been made to resolve historical issues with the lack of maintenance of OSD systems (O'Loughlin et al., 1995). One solution is the requirement that a maintenance schedule be submitted with a Construction Certificate or Occupation Certificate application. However, this does not guarantee that the system remains fully functional as it requires owners to routinely clean and maintain their OSD systems. Some of the issues arising for an owner cleaning their confined or underground OSD systems are that WorkCover authorities may require the cleaner to have gas testing equipment, twelve volt safety lighting and protective clothing (Smith, 1994). This has resulted in plumbing contractors offering this type of maintenance service. In addition, there may be limited resources within councils to enable compliance officers to inspect OSD systems regularly to ensure that the systems are maintained in accordance with maintenance schedules. Owners or tenants may inadvertently remove the orifice plate from the discharge control pit resulting in the system being ineffective (Nicholas and Cooper, 1984). This may occur when the proper use of the system has not been explained and the orifice is removed to address sudden ‘flooding’ of their property.

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 (UPRCT, 2005). The following points are suggestions. Site constraints will mean that they will not always be feasible (UPRCT, 2005).

- Surface storages are generally easier to maintain and should be specified where possible. If extended detention storage is to be provided on the surface then it should be in little used areas where inundation will not cause amenity problems;
- Attempt to locate access points to underground storages away from heavily trafficked areas. Manholes in the entrance driveway to a large villa and townhouse development can discourage property owners from regularly inspecting and maintaining a system;
- Use light duty covers and consider locating access points in areas not subject to vehicle traffic;
- Try to locate outlet(s) in an accessible location, often a slight regrading of the storage floor will allow a designer to move an outlet from a private courtyard into a common open space. Common areas are more readily accessible to council inspectors or persons doing maintenance and help ensure the responsibility for maintenance lies with the joint owners rather than an individual;
- Every attempt should be made to locate primary storage in common open space, because this is the storage most frequently filled and hence most likely to need maintenance;
- Manholes should be fitted with the same industry-standard lifting/keying system throughout a project to assist future property owners to replace missing keys;
- Consider using circular manholes, as they are often easier to remove and more difficult to drop into the storage when being replaced; and

- Use a guide channel inside the storage or discharge control pit to fix the screen and put a handle on the screen to assist removal. The guide channel prevents debris from being forced between the wall of the pit and the screen, and allows the screen to be easily removed and replaced in the correct position.

5.3.7 Tradeable OSD Permits

In 1993 a catchment based assessment was undertaken to initiate consideration of various practical issues involved in the application of tradeable permits in an OSD policy (Willing & Partners, 1993). To date this appears to be the only assessment undertaken in Australia of tradeable OSD permits. It predates the release of the fourth edition of the UPRCT On-Site Detention Handbook in 2005.

A key goal of the former UPRCT was to ensure new developments and redevelopments do not add to existing flooding and drainage problems within the upper Parramatta River catchment (UPRCT, 1990, 1991). The first edition of the UPRCT On-Site Detention Handbook defined SSR and PSD requirements for new development and redevelopment sites (UPRCT, 1991). The OSD policy, in effect, established a permit associated with each property.

Under a tradeable permit system, the owner of a development property which could discharge at less than the PSD could sell the unused discharge capacity to a nearby developer unable to economically limit discharges to his PSD. For example, a development property owner may be able to limit the discharge to 60 L/s/ha, whereas another developer can achieve 100 L/s/ha, but finds it very difficult and expensive to reduce the discharges to say a target 80 L/s/ha. Trading the spare 20 L/s/ha would be in the interests of both the buyer and the seller and, since an overall PSD of 80 L/s/ha is maintained and the OSD policy is not compromised. The OSD policy becomes made more flexible and thus more acceptable. Overall costs of complying with the policy are minimised.

An analytical review of the possible trading of PSDs in the *upstream* direction along a watercourse concluded that this direction of trading would not conform to the OSD policy. Conversely, trading of PSDs in the *downstream* direction would in principle conform to the OSD policy subject to potential distance limitations.

Based on the results of the various investigations and the range of conditions that exist within the catchment the following global guidelines for trading of PSDs within the upper Parramatta River catchment were determined from the investigations (Willing & Partners, 1993):

(i) Trading of PSDs can *only* occur in the *downstream* direction between subcatchments on the same "watercourse" ie. between a subcatchment which receives flows directly from the upstream subcatchment via a channel or watercourse;

(ii) Trading of PSDs may occur between subcatchments located no more than 5 km apart subject to any further investigations of specific creeks and drains which may determine a lower trading distance; and

(iii) Trading of PSDs not be permitted "across" detention basins ie. PSD on lot upstream of a detention basin cannot be traded to a lot located downstream of the detention basin.

5.4 ON-SITE RETENTION

5.4.1 Introduction

Since 1997 the use of retention measures has been promoted by researchers and practitioners including Joliffe, (1997), Andoh and Declerck (1999) Joliffe and Fryar (2000), Argue (1997, 1998, 2001, 2014), Argue and Scott (2000), Argue et al (2003, 2010), Argue and Pezzaniti (2007, 2009, 2010, 2012), Coombes et al. (2000, 2001, 2002), Coombes and Kuczera (2003), Coombes, Frost et al (2002), Coombes, P.J. and Barry, M.E. (2008), Kuczera (2008), Hewa et al (2009), Lucas and Coombes (2009), Bishop et al (2013) and van der Sterren (2013, 2014).

Herrmann and Schmida (1999), Andoh and Declerck (1999), Argue and Scott (2000) and Vaes and Berlamont (2001) discuss that the reduction of peak discharge on lot scale with retention is not significant. Argue and Scott (2000) indicated that with a large catchment scale model, OSD systems and rainwater tanks produce a similar hydrographs. While it is acknowledged that the peak discharge on a lot scale is larger for rainwater tanks than for OSD systems in "*medium large catchments the cumulative effect of volume reductions, under OSR, obliterates the effect of high peak discharges delivered by individual sites*" (Argue and Scott, 2000). This indicates that the cumulative effect of OSR can be significant on a catchment scale, due to the reduction in overall volume discharged.

5.4.2 Available Guidelines

There are many guidelines for OSR such as Argue, J R (Ed, 2004/2013), Department of Irrigation and Drainage (2000), Department of Water, Western Australia (2007), Derwent Estuary Program (2012), Government of South Australia (2010), Hobart City Council (2006), Melbourne Water (2005), Planning SA (2003), Planning SA (2003), UPRCT (2004) and Department of Planning and Environment (2015). These guidelines can be readily used for designing and modelling OSR systems, using the modelling and storm patterns as described in Book 2 and Book 7. In the absence of local guidelines or when formulating OSR guidelines or Council requirements the design procedures discussed in this chapter can be considered.

5.4.3 Design Procedures

The primary objective of OSR is to reduce the volume of discharge from a subject site. It is in essence a Category 3 system and its discharge control requirements should be based on a catchment wide assessment, as shown in section 5.2. A catchment wide assessment can be downscaled to site control requirements, such as:

- SSR – Site Storage Requirement with a PSD or SRD
- The post-development volumes of the subject site controlled to meet the pre-development volumes for the site for a range of complete storms (a "regime in balance" strategy).

Ten basic design procedures are given in *WSUD: Basic Procedures for 'Source Control' of Stormwater* (Argue, 2004/2013) for retention systems. Procedures 1-4 inclusive are applicable, primarily, to Category 1 installations; Procedures 5 - 8, to control frequent flows; and Procedures 9 and 10 relate to rainwater (domestic or industrial) harvesting systems. Most of these procedures have also been outlined in Melbourne Water (2005) and Engineers Australia (2006), however, for these and the Source Control procedures by Argue (2004/2103) it is important to ensure that a range of frequencies and magnitudes are considered, including emptying time. In addition, it is critical that these systems are designed to convey all flows up to including the 1% AEP safely through a site.

The ten procedures are summarised as follows.

Procedure 1 design formulae which enable the plan area required for an infiltration surface to be determined, matched to the 'peak (design) flow' generated in a contributing catchment. Two cases are detailed - cases where the infiltration surface is external to the catchment (case 1A) and cases where the infiltration surface is within the catchment (case 1B). For example: permeable pavement, biofiltration systems, filtration trench.

Procedure 2 design formulae which enable the dimensions of in-ground stormwater retention devices to be determined, matched to the (design) runoff volume generated in a contributing catchment. Three cases are detailed - "leaky wells (case 2A), infiltration trenches filled with gravel, gravel/pipe combinations or heavy duty plastic 'milk crate' cells (case 2B) or "soakaways" or mattress (trench) installations (case 2C). For example: soakaways, biofiltration systems, filtration trenches.

Procedure 3 design formulae which enables the dimensions (plan area) of infiltration or 'dry' ponds to be determined and matched to the (design) runoff volumes; For example: OSD basins in an above ground storage, large scale detention basins with infiltration potential.

Procedure 4 a three-step process which is a continuation of Procedures 2 and 3, above to be applied in cases where a design fails to meet a required emptying time criterion. Two cases are detailed - situations where aquifer access is available (case 4A) or situations where access to street drainage or a local waterway is available (case 4B). For example: Aquifer recharge, infiltration and reuse, lined biofiltration systems with discharge to council system.

Procedure 5 a multi-step process to determine the dimensions of in-ground "soakaway" reservoirs storing cleansed stormwater runoff. Two cases are detailed - systems dimensioned according to the outcomes of 'continuous simulation' modelling to temporarily store 95% of annual runoff anywhere in Australia (five Australian climatic regions) (case 5A) or systems matched to 'first flush' runoff (case 5B); For example: infiltration and reuse, lined biofiltration systems with discharge to council system.

Procedure 6 a multi-step process which is a continuation of Procedure 5 to be applied in cases where a design fails to meet a required emptying time criteria. Two cases are detailed - situations where there is access to an aquifer (case 6A) or situations where access is available to street drainage or a local waterway (case 6B); For example: Aquifer recharge, infiltration and reuse, lined biofiltration systems with discharge to council system.

Procedure 7 a multi-step process specific to the design of swale streetscape systems including in-ground "leaky" trenches. The system objective is to treat and temporarily store more than 95% of annual runoff eg. swales, biofiltration swales.

Procedure 8 a multi-step process which is a continuation of Procedure 7, above, to be applied in cases where a design fails to meet a required emptying time criterion. Two cases are detailed - situations where there is access to an aquifer access (case 8A) or situations where access is available to street drainage or a local waterway (case 8B) eg. lined biofiltration swale with a connection to the council drainage system.

Procedure 9 a multi-step process using daily rainfall records, a range of domestic roof areas and average household water use data to determine rainwater tank capacities to meet acceptable "failure to supply" criteria eg. rainwater tanks on a lot scale..

Procedure 10 a multi-step·process using continuous simulation modelling to determine the size of rainwater tank needed to give between 30% and 90% harvesting of roof area runoff anywhere in Australia (five Australian climate regions) eg. rainwater tanks on a lot scale for large scale developments.

Argue (2004/2013) indicates that OSR devices relying on infiltration can be employed in all soil categories including:

- Deep, confined or unconfined sands (homogeneous);
- Sandy clays (homogeneous);
- Medium clay soils (homogeneous);
- Heavy clay soils (homogeneous);
- Constructed clay soils; and
- Sites with rock or shallow soil cover over rock.

The key to these designs, is to assess how they perform their retention function. This can be either using simple systems or through accelerated emptying between successive storms. Argue (2004/2013) states that a soil hydraulic conductivity value of 1 x 10^{-6} m/s (which separates medium clay soils from those soils described as heavy) separates simple solutions (for soils with hydraulic conductivity values > 1 x 10^{-6} m/s) from those requiring more complex design approaches (for soils with hydraulic conductivity values < 1 x 10^{-6} m/s).

In the absence of or when formulating local OSR guidelines or Council requirements the following guidelines, which are based on the existing published council and government department guidelines should be considered.

5.4.4 Rainwater Storage or Stormwater Reuse Systems

While rainwater tanks have primarily been used to provide an alternative water supply source and reduce scheme water consumption in areas of limited infiltration (high water table, clay soils) rainwater tanks can also reduce catchment runoff (peak flow and volumes) in smaller rain events.

Gutter guards, first flush devices and filter socks can limit the transfer of sediment and debris to rainwater storage systems. Mesh screens on inlets, outlets and overflow devices will exclude animals and mosquitoes and other insects from entering tanks, therefore minimising the risk of harmful microorganisms and disease-carrying mosquitoes entering the tanks.

To prevent insects entering the tank, mesh should be no coarser than 12 × 12 meshes/25 mm^2 (WA Department of Health, 2003) or 7 x 7 meshes / 10 mm^2 (Northern Territory (2007)) for mosquito control in the tropical areas of Australia. Leaf diverters are also an important feature in roof water systems. Inline filters, UV disinfection, chlorination or boiling may be used depending on the use of the water (Department of Water, Western Australia, 2007; enHealth, 2010).

The location of the storage infrastructure will be dependent on aesthetic and space requirements for the chosen device. 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).

Appropriate flow rates need to be maintained for the user and therefore the majority of rainwater 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 rainwater from the tank. Generally a flow of less than 30L/min are suitable for most residential applications (NSW Department of Planning & Environment, 2015). Local government or State Government policy requirements may exist in regards to pump noise.

Runoff that is not collected in the tank and/or overflows should be diverted away from tank foundations, buildings or other structures (enHealth 2010). This water should be directed into gardens, infiltration systems or the drainage system. The overflow water should not be allowed to pool or to cause nuisance to neighbouring properties or to areas of public access.

The rainwater tank or stormwater harvesting system for reuse should be designed using continuous simulation (as identified in Book 7) and should consider the following:

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

5.4.5 Infiltration Systems

Infiltration systems should not be placed near building footings, as continually wet subsurface conditions or greatly varying soil moisture contents can impact on the structural integrity of these structures. The recommended minimum distances from structures (and property boundaries to protect possible future buildings in neighbouring properties) for various soil types is given in **Table 5.4** (Engineers Australia, 2006).

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.

Table 5.4 Recommended Minimum Set-Back Distances (after Engineers Australia, 2006)

Soil Type	**Minimum Distance from Building Footings for Infiltration System**
Sand	1.0 m
Sandy Clay	2.0 m
Weathered or Fractured Rock e.g. sandstone	2.0 m
Medium Clay	4.0 m
Heavy Clay	5.0 m

Soakwells

One method for infiltration of urban runoff into suitable soils is using soakwells (for soils with hydraulic conductivity values > 1 x 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. Soakwells 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).

Soakwells consist of a vertical perforated liner with stormwater entering the system via an inlet pipe at the top of the device (refer **Figure 5.2**). 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 soakwells are susceptible to clogging.

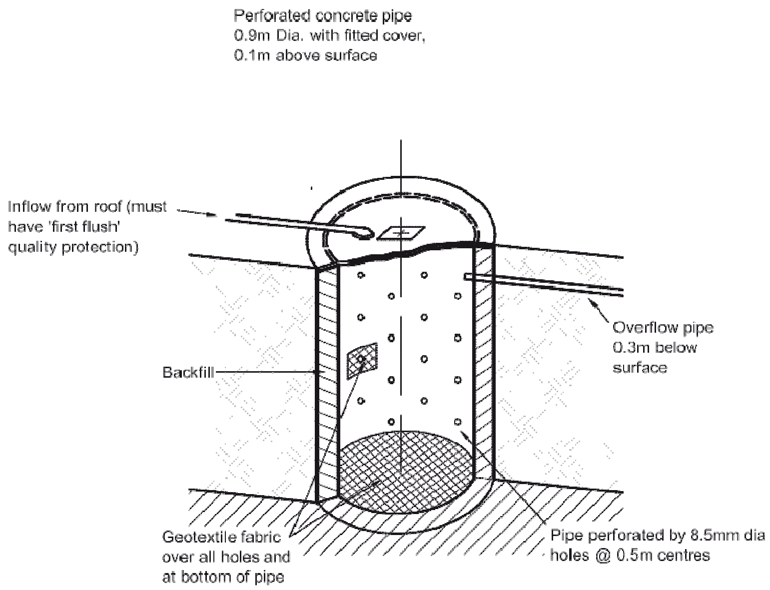


Figure 5.4 Leaky Well Infiltration System (after Department of Water, Western Australia, 2007)

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 (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.

Swales

Swales are shallow grassed channels – typically 0.30 to 0.50 m (maximum) deep, 5 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:

(i) they can be instrumental in retaining runoff through bed infiltration;
(ii) they can be effective in retaining pollutants conveyed in stormwater; (Breen et al, 1997; Lee et al, 2008) and,
(iii) they can fulfil a role in stormwater harvesting through soil moisture enhancement and, possibly, aquifer recharge and recovery (ASR).

The full potential of swales therefore includes each of the primary goals of stormwater retention as defined in Guidelines such as those listed in Sections 5.4.1 and 5.4.2. Clearly, this scope is very great: the following outline is therefore confined to "filter strip" swales which extend stormwater retention practice into an area of great challenge – that of surface runoff and associated pollution control in residential streets. The configuration of a "filter strip" swale in relation to a residential street carriageway is shown in Figure XX

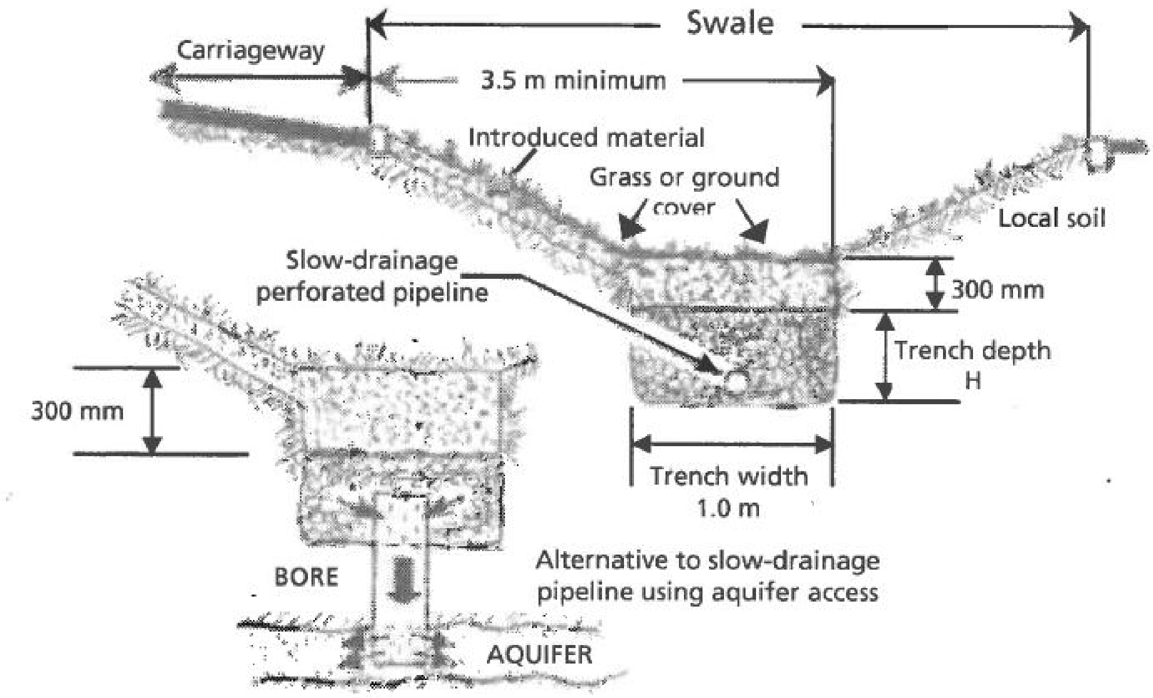


Figure 5.5 Main Components of a "Filter Strip" Swale (with Sub-structure)
(after Argue 2004/2013)

Swale systems of the type examined in this section abstract all 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 5.5**) with hydraulic disposal to aquifers (if available) or local waterways (slow-drainage) if necessary.

There are three broad types of "filter strip" swales:

(i) Swales whose cross-section includes a surface layer of sand and gravel mixture (100 mm thick) laid from the contributing carriageway edge to and across the bed of the excavated channel;
(ii) Swales whose cross-section includes the layer of introduced material described in (i), above, plus a gravel-filled trench located beneath the swale invert; and,
(iii) Swales whose cross-section includes the layer of introduced material and gravel-filled sub-structure, described in (i) and (ii), above, plus "hydraulic" means of removing/conveying accumulated, cleansed stormwater either by:
   a) direct disposal/recharge to a conveniently located aquifer; or
   b) access to a "slow drainage" perforated pipeline located in the base of the gravel-filled trench.

5.4.6 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, 2004/2013) 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 OSR measures given in **Table 5.5** are recommended in the interim.

Table 5.5 Interim Relationship between AEP and 'Emptying Time (after Argue (2004/2013))

	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

5.4.7 Operations and Maintenance

The following discussion of operation and maintenance of OSR measures is based on the guidance provided in published state government guidelines (Department of Water, Western Australia, 2007; Department of Planning & Environment, 2015, enHealth, 2010).

Rainwater Storage Systems

Rainwater storage systems require very little maintenance provided they are correctly installed. Manufacturers' maintenance guidelines should be adhered to, but critically assessed once the system is installed, to ensure that site specific conditions are taken into consideration in the maintenance regime. Typical maintenance requirements include:

- Every 3 months to 6 months:
  - cleaning of the first flush device;
  - removing leaf debris from gutters and roofs and prune of overhanging branches);
  - checking insect screens and other potential mosquito entry points at the onset of warm weather each year and whenever routine tank inspection and maintenance is undertaken;
  - checking the structural integrity of the tank, inlet and outlets. Repair any holes and gaps.
  - check for evidence of access by animals, birds or insects including the presence of mosquito larvae. If present, identify and close access points. If there is any evidence of algal growth (green growth or scum on or in the water), find and close points of light entry;
  - check for the pipework for structural integrity. Sections of pipework that are not self-draining should be drained. Buried pipework, such as with 'wet systems', can be difficult to drain or flush. Where possible drainage points should be fitted.

  - ensuring that water is not pooling beneath overflow outlets or taps whenever routine tank inspection and maintenance is undertaken
  - checking the rainwater tank pump and mains control switch are working properly (pump is operating when rainwater in tank, mains water flowing when rainwater tank is empty);
  - repairing all leaks and dripping taps to prevent cyclic starting and stopping of a typical pump that may occur in cases where there is a leak or a dripping tap; and
- Every 2 years:
  - checking sediment levels. If the water used from the tank becomes turbid prior to the 2 year mark, the sediment levels should be checked and if the top of the sediment layer is within 5 mm of the outlet, the tank should be drained and sediment removed.
  - Checking and replacing inline filters of the pumps and tank. If the water pressure into the development suddenly drops or is too low, the filters may need to be replaced at an earlier time. This is highly dependent on the size of the filter and the sediment size collected in the tank.

Soakwells / Infiltration systems

Soakwells and infiltration systems require maintenance for efficient operation and to reduce the risk of mosquito breeding, including regular inspection and cleaning to prevent clogging by sediments and litter. Pre-treatment measures can significantly reduce the maintenance requirements by preventing sediments and litter from entering the system. To prevent road/carpark soakwells or infiltration systems from being clogged with sediment/litter during road and housing/building construction, temporary bunding or sediment controls need to be installed.

Maintenance plans should identify owners and parties responsible for maintenance, along with an inspection schedule. Depending on the specific system implemented, maintenance should include at least the following:

- once the system is operational, inspections should occur after every major storm for the initial few months to ensure proper stabilisation and function. Attention should be paid to how long water remains standing after a storm; standing water within the system for more than 72 hours after a storm is an indication that soil permeability has been over-estimated;
- soakwells or infiltration systems should be inspected at least biannually to check and clean or repair if required: accumulated sediment, leaves and debris in any pre-treatment device, signs of erosion, clogging of any inlet and outlet pipes and surface ponding;
- when ponding occurs on soakwells, corrective maintenance is required immediately.
- When ponding occurs for longer than designed in infiltration systems, corrective maintenance is required immediately.

For the maintenance of swales and permeable pavement, the practitioner is referred to Melbourne Water (2005) and ARQ (Engineers Australia, 2004).

5.5 Integrated On-Site Detention and On-Site Retention

The opportunities to combine OSR with OSD has been considered by a number of researchers and practitioners including Scott et al (1998), Pezzaniti et al (2002), Coombes, P.J. (2009), Tennakoon and Argue (2011), Kannangara, D., Botte, M. and Thennakoon, A. (2013), van der Sterren et al (2007, 2012) and van der Sterren, M. (2012, 2014).

As discussed by van der Sterren (2014) increasingly homes are being designed in NSW and elsewhere to use less potable water by setting water reduction targets for houses and units (NSW Government, 2004a, BASIX Sustainability Unit, 2009). Local council policies often require new developments to consider the protection of local waterways and the capacity of existing drainage systems. This is resulting in a change towards OSR in combination with OSD, which often leads to the implementation of rainwater tanks and an OSD system on lot scale developments.

Each system is currently designed and considered as a stand-alone and therefore the stormwater controlled and collected through a rainwater tanks system is commonly not considered to contribute to the quantity or quality control of discharges from a site (Bankstown City Council 2006, Blacktown City Council 2005, 2006, Blue Mountains City Council 2005, Hawkesbury City Council 2000, Hills Shire Council 2010, Holroyd City Council 2003, Kogarah City Council 2006, Ku-ring-gai Council 2005, Parramatta City Council 2005, Penrith City Council 2010). The argument often remains that a reduction to OSD systems as a result of a rainwater tank cannot be applied, because of the uncertainty associated with antecedent conditions of a storm event. This has been investigated in the past, but solutions have not been readily adopted due to perceived risks or practical adoption barriers. For example, the volumetric reduction to OSD approach proposed by Coombes et al. (2001, 2002) or a reduction (credit) for Site Storage Requirements for OSD when a rainwater tank without or with dynamic airspace is installed on a lot proposed by Phillips et al (2005 and UPRCT, 2005) for the former UPRCT are not readily adaptable or applicable by Councils without a volumetric design requirement for OSD systems or outside of the Upper Parramatta River Catchment.

Some councils have followed the lead of the UPRCT and conducted modelling to determine PSD and SSR requirements. Penrith City Council (2008, 2010), for example, has conducted a simulation, which resulted in different PSDs for different areas of the LGA. On the other hand, most councils have not modelled their catchments, and use the pre-development runoff as the benchmark for the sizing of OSD systems (Blue Mountains City Council 2005, Hawkesbury City Council 2000). Hawkesbury City Council (2000) uses OSD to reduce the runoff from new developments or redevelopments to ensure that the capacity of the stormwater drainage infrastructure is not exceeded (ie. an “Infrastructure Compliant Stormwater Management” approach), while other Councils use OSD basins to control erosion where waterways are steep and increased runoff can increase erosion.

Councils develop their own Development Control Plans (DCPs) and therefore each council has different requirements for stormwater management. As a result, designers face differing regulations for developments in different council areas. This inconsistency has been highlighted by O’Loughlin et al. (1995) and Finnemore and Lynard (1982). While the former UPRCT formulated uniform control requirements across the four LGA which drain to the Parramatta River, three of the four Councils within the UPRCT have included additional requirements to the UPRCT requirements (Baulkham Hills Shire Council 2004, Blacktown City Council 2005, 2006, Holroyd City Council 2003).

5.5.1 An Integrated Approach

The increased uptake of rainwater tanks creates an opportunity to adopt an integrated approach to lot scale stormwater management. Some councils, such as Blue Mountains City Council (2005) and Hawkesbury City Council (2000), agree that the OSD volume requirement can be added to a rainwater tank solution for the BASIX requirements. This method results in rainwater tanks with three outlets, one for use of rainwater (e.g. connected to 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 (see **Figure 5.6**).

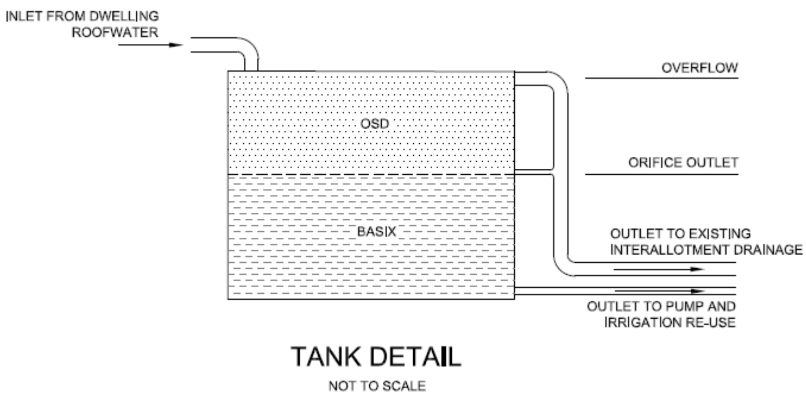


Figure 5.6 A Rainwater Tank with Three Outlets (after van der Sterren, 2014)

As outlined in **Section 5.9.1** the fourth edition of the UPRCT OSD Handbook accounts for dedicated and dynamic airspace in a rainwater tank (refer **Figure 5.7**). It was assumed that the rainwater tank was connected to the laundry, toilets and outdoor areas and the OSD system is designed according to the UPRCT handbook. The results showed that the airspace in a rainwater tank can be used as a partial credit towards the required SSR. This credit varies for each development type and ranges from 32% to 65% depending on the design of the system. Coombes, Frost and Kuczera (2001) also found that on the lot scale the OSD systems reduced the peak discharge as required, but the rainwater tanks only reduced the volume of discharge, the peak flows remained the same. Coombes, Frost and Kuczera (2001) argued that peak discharges at the lot scale had little or no bearing on the floods at a catchment scale, as flooding is a volume driven process.

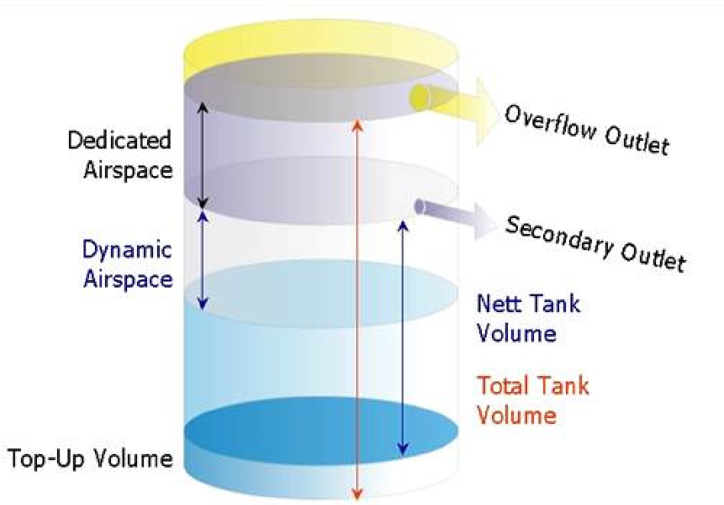


Figure 5.7 Key Parameters of a Rainwater Tank (after UPRCT, 2005)

The research undertaken by van der Sterren, 2012, 2014 was based in Hawkesbury LGA. This council requires that OSD systems limit post-development flows to pre-development levels to ensure that the post-development flows do not exceed the capacity of the aging stormwater infrastructure (Hawkesbury City Council, 2000).

Five rainwater tanks and an OSD system were investigated and a year-long data collection was conducted on the flow and quality of the discharges (van der Sterren et al. 2013). The data was used for detailed modelling on a lot scale (van der Sterren et al., 2014) and a catchment scale (van der Sterren, 2012). It was found that up to and including the 1 EY could be controlled using a rainwater tank on site connected to multiple end-uses (van der Sterren et al., 2012).

The outcome of the research (van der Sterren, 2012) highlighted the potential benefits of replacing the common OSD requirement in council guidelines with a recommendation to implement a lot scale treatment train system that:

- controls the post-development discharges to pre-development discharges for up to and including the 1% AEP event; and
- provides a non-potable water supply (including sites where mains water is available) for toilet-flushing, hot water, laundry and irrigation

These requirements can lead to a flexible approach by not mandating the type of system to be used and does not contradict BASIX or other state government policies, as the non-potable water supply is for stormwater management purposes, not water saving requirements. For example, the design with these development constraints will need to take into account the household water use and local rainfall characteristics. It should be further required that the overflow and first-flush from such a harvesting system are diverted to an additional control system, such as an OSD to control the 50% AEP runoff (with a possibility up to a 1% AEP, as per the commonly set OSD design guidelines). The OSD can be downsized, as the orifice would only need to control the 50% AEP runoff, instead of the 1 EY up to the 1% AEP event. The minimum discharge rate would be greater from the detention basin in comparison to the business as usual approach and can therefore reduce the total volume of OSD required. The discharge during a 1% AEP event will be governed by a 50% AEP orifice, rather than an orifice designed for a 1 EY event. This system will still maintain the post-development discharges to pre-development discharges for a 1 EY (rainwater tank) up to and including the 1% AEP (OSD). T This approach can also be applied to other types of retention devices, such as infiltration, soak wells, permeable pavement and other reuse and retention devices.

5.6 Community and Regional Detention

5.6.1 Introduction

The use of community and regional detention basins has grown since 1975 is widespread in NSW and elsewhere both in the outer suburbs and in country areas particularly in newly urbanising areas (IEAust., 1985).

Detention basins, also known as retarding basins in Australia, are measures which temporarily store stormwater to reduce downstream flowrates. Outflows are typically controlled by a low-level pipe or culvert and a high-level overflow spillway. An example basin inflow and outflow hydrograph are given in **Figure 5.8**.

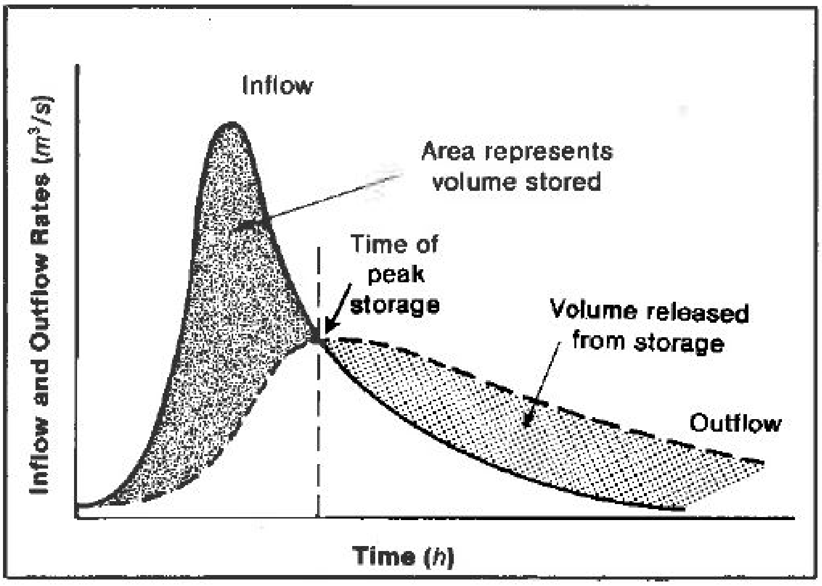


Figure 5.8 Basin Inflow and Outflow Hydrographs (after ARR, 1999)

Community and regional 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 spectator use. Basins used for passive recreation may include stands of trees and other vegetation.

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

5.6.2 Available Guidelines

There are many guidelines on community and regional detention include ACT Department of Urban Services (1998), Department of Irrigation and Drainage (2000), Department of Irrigation and Drainage (2012a), Department of Water, Western Australia (2007), Derwent Estuary Program (2012), Hobart City Council (2006), Melbourne Water (2010), Queensland Department of Energy and Water Supply (2013), Queensland Department of Natural Resources and Mines (2007) and Stormwater Committee Victoria (1999). These guidelines can be readily used for designing and modelling detention systems, using the modelling and storm patterns as described in Book 2 and Book 7. In the absence of local guidelines or when formulating guidelines or Council requirements the design procedures discussed in this chapter can be considered.

5.6.3 Design Procedures

In the absence of or when formulating local guidelines on community and regional detention or Council requirements the following guidelines should be considered. The primary objective of detention is to reduce the peak discharge from a catchment. It is in essence a Category 1 system and its discharge control requirements should be based on a catchment-wide assessment, as discussed in section 5.2.

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 partial 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.

Flood Capacity

Community and regional basins are considered dams, as they store significant volumes of stormwater, and therefore they 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, date) guidelines. A detailed risk assessment of a storm exceeding the Dam Crest Flood should be considered in the design of a retarding 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 of a basin.

Basins should be designed 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 ‘Assessment of the Consequence of Dam Failure’ (2000) should be undertaken for any proposed retarding basin to determine the hazard category of the structure. The “Initial Assessment” should include but is not limited to (Melbourne Water, 2010):

- Determining the potential downstream inundation extent, using Method 1 - Approximate determination (definition provided in Appendix C of the “Guidelines on Assessment of the Consequences of Dam Failure” (ANCOLD, 2000b), the associated Population at Risk (PAR) and the Severity of Damage and Loss;
- Determining the Hazard Category for the basin based on existing conditions, and for the proposed future 'ultimate development' scenario;
- Determining the required flood capacity (Spillway Design Flood) to satisfy the “Fallback” design flood criteria defined in Guidelines on Selection of Acceptable Flood Capacity of Dams” (ANCOLD, 2000a)
- Preparation of a report detailing the study and the findings, and summarising the ANCOLD requirements for the basin.

Depending on the findings of the “Initial assessment” a more detailed assessment (Intermediate or Comprehensive Assessment as defined under Consequence Assessment within 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 retarding 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:

1. The consequences of one basin failure cascading downstream into lower basins should be evaluated; and
2. 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.

Site Investigation

Site investigations should be undertaken as part of the overall design process. These investigations include:

- Geological assessment of the site;
- A program of bore holes to assess the retarding basin and spillway, foundations and any preferred borrow pits;
- Laboratory testing of soils to assess their suitability for placement in any earthen embankment and for assessments of the embankment foundation,
- The presence of desiccation fissuring; and
- Shear strength testing of the overall design.

Embankment Design

The embankments of the retarding basins should be designed using appropriate stability analysis and practices. In particular appropriate foundation treatment should be specified. For earthen embankments suitable compaction levels, vegetation 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. Chimney intercept filters and filter/drainage blankets should be used for all high and extreme hazard category retarding basins. Such filters may also be required for lower hazard category retarding basins. All earthen embankments constructed from dispersion soils must have a chimney filter and downstream filter/drain (Melbourne Water, 2010).

Minimum recommended top widths for earthen embankments are provided in **Table 5.6**.

Table 5.6 Minimum Recommended Top Width for Earthen Embankments (after Department of Irrigation and Drainage (2000)

Height of Embankment	Top Width
Under 3 m	2.4 m
3 m to 4.5 m	3.0 m
4.5 m to 6 m	3.6 m
6 m to 7.5 m	4.2 m

Internal batter gradients should consistent with personal safety and generally within the following upper limits (NSW Government, 2004b):

- where water depth is less than 150 mm when surcharging, 1(V):2.5(H) to 1(V):4(H) on earth structures and vertical on rock or gabion structures;
- where water depth is between 150 and 1,500 mm when unprotected and surcharging, a maximum slope of 5(H):1(V);
- where water depth is between 150 mm and 1.5 m when protected (e.g. fenced) and surcharging or greater than 1.5 m:
    - 2.5(H):1(V) to 4(H):1(V) on earth structures (with the actual gradient adopted depending on various soil characteristics);
    - 0.5(H):l(V) on rock gibber structures;
    - 1(H):4(V) on gabion basket structures; and
    - 1(H): 4(V) on stacked (rough squared) rock structures;

If the earthfill for any embankment is to be 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 1 m below the estimated depth of such sand layers/lenses at the retarding basin floor.

Basin freeboard requirements for a variety of basins are provided in **Table 5.7**.

Table 5.7 Basin Freeboard Requirements

(after 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]

Note: [1] Freeboard must fully contain the potential wave height if the resulting overtopping is likely to represent a safety risk to the embankment or undesirable erosion. US Army Corps of Engineers (1984) provides guidelines on the estimation of wave height.

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, where the degree of protection required is subject to the flow velocity.

The following treatments are recommended as a guide:

- $V \leq 2$ 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
- $V \geq 7$ m/s hard surfacing with concrete, riprap or similar

An example where a basin embankment has been protected by a layer of roller compacted concrete on the crest and downstream face of an earthen embankment is the Sierra Place Basin located on Toongabbie Creek in the suburb of Baulkham Hills in Sydney. The investigations which led to the adoption of this approach to scour protection are discussed in Phillips (1987a); Phillips et al (1990, 2000) and UNSW WRL (1988).

On occasions the embankment and spillway are combined into a single structure. Examples include the Loyalty Road Basin located on Darling Mills Creek in the suburb of North Rocks in Sydney and Wrights Retardation Basin located in the suburb of Banks in the ACT (Phillips, 1987b). Both basin walls, which are also spillways, were constructed in roller compacted concrete.

Basin Floor

The floor of basin shall be designed with a minimum grade of 1% to provide positive drainage to the basin outlet. Detention basins should be designed to include underdrains to empty the bottom of the detention facility for ease of maintenance.

Primary Outlets

The key function of primary outlets is to release flows from a detention basin as the designed discharge rate. Some typical primary outlets are shown in **Figure 5.9**. Book 6 details how these outlets are to be designed.

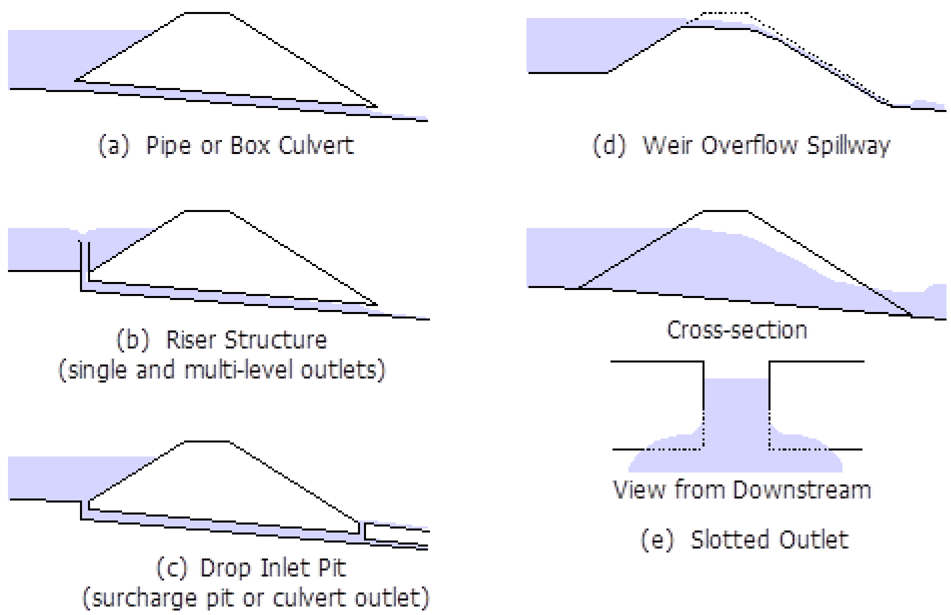


Figure 5.9 Typical Detention Basin Primary Outlets
(after Department of Irrigation and Drainage (2000))

Pipe or box culverts are often used as outlet structures for detention 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 flows. 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.

Rubber ring jointed pipes without lifting holes are recommended for pipe culverts. All culverts should be provided with suitable bedding and cut-off walls or seepage collars to prevent possible failure of earthen embankments due to piping (Department of Irrigation and Drainage (2000)).

Recommendations for the design of outlet structures are given in Book 6 as well as by ASCE (1985), US Bureau of Reclamation (1987) and FHWA (2001). Hydraulic relationships for various outlet structures are also provided in the User Manuals for industry software packages used widely in Australia.

Energy dissipaters can be used to mitigate erosion of outlets by dissipating energy and reducing flow velocity. They are usually permanent structures and can be constructed from rip rap, grouted rip rap, gabions, recycled concrete or concrete. Guidance on the design of energy dissipation structures on outlets is given in Book 6 as well as USBR (1984).

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 “Fallback” Design Flood. In some cases where the required “Fallback” 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 given 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 as well as by Govinda Rao and Muralidhar (1963); Bos (1978); Van Haveren and Bruce (1986); Ramamurthy et al. (1987); Swamee (1988); Swamee et al. (1994); Brater et al. (1996); Chanson and Montes (1998); French (1986); Hager (1987); Borghei et al. (1999); Johnson (2000); USBR (2001); Shesha Prakash and Shivapur (2004); Martínez et al. (2005); Göğüş et al. (2006); Jan et al (2006; 2009) and Sargison and Percy (2009).

Role of Physical Modelling

During the design of community and regional detention basins it can be found that there is a degree of uncertainty regarding the hydraulic performance of the basin geometry, embankment protection measures, and the inlet and/or outlet structures. One way to reduce design uncertainties can be to undertake physical modelling of a proposed basin and/or its inlet and outlet(s).

When designing the Sierra Place Basin located on Toongabbie Creek in the suburb of Baulkham Hills in Sydney uncertainties which arose included the:

- most effective method to protect the earthen basin embankment against overtopping flow up to the 0.01% AEP event,
- likelihood of air entraining vortices forming at the inlet and the most effective method to overcome the formation of vortices,
- most effective method to dissipate energy at the outlet to reduce 1% AEP flow velocities downstream of the outlet to around 2 m/s.

Physical modelling was undertake to inform design decisions regarding all three issues as discussed by Phillips et al (1990) and UNSW WRL (1988).

Similarly during the design of the William Slim Drive PMF Basin in the suburb of Baulkham Hills in Sydney the was found during the hydrological and hydraulic assessments of alternative basin configurations that the estimated peak PMF basin water level was sensitive to adopted inlet and outlet losses for the basin outlet comprising twin Bebo arches. This sensitivity supported the need to undertake physical model testing to allow the outlet configuration to be refined and to increase the confidence in the estimated peak PMF basin levels as discussed by Phillips et al (2007, 2008) and UNSW WRL (2006). These investigations are outlined in **Section 5.9.3.**

It is concluded that physical models continue to have a valuable role in the design of major hydraulic structures in urban areas particularly when predicted flood levels and flood storage are sensitive to assumed values of model parameters.

5.6.4 Routine Maintenance

The following discussion on operation and maintenance requirements for community and regional detention measures is based on the guidance provided in Chapter 9 Structural Controls in the Stormwater Management Manual for Western Australia (Department of Water, Western Australia, 2007).

A maintenance plan should be prepared and implemented for each community or regional detention measure. It should include removal of accumulated litter and debris in the detention area. The frequency of this activity may be altered to meet specific site conditions and aesthetic considerations.

Biannual inspections for sediment accumulation, pest burrows, structural integrity of the outlet, and litter accumulation are typical. In parkland settings, maintenance plans should also address irrigation, nutrient and pest management issues. Accumulated sediment in any forebay should be removed about every 5-7 years or when the accumulated sediment volume exceeds 10% of the basin volume. Sediment removal may not be required in the main detention area for as long as 20 years.

Vegetation harvesting should be timed so that it has minimal impact on factors such as bird breeding and there is time for re-growth for runoff treatment purposes.

5.6.5 Basin Safety Assessment and Emergency Planning

In catchments where one or more detention basins are located basin safety assessments should be undertaken periodically to monitor the performance and condition of existing basin structures and to determine if any remedial actions need to be implemented to maintain the safe functioning of each detention basin. A Dam Safety Emergency Plan (DSEP) should be also implemented to determine those conditions that could forewarn of an emergency and specify the actions to be taken and by whom and to identify all resources, special tools, equipment, keys and where they can be located if required in an emergency.

Basin Safety Assessment

Basin safety reviews need to consider a range of issue including:

- Basin design and design data, including structural, hydraulic, hydrologic and geotechnical aspects;
- Construction methods;
- Operational and maintenance history, photographs and reports;
- The performance and condition of existing basin structures;
- Conducting a failure modes analysis if not previously undertaken;
- Conducting a hazard analysis if not previously undertaken;
- Conduct other specific investigations and analysis as necessary;
- Compare the standards used for building and upgrading the basin against current design standards;
- Reach final conclusions and make recommendations; and
- Provide an independent peer review.
- Based on the findings of the basin safety review, a remedial action study may need to be undertaken and a remedial action report prepared for a detention basin and if needed recommend possible basin upgrade works.

A dam break analysis should be undertaken for a detention basin and would typically include investigation of the following dam break cases:

- Dam crest flood with and without dam failure
- Probable Maximum Flood (PMF) with and without dam failure
- Imminent failure flood (IFF) with and without dam failure

Impacts should be assessed to a point downstream where there is no longer a threat to the safety of non-itinerant persons and/or the catchment outlet whichever is closer to the basin.

The consequences of the dam break should be described in a report which should include mapping to show:

- Extent, depth, velocity and travel time of floodwater in relation to dwellings.
- Location of dwellings affected by above-floor flooding.
- Areas of high, medium and low flood risk.

Dam Safety Emergency Planning

In catchments where one or more detention basins are located a Dam Safety Emergency Plan (DSEP) should be prepared which:

- Lists and prioritises and provides contact details of all persons and organisations involved in the notification process, and preparation of a draft notification flow chart;
- Identify all jurisdictions, agencies, and individuals that could be involved in the preparation, adoption and implementation of the DSEP. Liaise with these stakeholders during the development of the DSEP;
- Determine those conditions that could forewarn of an emergency and specify the actions to be taken and by whom;
- Identify all resources, special tools, equipment, keys and where they can be located if required in an emergency;
- Identify primary and secondary communication systems, both internal (between persons at a Council) and external (between Council and outside entities);
- Provide draft DSEP to all relevant parties for review and comment;
- Hold meetings with all parties (including emergency management agencies) included in the notification list for review and comment on the draft DSEP;
- Make any revisions, obtain the necessary plan approval, and disseminate the final DSEP to those who have responsibilities under the plan.
- Create a “Summary Information Sheet for Emergency Agencies” to be included beside the “Notification Flowchart”. The sheet is to include background information (basin owner, basin location, basin safety and safety status) and notification protocols (owner’s actions, notifier’s actions and emergency response requirements).

5.7 Regional Retention

5.7.1 Introduction

Subject to the availability of suitable sites and soils the use of community and regional retention measures can be more cost-effective than distributed OSR measures. Typical community retention facilities include:

- Managed Aquifer Recharge (MAR) systems;
- Wetlands and Storage Ponds; and
- Infiltration trenches and Basins.

5.7.2 Available Guidelines

There are many guidelines on community and regional retention include Argue, J R (Ed, 2004/2013), Department of Energy and Water Supply (2013), Department of Irrigation and Drainage (2000), Department of Irrigation and Drainage (2012b), Department of Water, Western Australia (2007), Derwent Estuary Program (2012), Government of South Australia (2010) and Melbourne Water (2005).). These guidelines can be readily used for designing and modelling retention systems, using the modelling and storm patterns as described in Book 2 and Book 7.

5.7.3 Design Procedures

As outlined in Section 5.4.3, ten basic design procedures are given in *WSUD: Basic Procedures for 'Source Control' of Stormwater* (Argue, 2004/2013). Procedures 1-4 inclusive are applicable, primarily, to Category 1 installations; Procedures 5 - 8, to pollution control systems; and Procedures 9 and 10 relate to rainwater (domestic or industrial) harvesting systems. These procedures can be scaled up to larger retention devices.

In the absence of guidelines for community or regional retention measures or Council requirements the following guidelines, which are based on the guidance provided in Chapter 9 Structural Controls in the Stormwater Management Manual for Western Australia (Department of Water, Western Australia, 2007) should be considered. Further additional guidance can be found in the above mentioned WSUD guideline (Argue, 2004/2103) but also in ARQ (2006) and Melbourne Water (2005).

5.7.4 Managed Aquifer Recharge

Managed aquifer recharge (MAR), also known as artificial recharge, is the infiltration or injection of water into an aquifer (EPA, 2005). 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.

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

Aquifer Characterisation

Factors to consider in evaluating the suitability of an aquifer 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.

System Controls and Monitoring

Controls should be incorporated to shut down an injection pump or valve if any of the parameters determined for the project exceed the criteria for the environmental values of the aquifer. Examples of parameters to be measured include (Melbourne Water, 2005; Department of Water, Western Australia, 2007):

- standing water level in the well
- injection pressure
- electrical conductivity (salinity)
- turbidity
- temperature
- pH
- dissolved oxygen concentrations
- volatile organics
- other pollutants likely to be present in injected water that can be monitored in real time

Other ongoing monitoring should include monitoring water levels in valuable groundwater dependent ecosystems.

5.7.5 Infiltration Basins and Trenches

As discussed by Department of Water, Western Australia, 2007, two primary infiltration systems used at larger scales are infiltration trenches and infiltration basins.

Infiltration Trench

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 around 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 Basin

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).

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).

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).

Due to their flexibility in shape, trenches 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 (refer Table X.4).

In general, stormwater runoff should not be conveyed directly into an infiltration system, but the requirement for pre-treatment will depend on the catchment eg. residential or industrial, etc. 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 basins/trenches 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).

Root barriers may need to be installed around sections of infiltration systems that incorporate perforated/ slotted pipes or crate units where trees will be planted, to prevent roots growing into the system and causing blockages.

An example of a major infiltration basin is the stormwater (wet) infiltration basin constructed on a site at Cronulla, NSW (refer Phillips et al, 1999, Cardno Willing, 2005).

5.7.6 Maintenance

The following discussion on operation and maintenance requirements for community and regional retention measures is based on the guidance provided in Chapter 9 Structural Controls in the Stormwater Management Manual for Western Australia (Department of Water, Western Australia, 2007).

Managed Aquifer Recharge Systems

Pumps and pre-treatment equipment need to be maintained (e.g. by replacing filter media at manufacturer specified intervals or volumes). Keeping maintenance records is a component of good management practice.

Infiltration Systems

Regular maintenance is required for proper operation of infiltration systems.

Maintenance plans should identify owners and parties responsible for maintenance, along with an inspection schedule. The use and regular maintenance of pre-treatment measures will significantly reduce maintenance requirements for infiltration systems.

Depending on the specific system implemented, maintenance should include at least the following:

- once an infiltration system is operational, inspections should occur after every major storm for the initial few months to ensure proper stabilisation and function. Attention should be paid to how long water remains standing after a storm; standing water within the system for more than 72 hours after a storm is an indication that soil permeability has been over-estimated;
- inspect and clean pre-treatment devices biannually and ideally after major storm events. Important items to check and clean or repair if required include: accumulated sediment, leaves and debris in the pre-treatment device, signs of erosion, clogging of inlet and outlet pipes and surface ponding;
- when ponding occurs, corrective maintenance is required immediately.

In the case of infiltration trenches, clogging occurs most frequently on the surface. Grass clippings, leaves and accumulated sediment should be removed routinely from the surface. If clogging appears to be only at the surface, it may be necessary to remove and replace the first layer of filter media and the geotextile filter.

The presence of ponded water inside the trench after an extended period indicates clogging at the base of the trench. Remediation includes removing all of the filter media and geotextile envelope, stripping accumulated sediment from the trench base, scarifying to promote infiltration and replacing new filter media and geotexile. Vegetation can assist in prevention of clogging as the root network breaks up the soil and thereby promotes infiltration.

In the case of infiltration basins, sediment should be removed when it is sufficiently dry so that the sedimentation layer can be readily separated from the basin floor.

5.8 Integrated Community or Regional Detention and Retention

Regionalised detention and retention are not mutually exclusive and can be combined to ensure that both the volume and rate of urban runoff is reduced, mitigated or controlled. Regional combined facilities have been constructed in recent years throughout Australia and can take the form of wetlands with extended detention and harvesting, detention basin with subsurface infiltration, and many others. The design of these integrated regionalised facilities should take into account the catchment wide strategies and the mitigation strategies and can be considered a category 4 system.

The combined systems are recommended to be designed to:

- Control the discharges to the acceptable levels for downstream systems through extended detention and or detention;
- Control the volumes to the acceptable levels for downstream systems through infiltration, retention and reuse; and

- Convey larger than design flows safely through the system.

Additional benefits can be obtained with integrated regionalised facilities through the integration of water sensitive urban design techniques and pollution control. For more information regarding the design for pollution control of these regionalised facilities, please refer to ARQ (2004), Melbourne Water (2005) and relevant local and state government guidelines.

For the quantity design of these regionalised integrated facilities, the appropriate modelling techniques (see Book x) should be utilised. The modelling should also consider:

- the reuse potential, rate and occurrence;
- the infiltration capacity of the subsoils; and
- the maximum discharge rates for various storm events and those critical for the design.

Where possible existing guidelines should be used for the design of these systems, such as a combination of the appropriate methodologies in the Department of Land and Water Conservation (1999), Melbourne Water (2005), ARQ (2006), Urban Services, ACT (undated) and local state and federal guidelines in regards to recycling and reuse. Where these guidelines do not exist or are not applicable, the following guidance can be used.

5.8.1 Combined Design Procedure

Existing practice has shown that the following steps are generally undertaken to design an integrated regional detention and retention system:

1. Identify the discharge requirements for the site;
2. Identify the potential reuse demand, infiltration and/or recharge capacity to the aquifer;
3. Identify the volume reduction required;
4. Determine the design the storage for the appropriate storm event to cater for the discharge constraints of the site (eg. adopt the detention methodology);
5. Refine the detention storage requirement through continuous simulation or complete storm events methodology and emptying time to ensure that the volume reduction is obtained. This step should include the modelling of the reuse, infiltration and / or recharge to the aquifer;
6. If continuous simulation is conducted do a partial series analysis on the outflows to show that the discharge regime is as required;
7. Ensure that the components of the system (e.g. pre-treatment of the wetland / gross pollutant trap etc) are designed to their required removal rates as per Water Sensitive Urban Design guidelines and that the design flows for the retention and detention basin can be transferred effectively through these systems;
8. Finalise the detailed design of the combined system as is applicable to its type with appropriate side batters, vegetation and riprap (to prevent erosion) – see various sections throughout this Book for guidance;
9. Ensure that the requirements of detention and retention facilities as discussed in this book are met (including but not limited to Dam Safety, People Safety, Depth of Ponding);
10. Ensure that the overflow of the system, the individual components and treatment devices can cater and transfer flows through the system greater than the design events to minimise nuisance flooding and safely convey the discharges to the legal point of discharge (e.g. spillway, overflow system, bypass system).

Key to a good integrated system is to ensure that the flows from a very high recurrence to a very low recurrence can be safely conveyed and controlled through the system and to ensure that maintenance is easy (including proper access and draining capacities for cleaning) to prevent failure or blockages of the system.

5.8.2. Combined System Example

The combined systems can take a number of different forms and serve a number of different purposes. One such example is the irrigation of the Blacktown Sportsgrounds. The Angus Creek Stormwater Harvesting Scheme was developed to reduce the urban flows from Angus Creek. The harvested water is being used to irrigate the Blacktown International Sportspark fields, Anne Aquilina Reserve, Kareela Reserve, Charlie Bali Reserve and top up the Nurringingy Ornamental Lakes. In addition, this treatment system also provides water for other users and water for flushing toilets within the reserves and sportspark.

Angus Creek is a highly urbanised catchment that encompasses the suburbs of Rooty Hill and Minchinbury and is 655ha in size. Angus Creek catchment generates about 2 billion litres of stormwater each year of which 200 million litres will be harvested. Water will only be harvested in storm events and will ensure that an environmental flow is not taken. Once the stormwater is collected from the creek it will be pumped through a gross pollution trap to collect litter, branches, leaves, dirt and sand.

A basin collects and stores the stormwater from Angus Creek until it is pumped through a litter trap and into the wetlands in the Sportspark. In storm events, harvesting this stormwater reduces the amount of fast, damaging flows which can erode the creek channels and flush away native fish and insects. The lower sections of Angus Creek are important habitat for fish such as the Australian Bass (Blacktown Council (undated)).

5.9 CASE STUDIES

A number of case studies are presented to demonstrate the application of a number of the approaches and techniques described in this chapter. The case studies are outlined as follows.

Case Study No. 1 2005 UPRCT OSD Guidelines, NSW

> This case study demonstrates the catchment based assessment approach recommended in Chapter 5.2 leading to the adoption of control requirements for a frequency staged OSD systems installed in the upper Parramatta River catchment.
>
> Historically, the primary objective of OSD controls was to manage flooding in a 1% AEP event only. In contrast, the fourth edition of the OSD Handbook for the upper Parramatta River catchment details the sizing and design of OSD systems for lots located in a 110 km^2 urban catchment which limit peak flows in the 50% AEP and 1% AEP events. These OSD requirements are outlined in Section 5.9.1.

Case Study No. 2 Heritage Mews, NSW

This case study demonstrates the application of the **regime-in-balance** stormwater management strategy outlined in Section 5.2 which requires runoff volume from a developed site to be equal to its 'greenfields' discharge in the adopted critical design storm.

The strategy objectives of 'before-and-after' runoff volume equality and peak flow less than the permissible site discharge were achieved using rainwater tanks, in-ground trenches and 'slow-drainage' as outlined in Section 5.9.2.

Case Study No. 3 William Slim Drive PMF Detention Basin, ACT

This case study demonstrates the assessment of a PMF detention basin sited on the Ginninderra Creek floodplain adjacent to William Slim Drive upstream of the suburb of McKellar in Canberra, ACT. During the design of community and regional detention basins it can be found that there is a degree of uncertainty regarding the hydraulic performance of the basin geometry, embankment protection measures, and the inlet and/or outlet structures. One way to reduce design uncertainties can be to undertake physical modelling of a proposed basin and/or its inlet and outlet(s).

The case study demonstrates that physical models continue to have a valuable role in the design of major hydraulic structures in urban areas particularly when predicted flood levels and flood storage are sensitive to assumed values of model parameters. These investigations are outlined in **Section 5.9.3.**

Case Study No. 4 Gosnells, WA

This case study demonstrates the application of the **Infrastructure Compliant Stormwater Management** (ICSM) approach outlined in Section 5.2. It outlines the assessments undertaken to formulate a strategy to maintain the performance of existing conveyance infrastructure by controlling runoff from each development site with the LGA. These investigations are outlined in **Section 5.9.4.**

5.9.1 2005 UPRCT OSD Guidelines, NSW

The upper catchment of the Parramatta River is one of the most urbanised catchments in Sydney and Australia. In the wake of repeated flooding in the catchment the Upper Parramatta River Catchment Trust (UPRCT) was established in 1989 to address the impacts of urbanisation on flooding and water quality in the 110 km^2 catchment (see **Figure 5.10**).

There are two key components to the former UPRCT's flood mitigation effort. The first aimed to reduce and eventually eliminate the present flood threat. This involved public expenditure of more than $50 million on projects such as retarding basins, channel improvements and levees to protect some 2,200 properties threatened by mainstream and trunk drainage flooding. The second component aimed to prevent the growth of the already substantial flooding problem caused by increasing development of the catchment. This is achieved through planning and development controls of which On-site Detention (OSD) is an important element. This is in essence a yield minimum strategy using a Category 1 control.

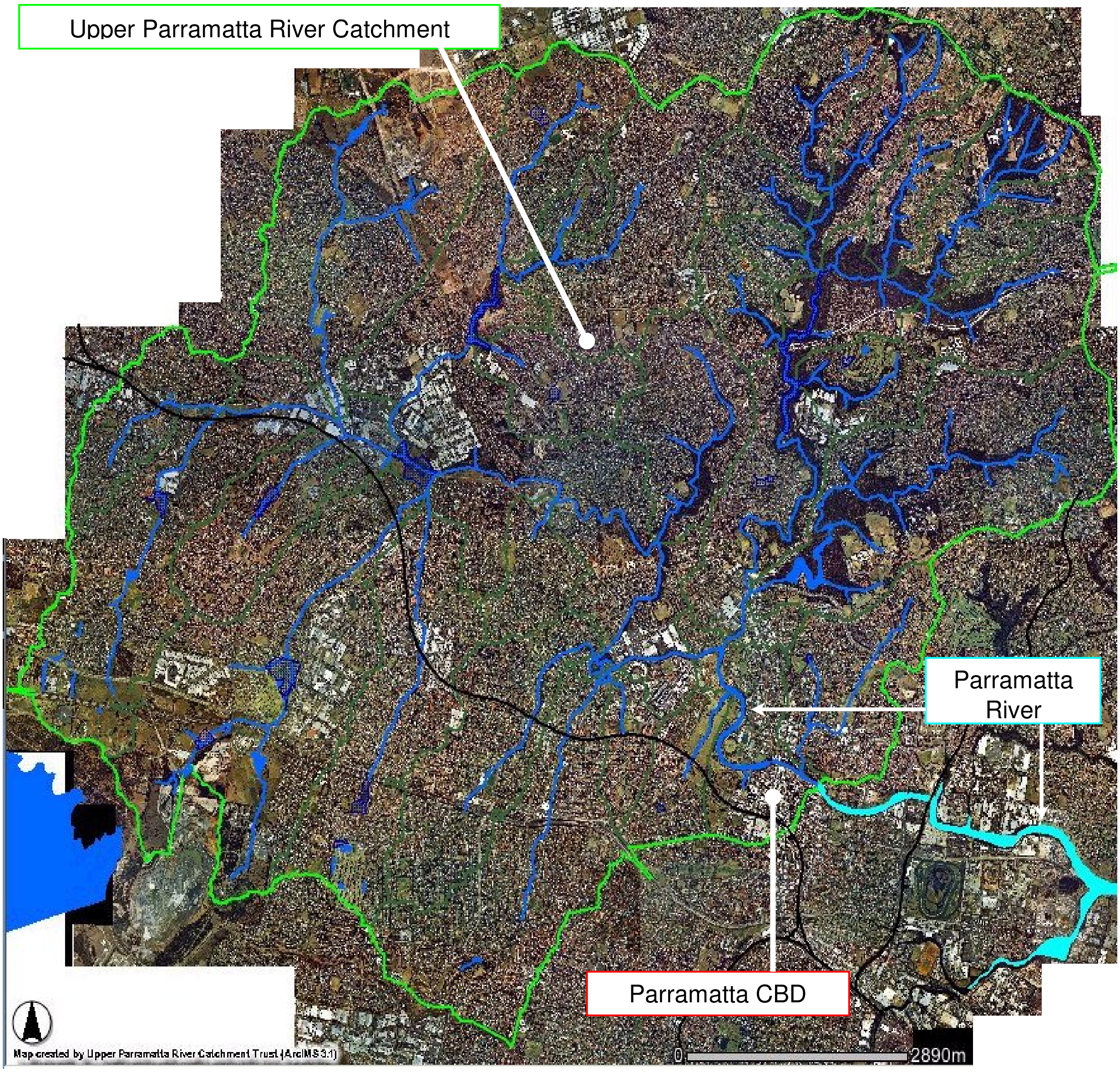


Figure 5.10 The Upper Parramatta River Catchment

The former UPRCT supported the implementation of OSD by publishing an On-site Stormwater Detention Handbook for experienced OSD designers. Since the publication of the *first edition* in September 1991 (UPRCT, 1991), the Handbook was purchased by over 500 OSD practitioners. The second edition in November 1994 was released to take advantage of the considerable body of practical experience that had been developed, as well as including the results of several Trust-sponsored research projects (UPRCT, 1994). The *third edition* reflected the experience gained by Council staff and consultants in the practical application of OSD (UPRCT, 1999).

The *fourth edition* released in December 2005 reflected the outcomes of detailed investigations undertaken in 2002 to 2004. In 2002 the Trust commissioned a review the Trust's OSD parameters. This review used the latest version of the XP-RAFTS rainfall/runoff package which explicitly models the rainfall runoff process on an individual lot and the adjoining strip of roadway, then combines countless individual single lot models to simulate flood behaviour at the neighbourhood, sub-catchment and catchment scales, based on the Trust's very detailed hydrologic XP-RAFTS model.

This modelling approach was described by Goyen et al (2002). It was used to determine the OSD parameters required to ensure no increase in flood peak flows under a plausible ultimate development scenario. The review recommended that there be no change to the PSD, but that it may be possible to reduce the SSR by up to 20%.

The OSD policy on which preceding editions of the handbook has been based will prevent increased flooding during very large (1% AEP) storms, but will have no impact on smaller, more frequent storms (50% AEP). In environmental terms, these smaller storms can cause more erosion damage to watercourses. From a sustainability viewpoint it would be desirable to have the stormwater runoff from developed sites more closely mimic pre-development conditions. Consequently, the possibility of using a two-stage outlet to control site runoff in both the 50% AEP and 1% AEP storms was investigated.

Catchment-based Studies

In further studies undertaken after the initial 2002 review, several significant changes to the OSD policy were assessed including:

- An on-line OSD storage;
- Dual outlets ie. primary and secondary outlets;
- An uncontrolled primary outlet ie. outlet without HED; and
- A discharge control pit for the secondary outlet only ie. outlet with HED

Under the alternative OSD arrangement that was investigated all site runoff is directed to the OSD storage. The water level in the OSD storage rises gradually. As it does the discharge through the orifice also increases gradually as the depth of water (the 'head') above the orifice increases. In small storms the discharge leaving the site through the primary outlet (orifice) would be much less than occurred previously due to the adoption of a reduced PSD for the primary outlet. In major storms a secondary outlet with a higher PSD would control outflows from the OSD tank. In combination these two outlets achieve the aims of reducing peak flows in frequent storms as well as in major storms.

Over a period of two years a large number of simulations were carried out in consultation with the Trust. The concept design of an OSD system that controls site runoff in both the 50% AEP and 1% AEP storms was refined and various design issues addressed. The findings of these studies are outlined in a several reports (Cardno Willing, 2003, 2004).

The outcomes of these investigations that were adopted in the fourth edition included:

- A modified OSD storage volume (SSR_T) of 455 m^3/ha;
- All site runoff to be directed to the OSD storage: that is for the site the storage is on-line;
- The OSD system is to have two orifice outlets and a small spillway;
- The primary or lower orifice normally has a PSD of 40 L/s/ha (PSD_L) and located as close as possible to the storage invert;
- There is also a secondary orifice located at the base of a DCP providing HED with a PSD of 150 L/s/ha (PSD_U);
- The crest of the DCP is at the water level of the 50% AEP storm when the volume in the lower storage reaches 300 m^3/ha (SSR_L);
- The secondary orifice starts to operate when the water level in the storage exceeds the crest level and water starts to overflow into the DCP;

- To reduce the likelihood of the primary or secondary outlets being blocked by debris, the outlet opening should have a minimum internal diameter or width of at least 25 mm and should be protected by an approved mesh screen; and
- A small spillway of suitable length to prevent flooding of the residence/business if the outlets become blocked is provided at the top of the storage (i.e. at 455 m^3/ha)).

A concept general arrangement for the primary and secondary outlets is given in **Figure 5.11** with a typical section shown in **Figure 5.12**.

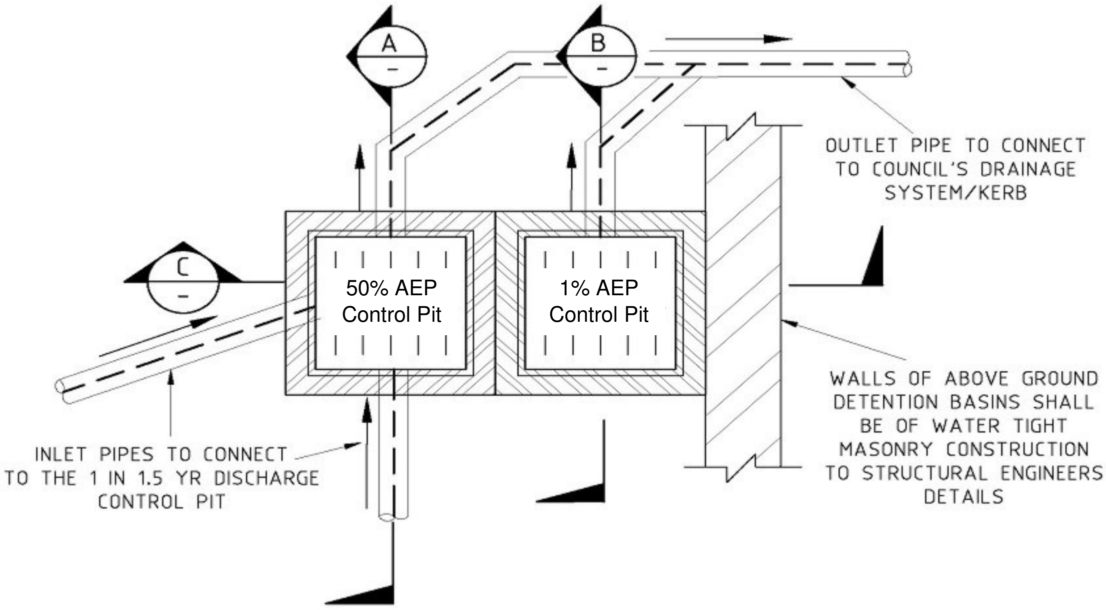


Figure 5.11 Concept Outlet Arrangement (after UPRCT, 2005)

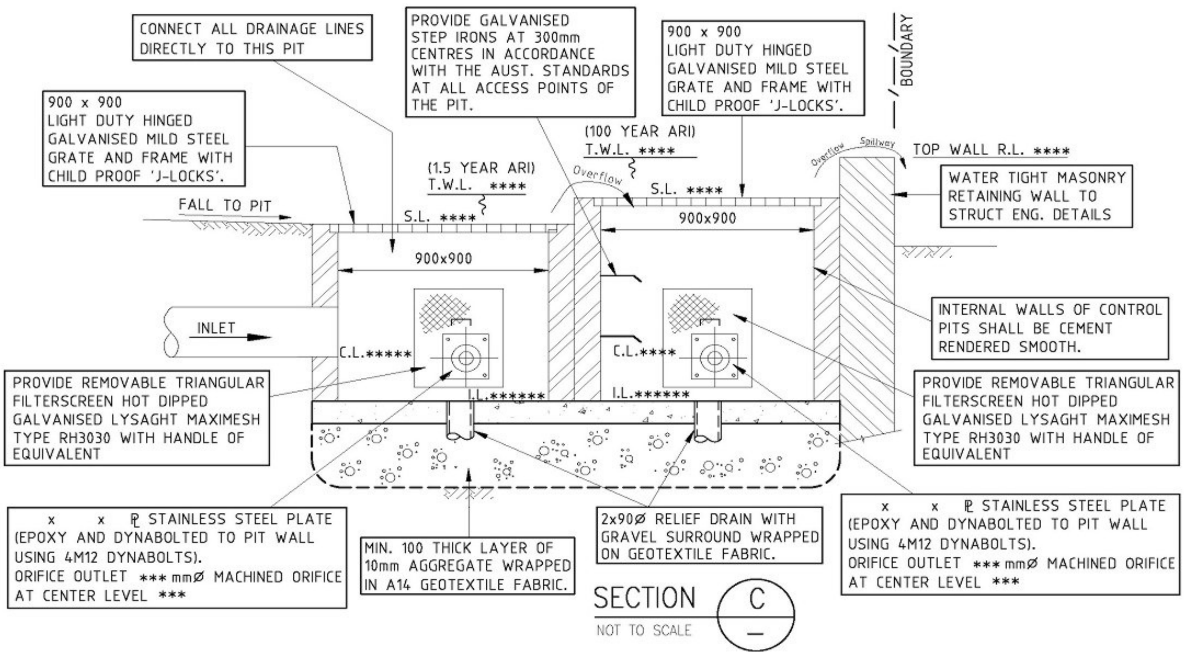


Figure 5.12 Concept Section of Primary and Secondary Outlets (after UPRCT, 2005)

Rainwater Tanks

In the past the four local councils and the UPRCT had always refused to allow rainwater tanks to be considered as part of an OSD facility, because of the probability that the tank would be full at the start of a major rainfall event. In recent years it has been argued, however, that a rainwater tank will not always be full at the start of a storm if its water is used inside and outside the dwelling for non-potable purposes – toilet flushing, laundry, hot water and garden watering.

Initial guidance was given in the *third edition* based on the research of Coombes et al, 2001 into what proportion of the volume of a rainwater tank can be counted as part of the site's OSD volume – assuming its water was used both inside and outside the dwelling. A 1,000-year rainfall record was generated based on a 53-year pluviograph record at West Ryde. This was applied to water-use models of different types of residences – a single dwelling, a duplex, a town house and an apartment building. The results of repeated simulations with different typical sizes of rainwater tanks (5, 10 and 15 m^3) showed that the average percentage of rainwater tank volume that can be counted as part of the site's OSD volume ranged from 32% to 50% if the tank had no airspace, and 51% to 72% if there was 50% air space. These results were considered to provide only an interim answer, because the study only looked at individual sites and did not investigate the cumulative impact on peak discharges from groups of dwellings with rainwater tanks.

As part of the detailed analyses of the cumulative impacts on peak discharges undertaken in 2003 and 2004, the interaction of rainwater tanks and OSD tanks was investigated. Analyses were undertaken of both rainwater tanks with dedicated airspace and dynamic airspace ie. airspace in a rainwater tanks that varies in response to rainfall and water demands (internal and/or external).

Dedicated Airspace

Based on the analysis of the results reported in Cardno Willing, 2004 the following reductions in the SSR values are allowed in the *fourth edition*:

- 50% of the dedicated airspace can be credited against the required extended detention volume (SSR_L);
- 100% of the dedicated airspace can be credited against the required overall detention volume (SSR_T);

subject to:

- a maximum dedicated airspace credit no greater than ratio of the area of roof discharging to the rainwater tank to the lot area times the overall site storage volume that is required;
- the rainwater tank has a dedicated outlet to ensure that the dedicated airspace is recovered after a storm event;
- the PSD for the dedicated rainwater tank outlet is no greater than 40 L/s/ha;
- all outflows from the rainwater tank (outflows from the dedicated outlet and overflows from the rainwater tank) are discharged to the OSD tank.

Dynamic Airspace

Based on the analyses of the results of various rainwater tank simulations undertaken in 2004 the following procedure is used in the *fourth edition* to calculate the reductions in the SSR values that are allowed. The rainwater tank dynamic airspace at the start of a storm is calculated using the following equation:

$$\text{Airspace } (m^3) = 8.7 \text{ x Tank Vol } (m^3)^{1.05} \text{ x Roof Area } (m^2)^{-0.5} \text{ x Demand } (m^3/d)^{0.35}$$

Daily demands can be estimated using the following average daily demands for Western Sydney, as reported by Coombes and Kuczera, 2003.

Outdoor (m^3/d)	**Indoor (Total) (m^3/d)**				
	No. of Occupants				
	1	2	3	4	5+
0.260	0.231	0.448	0.665	0.882	1.099

In the absence of detailed information on the proportion of various indoor uses for rainwater the following data was used for percentage of indoor demand: toilets (20%), laundry (25%) and hot water (25%).

The reduction in SSR values due to dynamic rainwater tank airspace is calculated using:

$$SSR_L = 300 - (1{,}950 \text{ x Airspace } (m^3)^{2.10} \text{ x Roof Area } (m^2)^{-1.50})$$

$$SSR_T = 455 - (1{,}650 \text{ x Airspace } (m^3)^{2.30} \text{ x Roof Area } (m^2)^{-1.50})$$

subject to:

- the development being residential, or its water usage can be considered to approximate that of a residence;
- the rainwater tank is plumbed into the household water supply system so that its water is automatically used for non-potable purposes;
- the design is in accordance with AS/NZS 3500.1.2: Water Supply – Acceptable Solutions (provides guidance for the design of rainwater tanks with dual water supply (rainwater and mains water)); and
- all overflows from the rainwater tanks are directed to the OSD tank.

Areas not directed to the OSD storages

Where possible, the drainage system should be designed to direct runoff from the entire site to the OSD system. Sometimes, because of ground levels, the receiving drainage system or because of other circumstances, this will not be feasible. In these cases up to 30% of the residual site area may be permitted to bypass the OSD systems, provided that as much as possible of the runoff from impervious site areas is drained to the OSD system. The residual site area is the area of the site excluding the roof area.

The storage volume is still calculated on the entire site area while the PSD is adjusted downwards in accordance with the values given in **Table 5.1**.

On-Site Detention Calculation Sheet

An On-Site Detention Calculation spreadsheet was assembled to ensure that calculations are undertaken by all OSD designers in a manner consistent with the procedures that have been described. The calculation sheet accounts for rainwater tanks (with or without dedicated airspace and dynamic airspace), possible bypass of the OSD storage and calculates the required OSD parameters based on input site data. It also undertakes a number of checks to ensure that appropriate data is entered and that resulting outlet orifice sizes meet the guidelines.

5.9.2 Heritage Mews, NSW

The "Heritage Mews" development applies the regime-in-balance stormwater management strategy which requires runoff volume from a developed site to be equal to its 'greenfields' discharge in the adopted critical design storm. The objectives of 'before-and-after' runoff volume equality and peak flow less than the permissible site discharge (PSD) were achieved using rainwater tanks, in-ground trenches and 'slow-drainage'. Continuous simulation was used to prove that the configuration of retention components planned for the development delivered the flow quantity objectives set by Council and, also, that the 3.0 kL rainwater tanks would provide some 22% of domestic water use. The development incorporated four "UniSAtanks" which provide a high standard of quality control to 95% of average annual flow (Argue et al (2003), Coombes et al. (2003)).

Introduction

The Heritage Mews site is located adjoining existing residential development in Castle Hill, Sydney's North West. The development consists of 62 large homes subdivided through community title. The site straddles a ridge which has an average longitudinal fall to the west of approximately 8% and transverse falls of approximately 11 % and 5% to the south and north respectively. The site is bounded by a new public road in the east and watercourses to the south and north which converge past the western end of the site as shown in **Figure 5.13**.

Soils on the site consist of silty clays overlying extremely weathered sandstone at about 1.0 m depth along the ridge increasing to over 2.5 metres towards the watercourses and the western end of the site. The ground water depth varied from 1.5 to 2 metres adjacent to the creeks and was well below the shallow sandstone layers along the ridge. At one location towards the western end of the site, in an area where the rock dipped away, the ground water was found to be under a relatively high "hydraulic head".

Being located within the Hawkesbury River Catchment the development must comply with permissible site discharge requirements as determined by the former Hawkesbury River Catchment Trust and now applied by Baulkham Hills Shire Council.

Analysis of Constraints

The constraints of the project and the site were assessed in consultation with the client/developer, Council, the technical experts and ourselves. These included:

- The site was generally too steep with insufficient room to consider the use of swales adjacent to carriageways;
- Density of development and location of services meant limitations on the size and location of rainwater tanks and limited opportunities for placement of trenches or swales within road reserves and/or lots;
- Planning and lot layouts had already been completed so there was no opportunity to realign or relocate roads;
- Presence of shallow rock in places necessitated a need to minimise excavation;
- Silty clay subsoils were relatively impermeable which made any infiltration devices ineffective;
- Similarly the underlying fractured sandstone layer was found to offer inadequate infiltration capacity;

Figure 5.13 Heritage Mews Site, Castle Hill, NSW

- Lack of readily available design tools made it difficult to present information to Council for their assessment;
- Lack of an accepted suite of details or acknowledged “good practice” made the preparation of detailed construction drawings a very time consuming process;
- The actual level and intricacy of detail required also consumed much of the design process;
- Being an alternate solution utilising new technologies required an alternate design approach;
- Time was, as with any commercial project, critical. The developer had made a commitment to the project but also had to face commercial realities. The project could not afford to get “bogged down” because it was delivering a WSUD solution;
- Capital costs of the WSUD solution had to be comparable to a conventional solution.

The Design Process

It became evident during the design process that our role of stormwater designers had changed. Traditionally stormwater systems are contained below ground and other than the design engineer no one else on the design team has much interest or input. However, rainwater tanks, drainage swales and other WSUD elements are visible and therefore everyone from the architect, landscape architect, builder, developer and marketing staff has an opinion that may impact on the use of the same. Compromises were necessary and certain elements were discounted, because of these concerns and the lack of sufficient data or history of performance to allay such concerns. Development in WSUD design and implementation since have changed perceptions significantly and solutions that previously were deemed unacceptable can now be readily implemented.

The issues that were considered during the design process included;

- the integration of building service hydraulics for the reticulation of stored rainwater;
- detailed layouts for downpipes and stormwater pipes and connections to rainwater tanks and gravel trenches;
- diligent separation of “clean” and untreated flows through detailing of “treatment trains”;
- coordination with landscape architect; and
- detailed review of site surface levels and landscape treatments.

The WSUD Approach

The integrated stormwater management system comprises a series of retention/detention storages distributed and connected in series throughout the catchment. Each of these devices assists in the control of both the quantity of runoff, through on-site retention and the quality of runoff through stilling and filtering runoff.

The OSR storages were determined independently by Dr Peter Coombes using the Probabilistic Urban Rainwater and Wastewater Reuse Simulator (PURRS) and Adjunct Professor John Argue using design storm events. Whilst the different approaches yielded different distributions of storages across the site the total storage requirements determined by both were within 10%. The resultant effect of this alternative approach is that peak site discharges are reduced to that required by The Hills Shire Council for sites within the Hawkesbury River catchment.

The stormwater system begins with the collection of roof water from each dwelling in a separate rainwater tank where it is stored and used for flushing toilets and outdoor uses. When full, rainwater tanks overflow to a slow drainage gravel trench which stores, filters and slows the overflows together with the runoff from yards and driveways before it is released to the adjacent creek.

Runoff from the roads is collected in “UNISATANKS”, where gross and fine pollutants are collected, before overflowing to slow drainage gravel trenches for further filtering into the “Atlantis” drainage cells where the runoff is stored before being released in a slow “controlled” manner.

Rainwater Tanks

From the outset an integral part of the WSUD for Heritage Mews was to include rainwater tanks (refer **Figure 5.13**). However, between the concept and final design there was much debate, not so much on technical grounds but on issues of amenity of occupants and aesthetics involved with the proposed rainwater tanks and associated pumps.

For example the question of rainwater tank size became not only a question of stormwater benefit versus water conservation benefits, but the type and location of tank made the rainwater tank an architectural, sales and marketing issue as well. It was considered essential to minimise the impact on the small yard areas. The ultimate choice of tank type also impacted on the proposed mains top up arrangement.

The option of undergrounding the rainwater tanks was considered during the design. At the time Sydney Water however, had no appropriate policy or customer agreement for the use of underground rainwater tanks fitted with a mains top up. We were unable to resolve the issues associated with such an installation, in a suitable time frame so underground rainwater tanks were not considered, irrespective of cost.

The modelling of the rainwater tanks and the opportunities for effective retention storages was undertaken by Dr Peter Coombes using the PURRS (Probabilistic Urban Rainwater and Wastewater Reuse Simulator) water balance model (Coombes and Kuczera, 2001). The preparation of initial details for the tanks was undertaken by Dr Peter Coombes with further modifications, as a result of the type of tank selected and Sydney Water requirements, documented by the designers as shown schematically below in **Figure 5.14.**

The roof water of all dwellings is passed over leaf guards in the eaves gutters and directed through a sealed, charged system via a first flush arrangement to a 3kL modular rainwater tank which was selected because it was felt by the client, architect and the sales people to be the most appropriate above ground tank option.

The 3kL tank was selected on architectural grounds, more so than technical grounds and therefore there was not as much stormwater benefit gained as would have occurred with say a 5 or 10kL tank. It was therefore necessary to provide a “trickle outlet” which slowly lowers the water level in the tank, when it is full, to provide an “air-gap”, or capacity to accept runoff from the next storm event.

The stored water level within the tank is constantly being drawn down below the “air-gap” by toilet flushing and outdoor usage. When the level of rainwater within the tank gets to 300 mm above the base of the tank, as determined by a float switch, a solenoid valve engages the Sydney Water mains top-up.

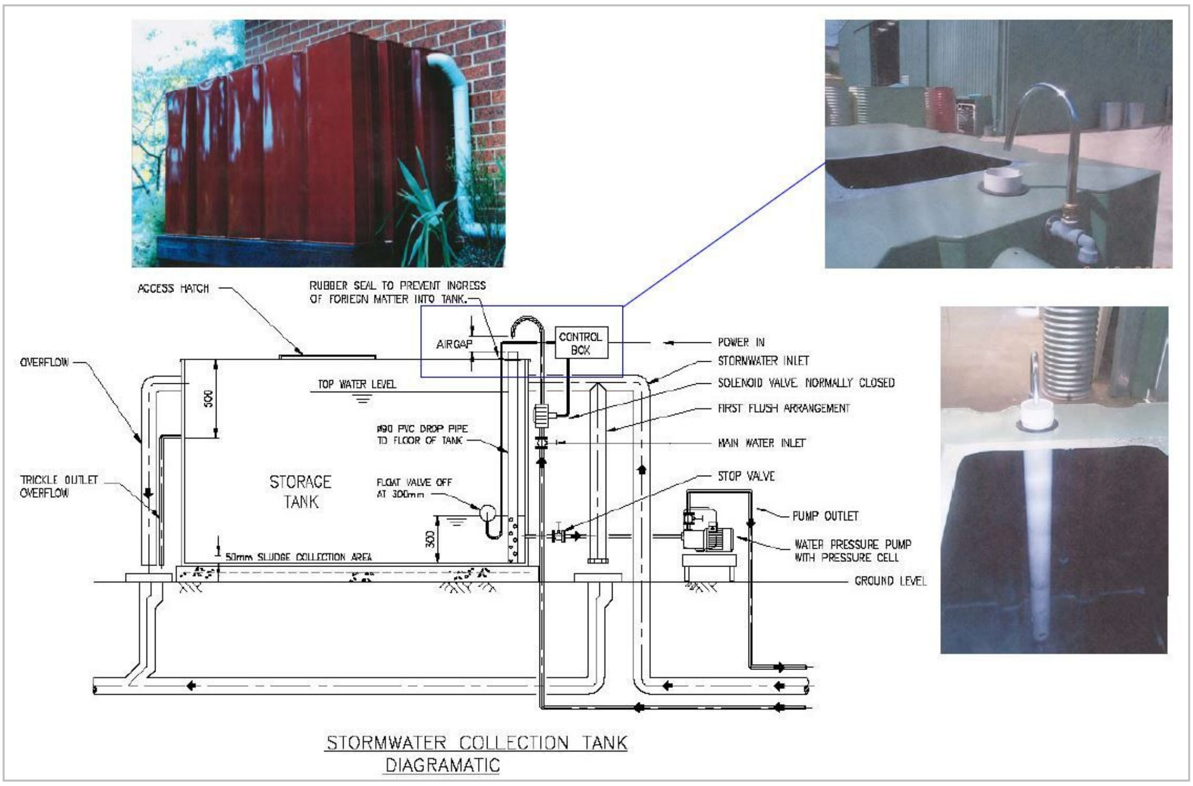


Figure 5.14 Typical Rainwater Tank Arrangement

At the time Sydney Water's cross connection policy required a visible air-gap between the outlet of the mains top-up and the top of the tank. This arrangement was in excess of the requirements of the National Plumbing and Drainage code AS3500. These requirements has since changed and been updated.

Stored rainwater is reticulated back into the dwelling via a small pump, fitted with a pressure cell, which will typically provide two half flushes to the toilet. There are also external, outlets provided for outdoor and garden uses. When the rainwater tanks are full they overflow to a slow drainage gravel trench.

Slow Drainage Trenches

The slow drainage gravel trenches collect, filter and store all overflows from rainwater tanks together with filtered runoff from yards and driveways (see **Figure 5.15**). The overflow from the rainwater tanks is considered clean and discharges directly into the gravel trenches. Runoff from yard areas and driveways are passed through silt arrestor pits fitted with an "Enviropod" filter to trap gross pollutants and fines before entry into the trenches.

The runoff from the roads is collected in kerb inlet pits fitted with trash screens to collect the gross pollutants and then passed through a sediment trapping pit, which traps the fine sediments and pollutants before ultimately discharging to the slow drainage trench for further filtering. This arrangement from inlet to gravel trench is referred to as a "UNISATANK" as developed by the University of South Australia and others.

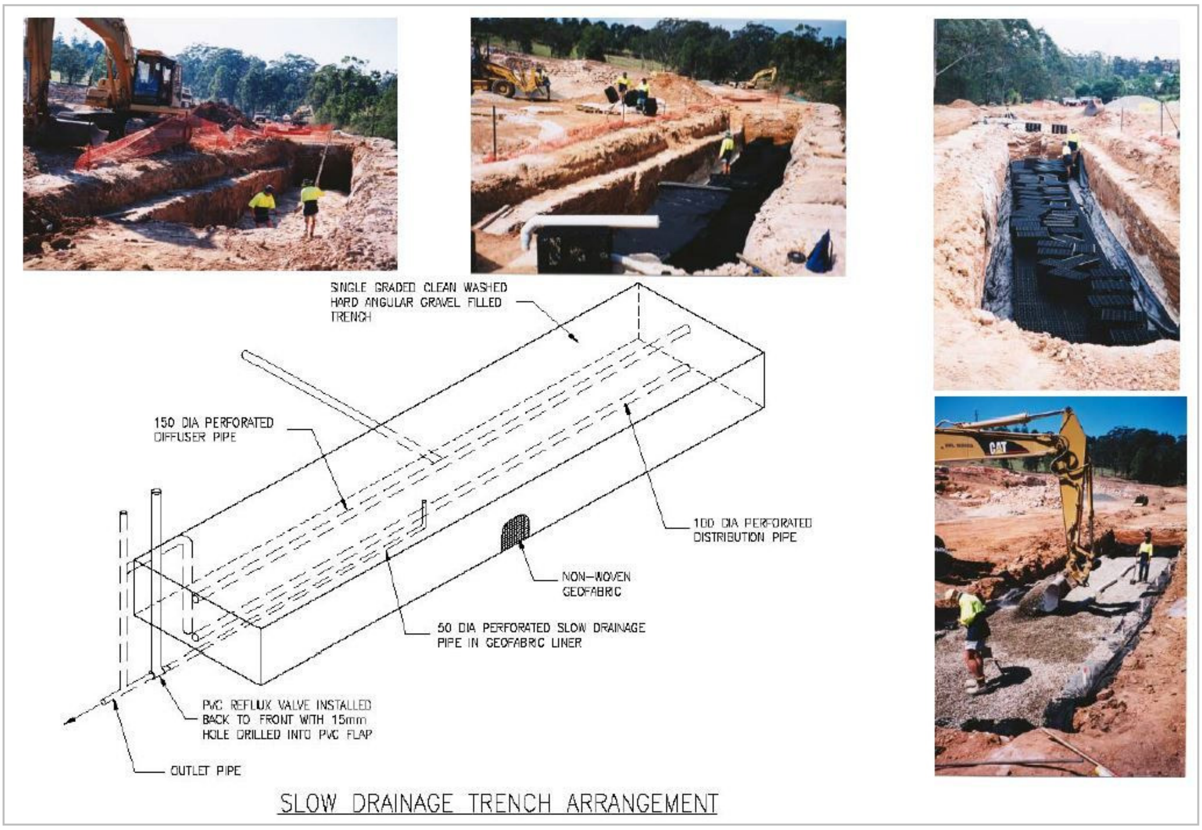


Figure 5.15 Typical Slow Drainage Trench Arrangement

The slow drainage trench arrangements and details were a collaborative effort between the University of Adelaide and the designers. The trenches were sized in accordance with recommendations by Argue, 2002 to store the runoff generated in the critical storm event for the overall catchment (ie 10 year ARI storm event with a duration of 2 hours).

Since the soil infiltration rates did not allow the trenches to empty within a reasonable period of time it was necessary to provide a slow drainage outlet for all trenches to ensure they emptied within 24 hours.

The typical arrangement of the slow drainage gravel trenches is shown in **Figure 5.15** and comprises;

- a trench filled with a single graded, clean, washed, hard, angular gravel wrapped in a non-woven geofabric;
- an inlet which connects directly to a slotted diffuser pipe wrapped in a geofabric sock at the top of the trench to evenly spread the inflow;
- a slotted distribution pipe wrapped in geofabric at the base of the trench to prevent stored flows from collecting at one point;
- a slow drainage outlet which allows the trench to empty within a 24 hour period The slow drainage outlet utilises a reflux valve inserted against the flow and a small bore hole to slowly "control" the outlet flow. This arrangement provides access for inspection of the outflow and maintenance or cleaning of the outlet; and
- in the event of a blockage or a major storm event, the trench is provided with an overflow pipe which allows any surcharge to be controlled and directed to the outlet.

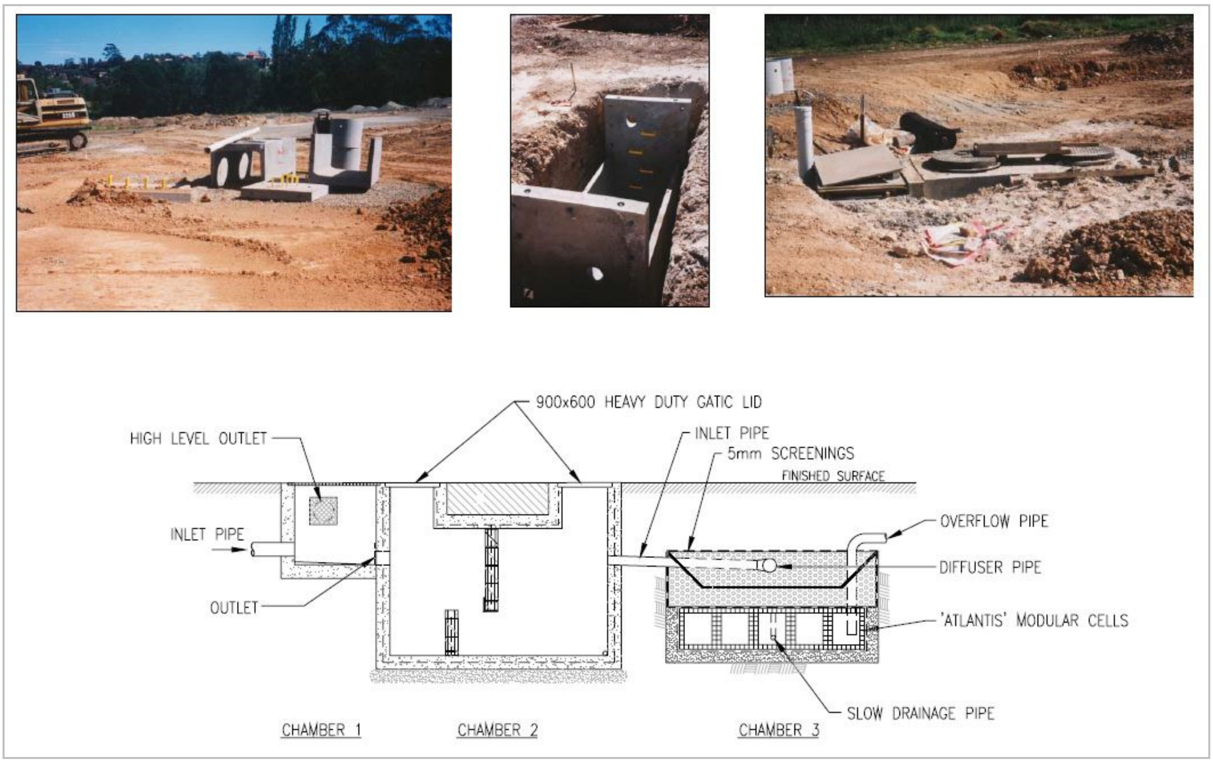


Figure 5.16 Typical UNISATANK Arrangement

The composite gravel over "Atlantis" drainage cell slow drainage trench arrangement was similar to the gravel trench with the exception that the distribution pipe was excluded.

UNISATANKs

The concept design for the "UNISATANKS" were provided by the University of Adelaide. The construction details were prepared by the designer in consultation with the University of Adelaide (see **Figure 5.16**). The sedimentation chambers of the "UNISATANKS" as constructed were later manufactured as precast units at the direction of the roadworks contractor.

"UNISATANK" refers to the arrangement of an inlet pit to trap gross pollutants (Chamber 1), a sediment trapping tank (Chamber 2) and the filtering/storage trench (Chamber 3) connected in sequence to treat runoff from road carriageways. The "UNISATANKS" at Heritage Mews are shown in **Figure 5.15** and comprise:

Chamber 1	Kerb inlet pits fitted with trash screens to collect gross pollutants.
Chamber 2	A precast sedimentation tank with baffles to collect fine sediments. There is access to each end of this chamber for cleaning and maintenance.
Chamber 3	A composite gravel trench over "Atlantis" cells to filter and store the collected runoff before slowly discharging same to the outlet.

Maintenance

The Heritage Mews project is subdivided by community title and as such the ongoing maintenance will be the responsibility of the Community Association.

A detailed schedule outlining the role of each element within the system, the maintenance procedure and responsibility for each was prepared and was included within the Management Statement attached to the title of each lot and administered by the Community Association.

Whilst the ongoing success of the WSUD elements will be dependent upon regular maintenance, it should be noted that all conventional drainage infrastructure, including on-site detention tanks and gross pollutant devices, also need regular maintenance to perform as designed.

5.9.3 William Slim Drive PMF Detention Basin, ACT

In 2006 the ACT Government investigated the possible construction of a PMF detention basin on Ginninderra Creek floodplain adjacent to William Slim Drive and upstream of the suburb of McKellar in Canberra, ACT (refer **Figure 5.17**). This basin in combination with minor upgrading works of the downstream Lake Ginninderra dam embankment to be undertaken by Roads ACT was intended to reduce the estimated PMF flows at the Lake Ginninderra dam embankment to “safe levels”. This system in essentially an Infrastructure Compliant Management Strategy with a focus on a Category 1 solution. It was estimated that the PMF overtopping of the Lake Ginninderra dam embankment would be reduced to below 0.3 m which is unlikely to initiate a breach and dam break and thus can be termed “safe”.

In 2004, Bill Guy & Partners completed a draft Feasibility Study Report on a William Slim Drive Retardation Basin and concluded that Option 2 (refer **Figure 5.18**) was the preferred option. A review of this option highlighted a number of concerns regarding its sustainability.

In view of these concerns further hydrological and hydraulic investigations were undertaken to establish if there was a further option that can address these concerns while still achieving the required reduction in PMF flood levels in Lake Ginninderra (Phillips et al., 2007, 2008).

Assessment of Options

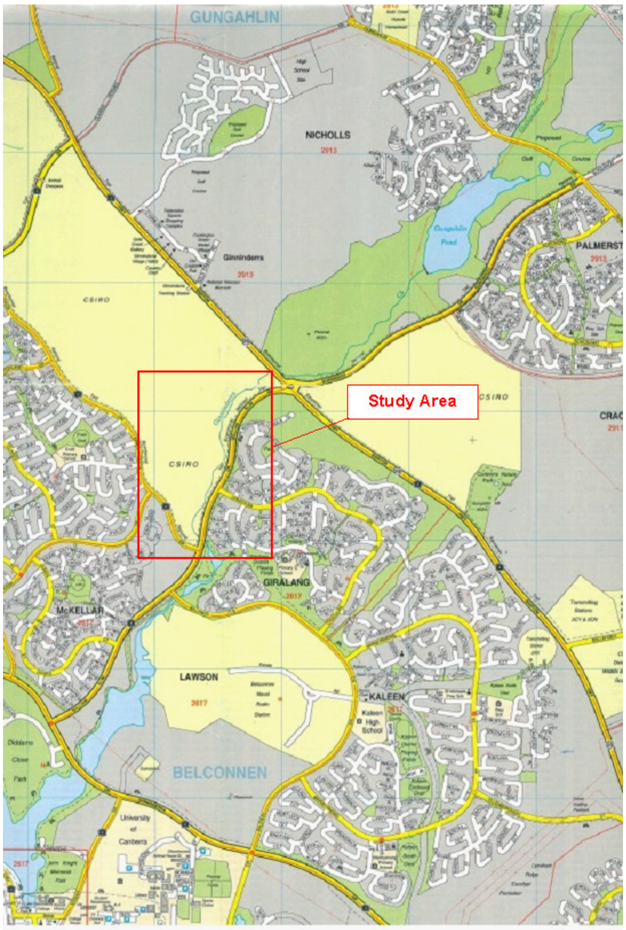
The final adopted configuration for the basin outlet and associated downstream energy dissipation measures evolved through a series of modelling assessments and physical model testing that are outlined as follows. The peak PMF inflow to the options is 1,909 m^3/s.

Option 2

As described in Cardno Young, 2005 the key features of Option 2 included:

- A 1.7 km long embankment that is typically 12 m high;
- An embankment crest level of 596 m AHD + a 0.75 m high crest wall;
- Equivalent upstream and downstream slopes of 1 (V): 1.75 (H);
- A primary outlet comprising 2 x 5 m (W) x 3 m (H) RCBCs that are 35 m long; and
- A secondary outlet comprising an elevated trapezoidal concrete line spillway with a crest level of 589 m AHD (around the 0.1% AEP flood level).

It was also noted that the Option 2 outlet configuration stored a considerable volume of floodwaters on the rising limb that contribute to the overall size of the basin. This suggested that there might be an alternative basin outlet arrangement that ideally does not retard flood flows up to say 1,000 m^3/s and then retards higher flows to achieve the PMF objective in Lake Ginninderra. Consequently a range of alternative concepts were explored leading to the detailed assessment of Option 3.



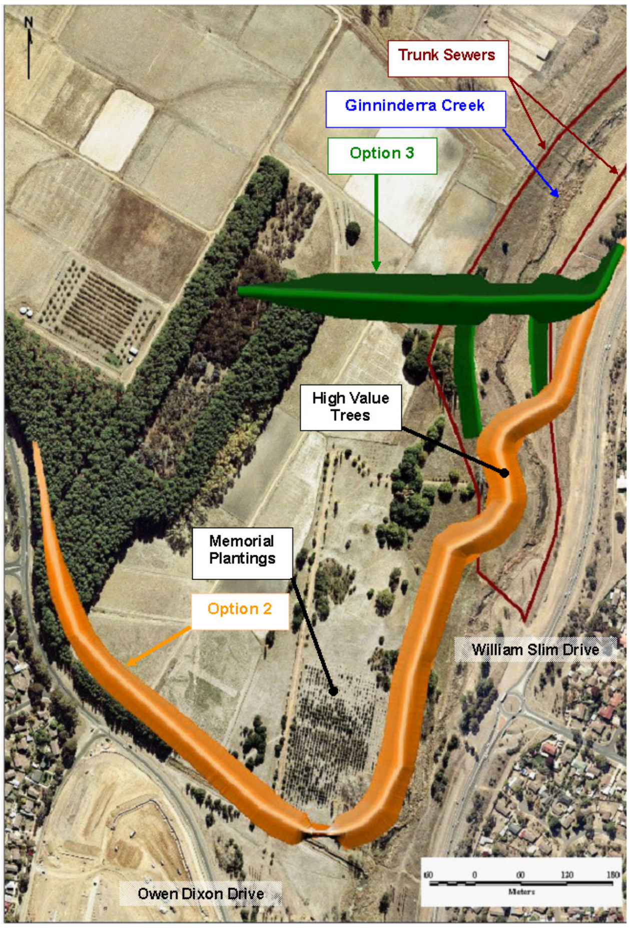


Figure 5.17 Locality Plan **Figure 5.18 Alignment of Option 2 and Option 3**

Option 3

The key features of Option 3 included:

- Relocating the main embankment north to avoid conflicts with heritage items and Exceptional/High Value trees;
- A 0.5 km long embankment that is typically 10 m high (refer **Figure 5.18**);
- An embankment crest level of 595 m AHD without a crest wall;
- Upstream and downstream slopes of 1 (V): 3 (H);
- A primary outlet comprising 2 x 21 m (W) x 7 m (H) Bebo Arches that are 40 m long in combination with a 200 m long flume that varies in width from 60 to 80 m (refer **Figure 5.19**)

While the outlet gave a peak basin level of around 594.5 m AHD and a peak outflow of 1,650 m^3/s it was at the expense of outlet velocities of 7-10 m/s.

A comparison of the performance of Options 2 and a range of alternative options concluded that the advantages of Option 3 over Option 2 included:

- No destruction of High Value trees on the Ginninderra Creek floodplain;
- No disturbance of the Memorial Landcare plantings;
- No changes to access to the Memorial plantings particularly for any disabled visitors;

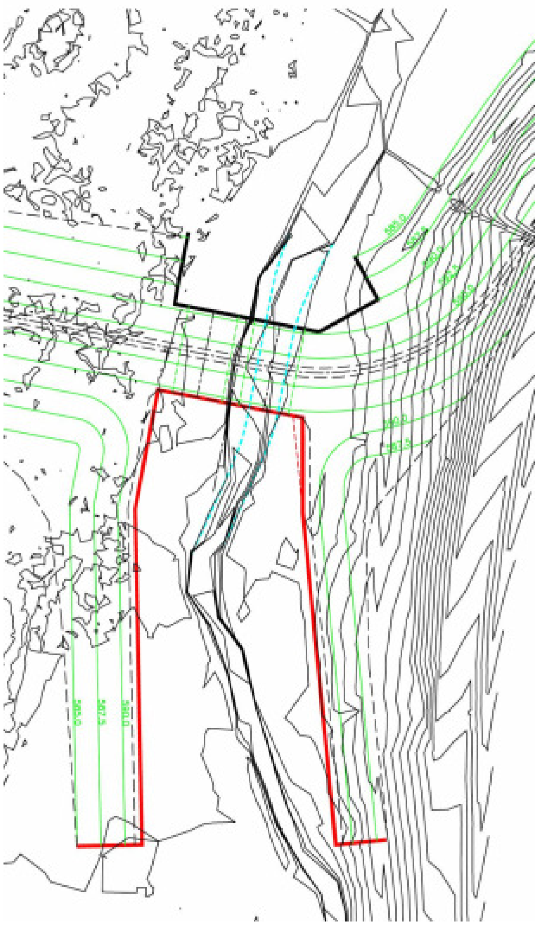


Figure 5.19
Option 3 Concept Basin Outlet Configuration

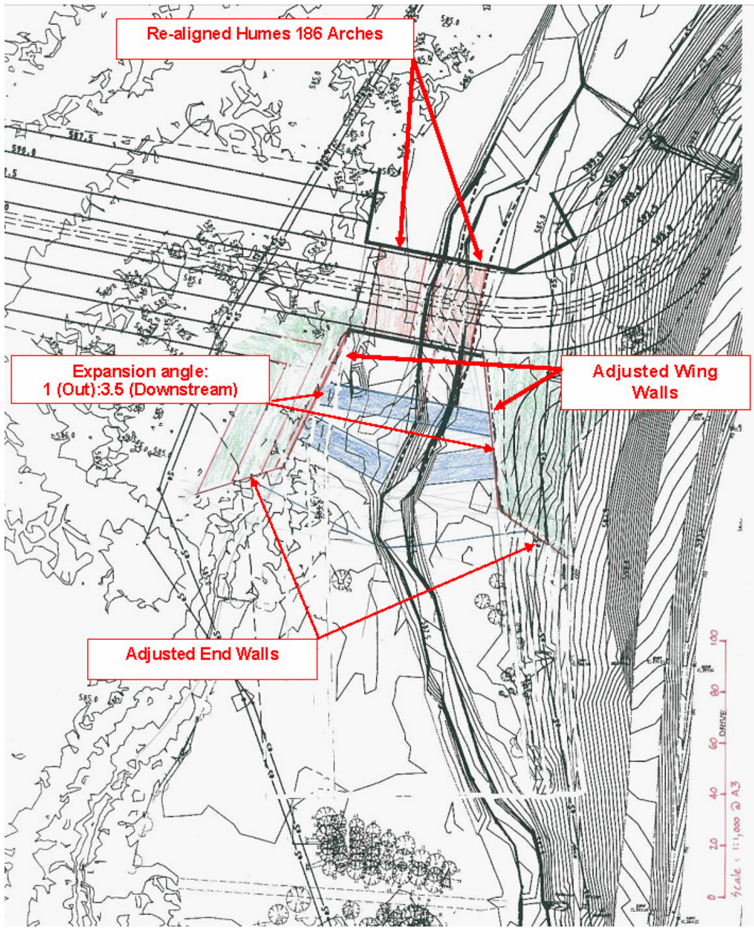


Figure 5.20
Option 4 Concept Basin Outlet Configuration

- No aesthetic impacts on new residences fronting Owen Dixon Drive because the embankment is relocated around 750 m north of residences;
- Reduced impacts on movement of aquatic fauna of a 40 m long but 7 m high Bebo arch installed over Ginninderra Creek; and
- More feasible maintenance of an earthen embankment with upstream and downstream slope of say 1 (V): 4 (H).

Initial Physical Model Testing of Option 3

It was found during the hydrological and hydraulic assessments of Option 3 that the estimated peak PMF basin water level was sensitive to adopted inlet and outlet losses for the twin Bebo arches. This sensitivity supported the need to undertake physical model testing of the Option 3 outlets to allow the outlet configuration to be refined and to increase the confidence in the estimated peak PMF basin levels.

As described in UNSW WRL (2006), a physical model was assembled in the inlet and outlet areas and a section of embankment based on the adopted Option 3 basin configuration (refer **Figure 5.19**).

The initial physical model results and observations of the model disclosed a number of issues of potential concern for the concept outlet arrangement. These issues included part full flow through the arches and high velocities and standing waves that formed downstream in the flume.

Subsequent "calibration" of the hydraulic model to the initial physical model results indicated that the concept outlet arrangement would be unlikely to achieve the flow retardation objective ie. the PMF water level in the basin would not reach RL 596 m AHD thereby not maximizing the usage of the available airspace for reduction of the peak PMF flow.

Option 4

The "calibrated" hydraulic model was then used to test modified outlet arrangements to guide the proposed modification of the physical model. The primary modifications were as follows (refer **Figure 5.20**):

Amended outlet dimensions

- Reduce the size of the arches from Humes 217 arches to Humes 186 arches (18 m (W) x 6 m (H))
- Raise the invert levels of both arches
- Reduce the depth of the re-aligned channel
- Retain the existing height of the wing walls but reduce the width of the headwall

Amended Outlet Configuration

- Substantially reduce the length of the wing walls
- Change the angle of the wing walls to an expansion of 1:3.5 in the downstream direction
- Change the angle of the end walls to 45^{o} to the wing wall

Conceptual energy dissipation zones

Based on the flow velocities and flow patterns observed downstream of the current model outlet it was concluded that there was a need to create an energy dissipation zone close to the arches to dissipate energy and to drown the outlets in a PMF to reduce flow velocities through the arches. Two potential dissipation zones were identified:

> *Primary dissipation zone* - The primary zone was located around 20 to 30 m from the headwall. The measures could be dissipator blocks, dissipator columns and or a solid weir. The measures would need to extend across the existing channel. The measures may not extend all the way to the wing walls.
>
> *Secondary dissipation zone* - The secondary dissipation zone was located 40 to 50 m from the headwall and is perpendicular to the wing walls. The measures could be dissipator blocks and or a solid weir. The measures would not extend across the existing channel.
>
> *Scour protection zone* - Downstream of the second line of dissipators it is possible that a hydraulic jump will form in extreme floods and scour protection measures may need to be installed. These measures could include armour rock, grouted rip-rap and/or a concrete apron. It is expected that a concrete apron would need to be constructed under the arches and on the creek banks from the headwall to a point downstream of the secondary dissipators. It was expected that the creek would need to be lined with armour rock that may need to be grouted in places to tie into the concrete aprons.

Table 5.8 Advantages and Disadvantages of Outlet Modifications

Modification	Advantage or Disadvantage
Reduced length of wing walls	**Advantage:** Cost saving and reduces the interference with existing sewers. Also reduces the visual impact of the longer flume.
Increased splay of the downstream wing walls	**Advantage:** Improves the spread of the flows across the floodplain. Reduces the average flow velocity at the end of the wing walls. Reduces the likelihood of an adverse impact on existing valued trees.
Reduce the size of the twin arches	**Advantage:** Cost saving **Disadvantage:** Increases the flow velocity through the arches due to reduced waterway area
Inclusion of energy dissipation measures in outlet flume	**Advantage:** Will dissipate energy in a controlled manner rather than relying on vegetation that may fail in high velocities **Disadvantage:** A harder engineering solution that adds cost
Inclusion of scour protection in the form of concrete aprons or grouted armour rock.	**Advantage:** Will protect against scour in energy dissipation zone **Disadvantage:** Replaces the softer vegetation approach with a harder finish that adds cost. May require dissipator columns in the creek

The anticipated advantages and disadvantages of the modified outlet arrangement are summarised in **Table 5.8**.

The physical model was modified in accordance with Option 4 and further tests were undertaken. One of the aims of the additional model runs was to identify a practical (and if at all possible an aesthetic) energy dissipation system.

The results of these tests are reported in WRL, 2006 (referred to as Design Modification 1). The results of the physical modelling of Option 3 and Option 4 (without dissipators) are compared in **Table 5.9** and in **Figure 5.21**:

Table 5.9 Comparison of Option 3 and Option 4 Stage-Discharge Rating Curves

Option 3		Option 4	
Discharge (m3/s)	Upstream WL (m AHD)	Discharge (m3/s)	Upstream WL (m AHD)
1909	595.97	1909	
1494	592.93	1494	596.31
1001	590.22	1001	592.28
		737	590.48
		500.3	589.00
		250	587.26
148	585.64	150	587.21

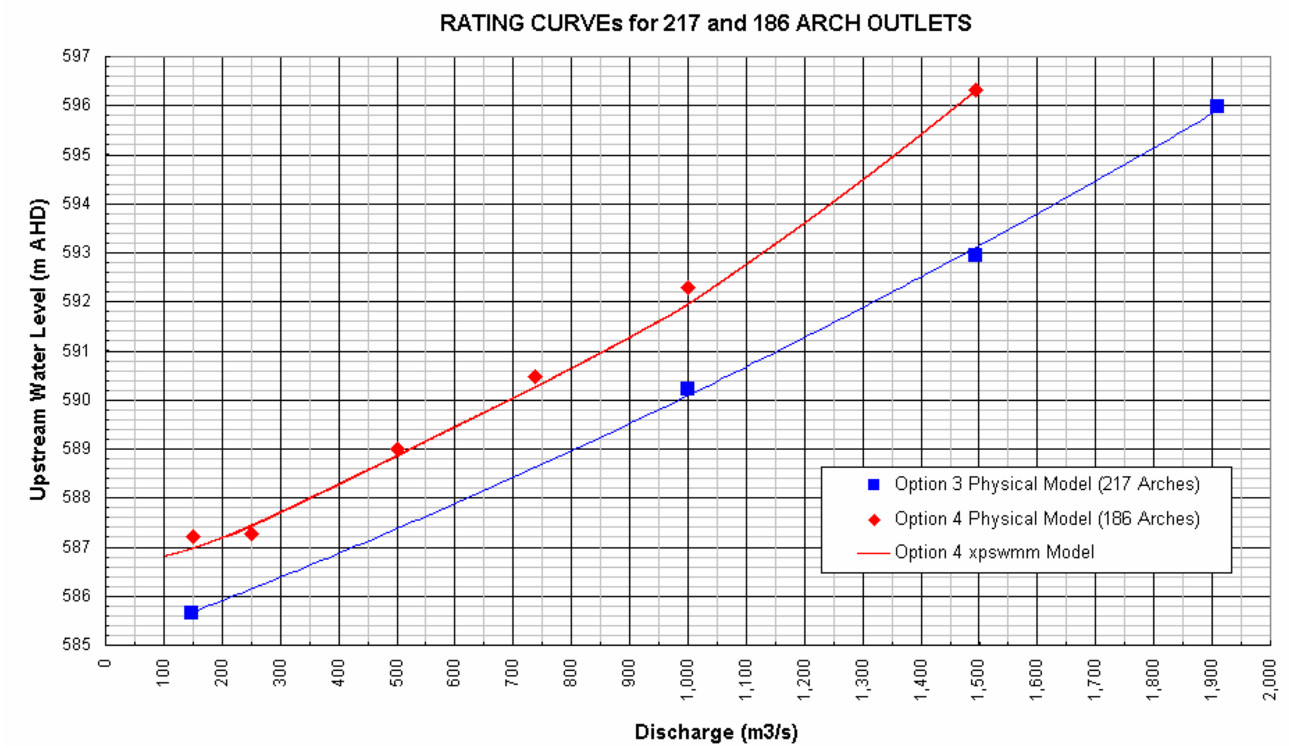


Figure 5.21 Comparison of Option 3 and Option 4 Stage-Discharge Rating Curves

Option 5

Visual observations of a model run of the PMF with the preferred dissipator columns in place (a double row of dissipator columns 10 m apart comprising 2 m x 2 m columns at 4 m centres) showed that the dissipator columns work well and distribute the flow uniformly between the wing walls. An issue of concern was the high velocities on the floodplain (around 10 m/s) downstream of the dissipator columns. These high velocities are attributed in part to flow continuity. ie. the flow can only expand at around 1(sideways):4(downstream) and all 1,500 m^3/s is concentrated into a single 80-90 m wide jet downstream of the dissipator columns.

It was concluded that the only way to more evenly downstream velocity across the floodplain would be to separate the two arches ie. **Option 5**. One arch would be aligned with the creek while the other arch would possibly be aligned beside the trunk sewers west of the creek. The width of the headwall of the outlet would change from a single headwall (54 m wide) to two separate headwalls (say each around 29 m wide). A similar arrangement would be required for the wing walls and dissipator columns for each separate arch outlet.

An approximate model test was undertaken using the physical model by halving the PMF flow and blocking one arch. The flow distribution downstream of the dissipator columns was not uniform but was nevertheless promising. The downstream velocity on the floodplain was around 5 – 8 m/s. The downstream tailwater level was broadly adjusted to represent the full PMF flow through two separate arches. The results of this test are reported in WRL, 2006 (referred to as Design Modification 2).

Based on the outcomes of a series of modelling assessments and physical model testing it was recommended that Option 5 be adopted for William Slim Drive Basin.

Conclusion

It was concluded that the hydraulic assessments delivered an improved PMF basin configuration that avoids adverse impacts on Exceptional and High Value trees on the Ginninderra Creek floodplain; protects heritage items on the floodplain and existing Memorial Landcare plantings. It was also concluded that physical models continue to have a valuable role in the design of major hydraulic structures in urban areas particularly when predicted flood levels and flood storage are sensitive to assumed values of model parameters.

5.9.4 Gosnells Case Study, WA

This case study demonstrates the application of the Infrastructure Compliant Stormwater Management (ICSM) approach outlined in Section 5.2. It outlines the assessments undertaken to formulate a strategy to maintain the performance of existing conveyance infrastructure by controlling runoff from each development site with the LGA. It is based on the research previously described in Argue and Tennakoon (2011) and Botte et al. (2015).

Introduction

The City of Gosnells, Western Australia, is a municipality serving a population of around 100,000 people located 15 km south-east of Perth CBD encompassing an area of 127 km^2 on the edge of the Perth Plain and extending into the foothills of the Darling Ranges. The City was established in 1907 as one of the earliest population areas away from Perth and Fremantle. Unlike most of the other municipalities making up the Greater Perth Region, soils in the Gosnells area range from deep sands through peaty sands and sandy clays to gravelly sandy clay, granite and laterite – the latter soil/geological conditions being encountered in the foothills region of the City.

This range of soils is in sharp contrast to the fairly uniform deep sands which characterise soil conditions of the bulk of other Perth municipalities along the seaboard and on the sandy plain. Common practice in the Perth region is for roof runoff to be diverted to “soakwells” which pass the stored flow, by percolation, into the local sands and, ultimately, the unconfined aquifers (water table). The prime focus of residential street drainage infrastructure – in such circumstances – is therefore to cater, almost entirely, for runoff generated within road carriageways, only, and from connected paved areas such as allotment driveways. The soil/geological situation in Gosnells support this common practice, but only in about half of the area within the municipality.

The City has attracted increased business and industrial activity in recent years and, associated with it, increased population. The resulting development has the potential to overload the existing street drainage network in many areas.

This magnitude of the issue and potential solutions was assessed in a number of steps as follows:

- A survey and assessment of Council’s drainage networks was undertaken. This identified which drainage lines were operating within capacity and which drainage lines were operating beyond capacity;
- The next step was to assess the drainage systems augmentation works which would be required to cope with additional runoff from current and planned future development. The estimated cost of the drainage augmentations was around $120 million.

- In view of the magnitude of the capital expenditure which would be required the City of Gosnells decided to explore an alternative stormwater management strategy based on stormwater source control whereby post-development discharges to the drainage network would not be increased beyond the capacity of the existing drainage system ie. an Infrastructure Compliant Stormwater Management (ICSM) strategy.

By relying on source control stormwater quantity solutions Council aims to achieve an internalisation of development costs in relation to stormwater management, maintenance and renewal in order to minimise adverse equity implications to the broader community and the environment.
Furthermore, the approach encourages Water Sensitive Urban Design (WSUD) options, such as rainwater tanks, raingardens, bio-retention and green roofs etc. at the lot scale, thereby benefitting from the synergies that a site-based approach can have for catchment wide water quality improvement.

Decision-Support Matrix for Strategy Selection

The selection of best practice stormwater management strategies requires consideration of multiple factors, such as soil permeability (classification of soils based on infiltration capacity), catchment management objectives (objective function) and site characteristics (scale), to name but a few. However, the following aspects also need attention and may require consideration to achieve a balance between quantity and quality management objectives and to create a sustainable outcome: target pollutants, social values, capital and operating costs.

When considering the wide range of competing factors and the potential benefits and limitations of each available stormwater management approach, it was decided that a strategy selection matrix could assist in simplifying the decision making process. One possible matrix has been prepared and is shown in **Table 5.10**.

Table 5.10 Decision-Support Matrix for Stormwater Management Strategy Selection
(after Botte et al, 2015)

No	STRATEGY (Best Management Practice (BMP))	PERMEABILITY CATEGORY (m/day)				SCALE				OBJECTIVE FUNCTION		
		1.56 <	0.48 <1.56	0.12 < 0.48	< 0.12	Lot	Street	Precinct	Regional	Water Quantity	Water Quality	Water Conservation
		Very High	High	Moderate	Slow							
	Infiltration Systems - I											
1	Soakwells I1	√	√	~		√	√			R	√	~
2	Pervious Pavement I2	√	√	~		√	√			R	√	~
3	Infiltration Trenches I3	√	√	~		√	√	√		R	√	~
4	Infiltration Basins I4	√	√	~				√	~	R	√	~
	Conveyance Systems C											
5	Swales and Buffer Strips C1	√	√	√	√	√	√	√	√	C R D	√	~
6	Bioretention Systems (Swales, Basin) C2	√	√	√	√		√	√		C R D	√	~
7	Rain Gardens C3	√	√	√	√	√	√			C R D	√	
8	Sand filters C4	√	√	√	√		√	√		C R D	√	~
9	Retention trenches C5	√	√	√	√		√			C R	√	
10	Living Streams C6			√	√			√	√	C R D	√	~
	Detention Systems D											
11	On-site Detention system D1			√	√	√	√	√	√	D		
12	Dry/Ephemeral Detention Areas D2	√	√	√				√	~	D	√	
13	Ponds and Lakes D3			√	√			√	√	D	~	
14	Wet basin D4			√	√			√	√	D	~	
15	Constructed Wetlands D5	√	√	√	√			√	~	D	√	
	Stormwater Storage and Use S											
16	Rainwater Storage Systems S1	√	√	√	√	√				R		√
17	Managed Aquifer Recharge S2	√	√	√	√	√		√	√	R		√
	Pollutant Control P											
18	Litter and Sediment Management P1	√	√	√	√	√	√	√			√	
19	Hydrocarbon Management P2	√	√	√	√	√	√				√	
20	Sediment Basin P3	√	√	√	√			√		R	~	

Note: √ BMP is applicable , ~ BMP is applicable to some extent, R = Retention, D = Detention and C= Conveyance

This matrix provides key guidance to the designers on the selection of sustainable infiltration based best practice stormwater management strategies and also allows identification of a suitable combination of these measures to allow adaptation to locally prevailing circumstances. The City's drainage strategy selection matrix, together with spatial mapping of suitability for infiltration based stormwater disposal have become a useful tool for land developers as well as statutory authorities, decision makers and policy makers within the City to ensure delivery of functional and sustainable land development proposals.

Following extensive empirical research of local soil types and drainage conditions, including site based and laboratory soil and permeability investigations, and subsequent identification of a range of suitable at-source stormwater management approaches, a spreadsheet based design calculator was developed for use by developers and consultants active in the process of growth development within the City of Gosnells.

The ICSM Strategy

The new strategy for stormwater management in Gosnells was based on the following decisions/practices:

1 (a) Street drainage networks, generally, are near or beyond capacity; additional development on allotments greater than 350 m2 must therefore be designed – generally – to fully retain "100-years" (ARI) storms with no outflow to (fronting) street drainage systems, if at all possible. [The retained storm runoff will be stored temporarily in „soakwells‟ (common practice in Perth sandy soils).

(b) Practice (a), above, is relaxed, however, in certain areas of the City, in particular, those areas characterised by clay and silty soils which have low percolation capability. In these cases a small outflow is permitted equivalent to a maximum permissible pre-development flow from the lot area calculated using 20% AEP and storm duration given by the allotment time of concentration, typically 20 minutes. The runoff coefficient adopted for calculating this flow is C = 0.143. [Concrete, circular pipes (in-ground) are to be used for temporary retention of storm runoff in these cases.]

2. All properties smaller than 350 m2 are allowed to discharge a small outflow to the (fronting) street drainage system. This outflow is set at equivalent to a maximum permissible pre-development flow from the lot area with 20% AEP and storm duration given by the allotment time of concentration, typically 15 minutes. The runoff coefficient adopted for calculating this flow is C = 0.143;

3. Additional assumptions/practices:
   - All roof areas and connected paved areas are connected to in-ground "soakwells";
   - Pervious and unconnected paved areas do not contribute stormwater to "soakwells‟;
   - Design runoff volume is determined using rainfall intensities drawn from the full range of storm durations – 6 mins to 72 hours. This produces the greatest runoff volume which must be (temporarily) stored and, therefore, the greatest number of in-ground retention devices; the optimum storm duration given by this process is called critical storm duration.

This interpretation of critical storm duration based on the "...full range of storm durations..." instead of on values derived from catchment-wide analyses, is justified by the particular circumstances presented by Gosnells, namely, "at capacity" drainage networks and the acceptance of a small flow from each site (maximum permissible pre-development, 20% AEP) into the formal drainage path. It also enables standardisation of the design procedure to be incorporated into the developer spreadsheet.

Implementation on a Site

The City of Gosnells provides a Typical Residential Layout for developers to be used with the spreadsheet in preparing a development application. This is illustrated in **Figure 5.22**.

Also provided to developers are standard drawings showing construction details of „soakwells" and concrete (in-ground) tanks as required by the City for inclusion in development applications. These drawings remove any uncertainty or misunderstanding about the City's intentions and requirements for stormwater management on new developments. The details are illustrated in **Figures 5.23** and **5.24.**

The Developer Spreadsheet

The theory upon which the Spreadsheet calculations are based for specifying "soakwells" at a given site is set out in Argue and Tennakoon (2011) and Botte et al. (2015). The spreadsheet is set up for easy and speedy use by developers and their consultants, ensuring a competent outcome acceptable to the City.

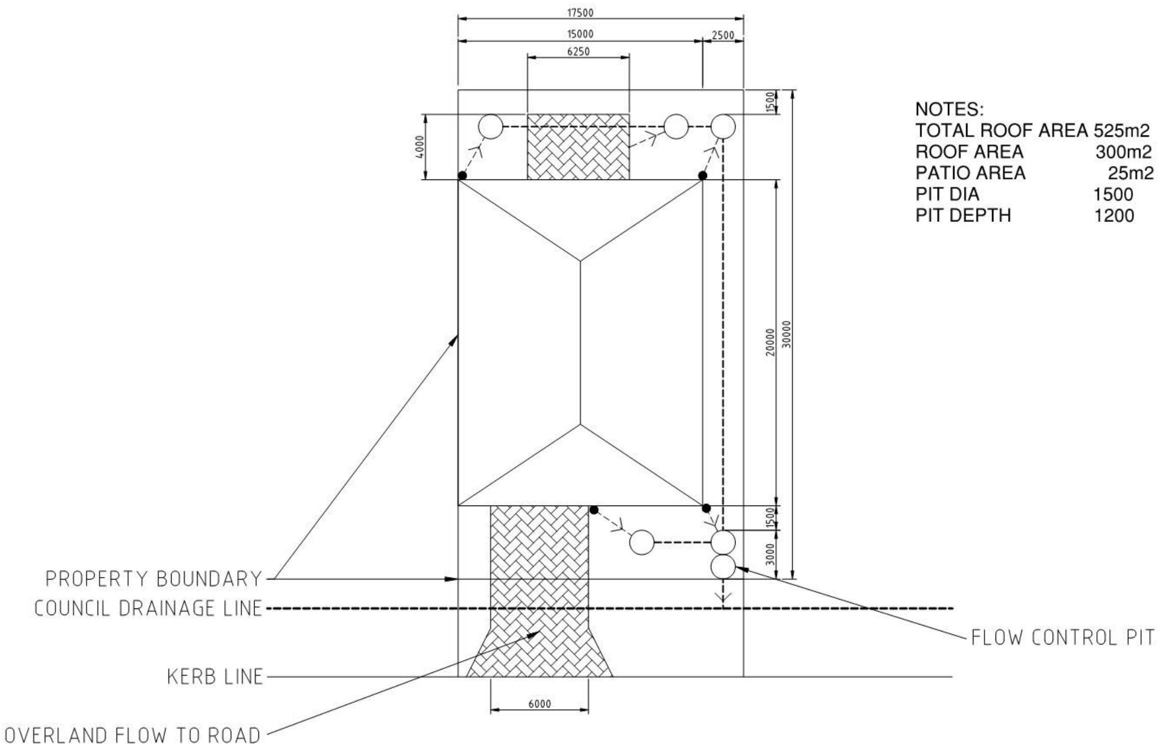


Figure 5.22: Typical Residential Layout showing Arrangement of 'Soakwells' and Other Features (Model for Development Application) (after Argue and Tennakoon (2011))

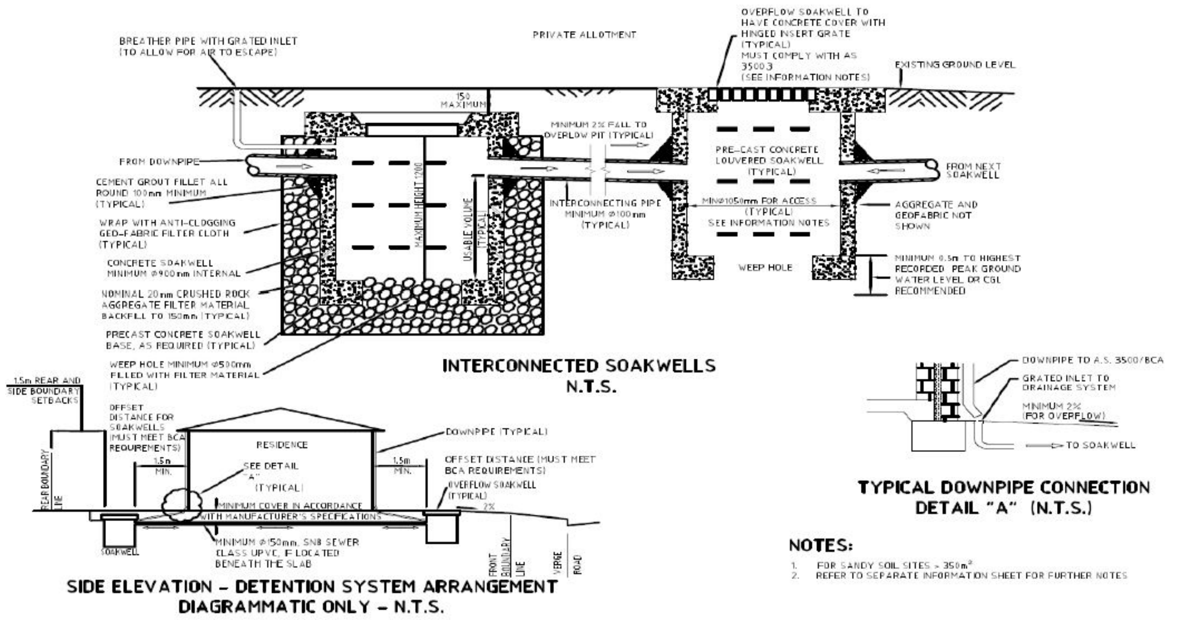


Figure 5.23 Details of Inter-Connected 'Soakwells' and House/Soakwells Arrangement: High Soil Permeability Sites (after Argue and Tennakoon (2011))

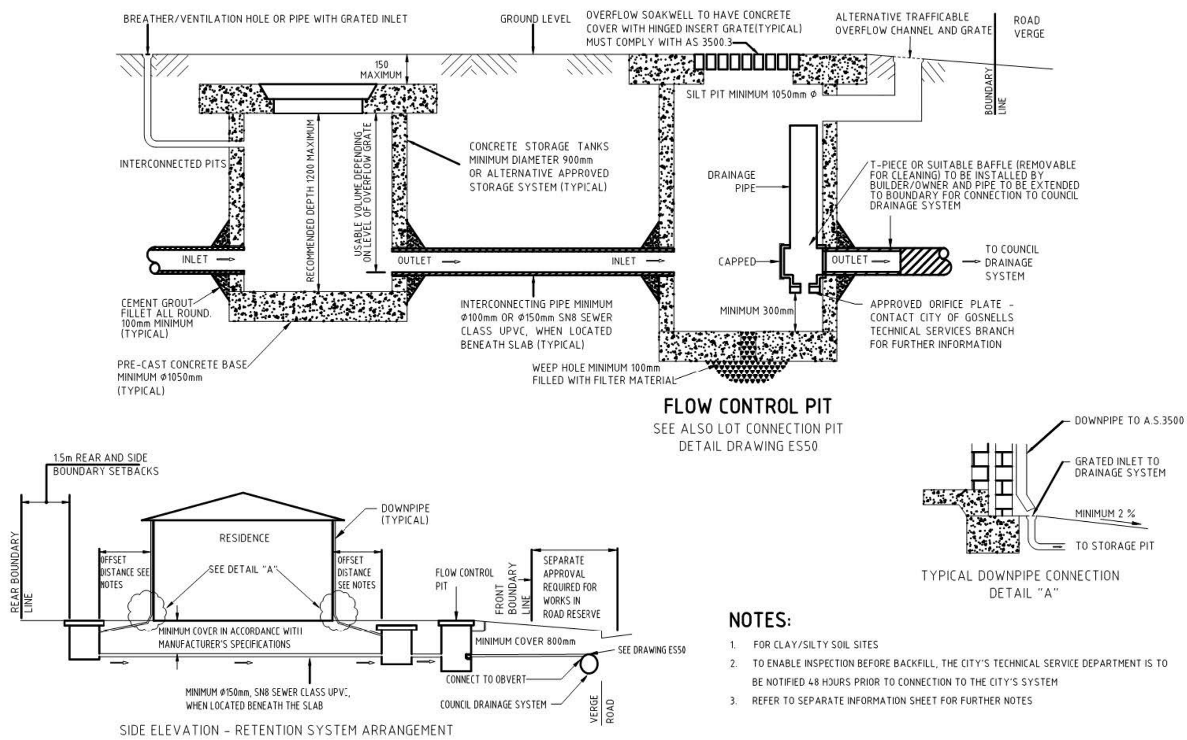


Figure 5.24 Details of Inter-Connected Circular Concrete Tanks and Flow Control Pit: Low-Permeability Sites (after Argue and Tennakoon (2011))

The intention of the spreadsheet is twofold.

- First, it provides a clear and cost-effective practice enabling the City to achieve its development and re-development goals despite the presence of a largely "at capacity" stormwater infrastructure. Intelligent use of the infiltration capabilities of the municipality's sandy soils makes this possible in a large part of the City; the concepts of retention and extended detention, intelligently applied, account for the remaining areas of less permeable soils; and,

- Second, it provides a tool for use (and submission) in the approval process, ensuring that practices acceptable to the City are followed with minimum design effort on the part of proponents of development/re-development projects, and minimum staff time required to carry out checking procedures;

It involves the following steps:

Step 1: Insert roof area and total area of paving draining to on-site „soakwells‟ and concrete tanks;

Step 2: Select size (diameter and height) of proposed „soakwells‟ or tanks from drop down menu;

Step 3: select soil type (sand or sandy clay or clay) from drop down menu.

Three default values of a Moderation Factor, U, are incorporated into the calculations for soils nominated as sand or sandy clay or clay. However, these values can be over-ridden and a value for U inserted from a regression relationship (U versus Hydraulic Conductivity) where information on field soil permeability measured at a site is known.

Step 4: Select "Yes" or "No" to the question: "Permission to connect to Council drainage?"

The intent of this step is to prevent allotments of (relatively) large size discharging storm runoff into the street drainage system. Special consideration – derived from the answer to the STEP 4 question – is given to sites where this requirement causes distress, for example, at a site slightly larger than 350 m2 located in heavy clay.

The spreadsheet produces three outcomes:

Outcome 1: Number of soakwells or concrete tanks needed on the property;
Care needs to be exercised in locating the soakwells on the site layout to ensure that sufficient clearance distance between soakwells is provided, and between soakwells and footings/boundaries.

Outcome 2: Volume required to be retained/detained within the soakwells or tanks;

Outcome 3: Diameter of orifice of the outlet pipe to council drainage (with concrete tanks – see **Figure 5.24**).

Conclusion

It is concluded that the initiative taken by the City of Gosnells in developing its cost-effective stormwater management strategy in the circumstances of a substantially “at capacity” infrastructure and faced with demand for urban growth, provides state agencies and other municipal agencies, not only on Perth but also elsewhere across the nation, a large scale example of an Infrastructure Compliant Stormwater Management strategy in action.

As previously discussed by Argue and Pezzaniti (2012), *the nature of this investment cost as well as its magnitude are, clearly, case-specific matters which raise some vital questions. First: is the overall cost that follows implementation of the source control option less than the cost involved in conventional upgrade practice? Second: are the owners who take over the re-developed properties expected to carry the full weight of this cost - passed to them by developers or should councils share this burden with (re- development) owners? And in this (latter) scenario: what proportion should be applied to each partner? The discussion/negotiation of these (economic) matters needs to take account of the particular advantage which the source control practices hold over conventional upgrades for council investment: this would be called upon progressively over time as opportunities for re-development were taken up, and not as massive injections applied at time intervals of 20 to 30 years*.

Based on the evidence at hand, the City of Gosnells is convinced that an at-source approach to stormwater management in redevelopment areas has provided the community and developers at large with real cost-savings as well as significant resource benefits.

5.10 References

Guidelines

ACT Department of Urban Services (1998) *Design Standards for Urban Infrastructure, Section 1 Stormwater,* 1.9 Retarding Basins, Section 1.9 Retarding Basins, 4 pp.

ANCOLD (2000a) “Guidelines on Selection of Acceptable Flood Capacity of Dams”, March, 36 pp + Apps.

ANCOLD (2000b) “Guidelines on Assessment of the Consequences of Dam Failure”, May, 15 pp + Aps.

American Public Works Association Research Foundation (APWA) and Institute for Water Resources of the American Public Works Association (1981). *Urban stormwater management*. A. P. Works. Chicago, American Public Works Association: 34-199.

Argue, J R (Ed, 2004/2013) “*WSUD: basic procedures for ‘source control of stormwater – a Handbook for Australian practice*”. Urban Water Resources Centre, University of South Australia, 7th Printing, Adelaide, 245 pp + Apps.

Australian and New Zealand Environment and Conservation Council (2000) “Australian Guidelines for Urban Stormwater Management”, *National Water Quality Management Strategy Report No. 10*, Canberra, 72 pp.

BASIX Sustainability Unit. (2009). "*Building Sustainability Index*", from www.basix.nsw.gov.au

Department of Irrigation and Drainage (2000) “*Urban Stormwater Management Manual for Malaysia (Maunual Saliran Mesra Alam Malaysia)*” Vol 7 Detention (Chapter 18, 19, 20), Volume 8 Retention (Chapters 21, 22), Kuala Lumpur.

Department of Irrigation and Drainage (2012a) "*Urban Stormwater Management Manual for Malaysia (Maunual Saliran Mesra Alam Malaysia)*" Edition 2, Chapter 5 On-Site Detention, 33 pp.

Department of Irrigation and Drainage (2012b) "*Urban Stormwater Management Manual for Malaysia (Maunual Saliran Mesra Alam Malaysia)*" Edition 2, Chapter 6 Rainwater Harvesting, 18 pp.

Department of Water, Western Australia (2007) "*Stormwater management manual for Western Australia*". Department of Water, WA, Perth, August.

Derwent Estuary Program (2012) "*Water Sensitive Urban Design: Engineering Procedures for Stormwater Management in Tasmania*", Hobart

Engineers Australia (2006). *Australian Runoff Quality – a guide to water sensitive urban design*, Wong, T. H.F. (Editor-in-Chief), Barton.

Government of South Australia (2010) "*Water Sensitive Urban Design Technical Manual: Greater Adelaide Region*", Department of Planning and Local Government, South Australia

Hobart City Council (2006) "*Water Sensitive Urban Design Site Development Guidelines and Practice Notes*", Hobart City Council, Hobart

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

Melbourne Water (2010) "Guidelines for the Design and Assessment of Flood Retarding Basins", 9 pp.

New South Wales Government (2004a). State Environmental Planning Policy (Building Sustainability Index: BASIX), Sydney.

New South Wales Government (2004b). "Managing Urban Stormwater: Soils and Construction", Volume 1, 4th Edition, March,

Planning SA (2003) "*Minister's Specification SA 78AA – On-site Retention of* Stormwater", Building Policy Branch, Department for Transport and Urban Planning, South Australia

Queensland Department of Energy and Water Supply (2013) "*Queensland Urban Drainage Manual*", Provisional Third Edition, Chapter 5.00 Detention/retention systems, 17 pp.

Queensland Department of Natural Resources and Mines (2007) Queensland Urban Drainage Manual, Volume 1, Second Edition, Book2, Chapter 5.00 Detention/retention systems, 25pp

Stormwater Committee Victoria (1999). *Urban stormwater - Best practice environmental management guidelines*. Collingwood, CSIRO.

Upper Parramatta River Catchment Trust (1991). "*On-Site Detention Handbook*. First Edition.

Upper Parramatta River Catchment Trust (1994). *On-site Stormwater Detention Handbook*. Second Edition.

Upper Parramatta River Catchment Trust (1999). *On-site Stormwater Detention Handbook*. Third Edition.

UPRCT (2005) "*On-Site Stormwater Detention Handbook*", Fourth Edition, December.

US Army Corps of Engineers (1984) *Shore Protection Manual,* 4th Edition.

US Bureau of Reclamation (1984), Hydraulic *Design of Stilling Basins and Energy Dissipators*, Engineering Monograph No. 25, Eighth Printing, May, US Department of the Interior, Washington, DC.

US Bureau of Reclamation (1987), *Design of Small Dams*, US Department of the Interior, Washington, DC.

Council Guidelines

Alexandrina Council (2008) *Stormwater Retention and Detention Options for Residential, Commercial and Industrial Development*. Alexandrina, SA

Bankstown City Council (2006). *Development engineering standards*. Bankstown, NSW

Banyule City Council (2012). *Stormwater Drainage Requirements for New Developments - On-Site Stormwater Detention and Outfall Drainage Requirements*. Banyule, VIC

Baulkham Hills Shire Council (2004). *Development Control Plan no.5: dual occupancy*. Castle Hill, NSW

Blacktown City Council (2005). *Engineering guide for development 2005*. Blacktown, NSW

Blacktown City Council (2006). *Blacktown development control plan 2006*. Blacktown, NSW

Blue Mountains City Council (2005). *Better Living Development Control Plan*. Katoomba, NSW

Canterbury City Council (2014). *Canterbury Development Control Plan 2012 - Section 6.4: Development engineering flood and stormwater*. Canterbury City, NSW

City of Busselton (2014). *Water Sensitive Urban Design Guidelines for Individual Lots, Infill Development and Subdivision*, Busselton, WA

City of Casey (2013). *On-Site Stormwater Detention Policy*, City of Casey, VIC

City of Kingston (2013). *Civil Design Requirements for Developers - Part A: Integrated Stormwater Managemen*t. Kingston City, VIC

Hawkesbury City Council (2000). *Development control plan 2000*. Windsor, NSW

Hills Shire Council (2010). *Design guidelines subdivision/developments*, Castle Hill, NSW

Hobart City Council (2006), *Water Sensitive Urban Design Site Development Guidelines and Practice Notes*, Hobart, TAS

Holroyd City Council (2015) *On-Site Stormwater Detention Policy*. Holroyd, NSW

Kogarah City Council (2006). *Water management policy*, Kogarah, NSW

Ku-ring-gai Council (2005). *Water management development control plan - DCP 47*, Gordon, NSW

Manly Council (2003) *Specification for On-site Stormwater Management 2003*. Manly, NSW

Manningham City Council (2003) *On-Site Stormwater Detention Guide*, Manningham, VIC

Marrickville Council (1999) *Marrickville Council Stormwater and On-Site Detention Code*. Marrickville, NSW

Melton Shire Council (2009) *Guidelines for On-Site Stormwater Detention*, Melton, VIC

Parramatta City Council (2005). *Parramatta Development Control Plan*. Parramatta, NSW

Penrith City Council (2008). *Exhibition of Stage 1 Planning Documents*. Penrith, NSW

Penrith City Council (2010). *Penrith City Council Development Control Plan*. Penrith.

Pine Rivers Shire Council (2006) *Pine Rivers Shire Council Design Manual: Design Standards - Part 2: Drainage*. Now merged into the Moreton Bay Regional Council, Caboolture, QLD

Singh, G., Ghetti, I. and Chanan, A. (2007) "Developing sustainable water management policy for Kogarah". *Proceedings*, 5th Intl Water Sensitive Urban Design Conference, 21-23 August, Sydney

Tea Tree Gully Council (2013). *Development Plan*. Department of Planning, Transport and Infrastructure, SA

Detention

Beardshaw, C. and Law, S. (2012) "When to Slow and When to Flow – A Case Study of the Interaction Between Retardation and Water Quality, *Proceedings*, Stormwater 2012, National Conference of the Stormwater Industry Association, 15-19 October, Melbourne.

Beecham, S., Hourigan, P., Wells, J. and Brisbin, S. (2004). "Estimating the treatment performance and OSD characteristics of both proprietary and non-proprietary WSUD systems at Castle Hill in Sydney". *Proceedings*, 3rd International Conference on Water Sensitive Urban Design, 21-24 November, Adelaide.

Beecham, S., Kandasamy, J. Khiadani, M. and Trinh, D. (2005) "Modelling on-site detention on a catchment-wide basis." *Urban Water Journal*, Vol 2, pp 23-32.

Bill Guy & Partners (2004) "William Slim Drive Retardation Basin", *Draft Feasibility Study Report*, prepared for Procurement Solutions, 55 pp + Apps.

Cardno Young (2005) "William Slim Drive Retardation Basin", *Discussion Paper*, prepared for Procurement Solutions, March, 8 pp.

Cardno Young (2006) "William Slim Drive PMF Basin, Hydrology & Hydraulics", *PSP Report*, prepared for Procurement Solutions, October, 9 pp + Apps.

Carse, J., Lees, S., Phillips, B.C. and Goyen, A.G. (2006) "Improving the Management of Urban Runoff using On-Site Detention", *Proceedings*, 46th Annual Floodplain Management Authorities Conference, 1-3 March, Lismore

Clark, T.C., Phillips, B.C. and Bresnan, S (1997) "Catchment Based On-Site Stormwater Detention Policy and Procedures for Canberra Central", *Proceedings*, 24th Hydrology and Water Resources Symposium, IEAust, 24-28 November, Auckland

Institution of Engineers, Australia (1985) Guidelines for the Design of Detention Basins & Grassed Waterways for Urban Drainage Systems, April , 11 pp + App.

Lee, J.S. and M. H. Li (2008) "The impact of detention basin design on residential property value: Case studies using GIS in the hedonic price modelling." *Landscape and Urban Planning* **89**, 7-16 DOI: 10.1016/j.landurbplan.2008.09.002.

Nicholas, D.I. (1995). "On-site Stormwater Detention: Improved Implementation Techniques for Runoff Quantity and Quality Management in Sydney." *Water Science and Technology* **32**(1): 85-91.

Nicholas, D.I. and Cooper, G. (1984). *On-Site Detention Experience in Ku-Ring-Gai*. Seminar on Problems of existing stormwater drainage systems, Water Research Foundation of Australia.

O'Loughlin, G., Beecham, S., Lees, S., Rose, L. and Nicholas, D. (1995) "On site stormwater detention systems in Sydney." *Water Science and Technolog*y **32**, 169-175.

Phillips, B.C. (1987a). "Embankment Protection Measures against Overtopping", ANCOLD Bulletin, *Issue No 78*, December, pp 26-35.

Phillips, B.C. (1987b). "Urban Application of Large Dam Technology", ANCOLD Bulletin, *Issue No. 78*, December pp 36-44.

Phillips, B.C., Blacka, M., Bresnan, S. and Neal, J. (2007) "Hydraulic Assessments of the William Slim Drive PMF Retarding Basin", *Proceedings*, 47th Floodplain Management Authorities Conference, 27 February – 2 March 2007, Gunnedah.

Phillips, B.C., Bresnan, S. and Blacka, M. (2008) "Hydraulic Design of the William Slim Drive PMF Retarding Basin", *Proceedings*, 9th National Conference on Hydraulics in Water Engineering, 23-26 September, Darwin.

Phillips, B.C., Hranisavljevic, D. and Corin, J. (1990) "Hydraulic Lessons - Retarding Basin 3 - Toongabbie Creek", *Proceedings*, Hydraulics in Civil Engineering Conference, 1990, IEAust, Sydney, 3-5 July, pp. 57-61.

Phillips, B.C., Lees, S., Lucas, E. and Carse, J. (2000) "Upper Toongabbie Creek Flood Mitigation Strategy – Stage 1", *Proceedings*, 40th Floodplain Management Conference, NSW Floodplain Management Authorities, 9-12 May, Parramatta.

Silveri, P. and Rigby, T. (2006). "Experiences in Developing an Upgraded OSD Policy for the City of Wollongong". *Proceedings*, 30th Hydrology and Water Resources Symposium, 4-7 December, Launceston.

Smith, K. (1994). "On-site detention: "Is it the most effective way to resolve flooding problems?". *Proceedings*, 2nd Annual Conference on Soil and Water Management for Urban Development - creative stormwater management, Sydney.

Thomson, R.S., Maratea, E., Phillips, B.C., Kilaparty, B., Perera, J. and Paton, D. (2013) "Assessing the Performance of OSD in Highly Urbanised Areas, *Proceedings*, Stormwater NSW Conference, 17-19 September 2013, Blue Mountains.

UNSW Water Research Laboratory (1988) "Toongabbie Creek – Basin 3 Outlet Structure Model Study", *Final Report*, prepared for Blacktown City Council.

UNSW Water Research Laboratory (2006) "Physical Modelling of William Slim Drive Retarding Basin Outlet, Canberra, ACT", *Technical Report 2006/17*, prepared for Cardno Young, 20 pp.

Upper Parramatta River Catchment Trust (1990) "On-Site Detention of Stormwater - Development of an Effective Common Policy", *Discussion Paper*, Prepared by the Development Controls Steering Committee, November.

Urbanos, B. and P. Stahre (1993). *Stormwater - Best Management Practices and Detention for Water Quality, Drainage and CSO Management*. New Jersey, Prentice Hall.

Urbonas, B. and M. W. Glidden (1983). *Potential Effectiveness of Detention Facilities*. Southwest Storm Drainage Symposium, Texas.

Willing and Partners (1993). "Feasibility Assessment of Tradeable On-site Stormwater Detention Permits", *Final Report*, prepared for the Upper Parramatta River Catchment Trust, October, 10 pp.

Retention

Andoh, R.Y.G. and Declerck, C. (1999) "Source control and distributed storage - a cost effective approach to urban drainage for the new millennium?" *Proceedings,* 8th International Conference on Urban Storm Drainage, Sydney, Engineers Australia.

Argue, J.R. (1997) *On-Site Retention and Use (OSRU): The new kid on the stormwater block.* Stormwater and Soil Erosion, Future Directions for Australian Soil and Water Management, Brisbane.

Argue, J.R. (1998). "Demonstration Projects in "Source Control" Technology: Theory and Practice". *Proceedings*, HydraStorm'98 Conference, IEAust, 27-30 September, Adelaide.

Argue, J.R. (2001) "Recent developments in Australian 'source control' technology and practice". *Proceedings*, 1st National Conference on SuDS, Coventry, UK.

Argue, J.R. (2002) "OSR (Underground Devices) in Soils of High/Medium and Low Permeability". *Workshop Proceedings*, Stormwater Industry Association, 30 May 2001, Concord.

Argue, J.R. (2013) "WSUD and Green Infrastructure: a cost-effective and sustainable strategy for urban re-development", *Proceedings*, 8th International WSUD Conference, 25-29 November 2013, Gold Coast.

Argue, J.R. (2014) "'Source Control' Strategies in Urban Development and Re-Development: Can they also accommodate Climate Change?", *Proceedings*, 2014 Stormwater Australia National Conference, 13-17 October, Adelaide.

Argue, J.R. and Pezzaniti, D. (2007) "WSUD (Stormwater) Practices - Learning from Nature: A Discussion". *Proceedings*, Rainwater and Urban Design Conference, IEAust., Sydney.

Argue, J.R. and Pezzaniti, D. (2009) "The Need for Quantity-related Criteria in Regional WSUD Guidelines: A Demonstration", *Proceedings*, 2009 Annual Conference, SIA NSW & SIA VIC, 7 – 10 July, Albury.

Argue, J.R. and Pezzaniti, D. (2010) "The Quantity Domain of WSUD: Flooding and Environmental Flows", *Proceedings*, SIA National Conference, 9-12 November, Sydney.

Argue, J.R. and Pezzaniti, D. (2012) "Use of WSUD 'Source Control' Practices to Manage Floodwaters in Urbanising Landscapes: Developed and Ultra-Developed Catchments", *Proceedings*, Stormwater 2012, National Conference of the Stormwater Industry Association, 15-19 October, Melbourne.

Argue, J.R. and Scott, P. (2000) "On-Site Stormwater Retention (OSR) in residential Catchments: A Better Option?", *Proceedings*, 40th Floodplain Management Conference, NSW Floodplain Management Authorities, 9-12 May, Parramatta

Argue, J.R. and Tennakoon, A. (2011) Managing Urban Regrowth with an 'at capacity' Stormwater Infrastructure, *Proceedings*, 34th International Association of Hydro-Environment Engineering and Research World Congress: 33rd Hydrology and Water Resources Symposium: 10th Conference on Hydraulics in Water Engineering, 26 June - 1 July 2011, Brisbane.

Argue, J.R., Hewa, G and Pezzaniti, D. (2010). The Quantity Domain of WSUD: Flooding and Environmental Flows. *Proceedings*, 14th Annual Conference on Urban Stormwater: Stormwater 2010 - Creating a Storm: the future of Stormwater Management in Australia, Sydney, NSW and Victoria Stormwater Industry Association.

Bishop, W., Law, S., Newton, J. and Godfrey, M. (2013) “Integrated Water Management Opportunities for Inner Suburban Areas”. *Proceedings*, 8th Intl Water Sensitive Urban Design Conference, 25-29 November, Gold Coast.

Blacktown City Council ( ) “*Angus Creek Stormwater Harvesting Project*”, Brochure, Blacktown.

Botte, M., Kannangara, D.I. and Tennakoon, A. (2015) “On-Site Stormwater Management In Urban Infill Areas - The City of Gosnells' Drainage Challenge”, *Proceedings*, 2015 IPWEA Conference, 5-8 June 2015, Rotorua, NZ.

Breen, L., Dyer, F., Hairsine, P., Riddiford, J., Sirriwardhena, V. and Zierholz, C. (1997). *Controlling sediment and nutrient movement within catchments*. CRC for Catchment Hydrology, Industry Report 97/9, Melbourne.

Cardno Willing (2005) “Cronulla Business Park Stormwater Treatment Train Management Plan”, *Final Report*, prepared for Australand Holdings Ltd, May, 16 pp + Apps.

Coombes, P. and Kuczera, G. (2003) "Analysis of the Performance of Rainwater Tanks in Australian Capital Cities", *Proceedings*, 28th International Hydrology and Water Resources Symposium, Engineers Australia, Vol 2, Wollongong.

Coombes, P.J. and Barry, M.E. (2008). “Determination of Available Storage in Rainwater Tanks Prior to Storm Events”. *Proceedings*, 31st Hydrology and Water Resources Symposium, 15-17 April, Adelaide.

Coombes, P.J. and Kuczera, G. (2001) “Rainwater Tank Design for Water Supply and Stormwater Management”, *Proceedings*, SIA Regional Conference, April, Port Stephens.

Coombes, P.J., Frost, A. and Kuczera, G. (2001). *Impact of rainwater tank and On Site Detention options on stormwater management in the Upper Parramatta River Catchment*. Newcastle, Department of Civil, Surveying and Environmental Engineering, University of Newcastle,

Coombes, P.J., Kuczera, G and Kalma, J.D. (2000). “Economic benefits arising from use of Water Sensitive Urban Development source control measures”, *Proceedings*, 3rd International Hydrology and Water Resources Symposium Proceedings, IEAust., 20-23 November, Perth.

Coombes, P.J., Kuczera, G. Kalma, J.D. and Argue, J.R. (2002) "*An evaluation of benefits of source control measures at the regional scale.*" Urban Water **4**, 307-320.

Coombes, P.J.; Frost, A.; Kuczera, G.; O'Loughlin, G. and Lees, S. (2002). “Rainwater Tank Options for Stormwater Management in the Upper Parramatta River Catchment”, *Proceedings*, 27th Hydrology and Water Resources Symposium, 20-23 May, Melbourne pp. 474-482.

Cunliffe, D. A. (1998). *Guidance on the use of Rainwater Tanks*. National Environmental Health Forum Monographs, Water Series Number 3. Rundle Mall, Department of Human Services.

Department of Health (2003) *Urban Rainwater Collection*, Department of Health, Perth, Western Australia. Available via <http://www.health.wa.gov.au/envirohealth> or <http://www.health.wa.gov.au>, go to Services A-Z/E/Environmental Health.

Department of Land and Water Conservation (1998) “*The Constructed Wetlands Manual*”, 2 Vols, ISBN 0 7313 1329 6, 444 pp.

Environmental Protection Authority (2005) “Strategic Advice on Managed Aquifer Recharge using Treated Wastewater on the Swan Coastal Plain”, *Bulletin Report No. 1199*, Environmental Protection Authority, Perth, Western Australia. Available via <http://www.epa.gov.wa.au>.

Evans, C., Coombes, P.J. Dunstan, H., Harrison, T., Martin, A. and Morrow, A. (2008). "*Rainwater Tanks and Mircobial Quality: Are the Indications Clear?*" Australian Journal of Water Resources **12**(2): 143-152.

Evans, C.A., Coombes, P.J. and Dunstan, R.H. (2006) "*Wind, Rain and Bacteria: The Effect of Weather on the Microbial Composition of Roof-Harvested Rainwater.*" Water Research **40**, 37-44 DOI: 10.1016/j.watres.2005.10.034.

Gardiner, A. and Hardy, M. (2005) "*Beyond Demonstration Mode: The Application of WSUD in Australia.*" Australian Planner **42**, 16-21.

Herrmann, T. and U. Schmida (1999) "*Rainwater Utilisation in Germany: Efficiency, Dimensioning, Hydraulic and Environmental Aspects.*" Urban Water **1**, 307-316 DOI: 10.1016/S1462- 0758(00)00024-8.

Hewa, G., Argue, J.R. and Pezzanti, D. (2009) "Setting Criteria for Environmental and Channel Forming Flows for Waterways in Urbanising Catchments". *Proceedings*, 6th International Conference on Water Sensitive Urban Design, 5 – 8 May, Perth.

Joliffe, I.B. (1997). *Roof Tanks - A Role in Stormwater Management*. Stormwater and Soil Erosion, Future Directions for Australian Soil and Water Management, Brisbane.

Joliffe, I.B. and Fryar, R. (2000). Roofwater Tanks-Hydrologic and Other Benefits.", *Proceedings*, 3rd International Hydrology and Water Resources Symposium, IEAust, 20-23 November, Perth.

Kannangara D.I., Botte M., Sarukkalige P.R. (2012) "An Innovative Approach To Stormwater Management Accounting For Spatial Variability In Soil Permeability", *Proceedings*, 7th International Conference on Water Sensitive Urban Design, 21-23 February, Melbourne.

Khastagir, A. and L. N. N. Jayasuriya (2010) "*Impacts of using rainwater tanks on stormwater harvesting and runoff quality.*" Water Science and Technology **62.2**, 324-329 DOI: 10.2166/wst.2010.283.

Kuczera, G. (2008). "Urban Water Supply Drought Security: A Comparative Analysis of Complimentary Centralised and Decentralised Storage Systems.", *Proceedings*, 31st Hydrology and Water Resources Symposium, Adelaide.

Lee, A., Hewa, G., Pezzaniti, D. and Argue, J.R. (2008) "Improving Stream Low Flow Regimes in Urbanised Catchments using Water Sensitive Urban Design Techniques". *Australian Journal of Water Resources*, Vol 12, No 2, Engineers Australia, pp 121-132.

Lucas, S.A. and Coombes, P.J. (2009) "Mains Water Savings and Stormwater Management Benefits from Large Architecturally-Designed Under-Floor Rainwater Storages". In: *SSEE 2009: Solutions for a Sustainable Planet*. Barton, ACT.: Engineers Australia, 2009: pp. 386-397.

Phillips, B.C., Daniels, C. and McKenzie, G. (1999) "An Infiltration-Based Stormwater Management Strategy for the Cronulla Integrated Resort, NSW", *Proceedings*, Water99 Joint Congress, IEAust, 6-8 July, Brisbane, pp. 932-937.

Vaes, G. and J. Berlamont (2001) "*The Effect of Rainwater Storage Tanks on Design Storms.*" Urban Water **3**, 303-307 DOI: 10.1016/S1462-0758(01)00044-9.

van der Sterren, M., Rahman, A. and Dennis, G.R. (2013). "Quality and quantity monitoring of five rainwater tanks in Western Sydney, Australia." *Journal of Environmental Engineering*, **139**(3): New York, pp. 332-340.

van der Sterren, M., Rahman, A. and Ryan (2014), G. "Modelling of a lot scale rainwater tank system in XP-SWMM: A case study in Western Sydney, Australia" *Journal of Environmental Management*, **141**: pp.177-189.

Integrated Detention and Retention

Argue, J.R., Boubli, D. and Coombes, P.J. (2003) "Innovative Water Cycle Design/Management at a Difficult Site: "Heritage Mews" in Western Sydney, Australia", *Proceedings*, Second National Conference on Sustainable Drainage, 23-24 June 2003, Coventry, UK.

Butler, D. and J. Parkingson (1997) "*Towards sustainable drainage.*" Water Science and Technology **35**, pp. 53-63.

Cardno Willing (2002) "Study on the Combined Effects of OSD and Rainwater Tanks on the Upper Parramatta River Catchment at Varying Sub-Catchment Scales", Final Report, prepared for the UPRCT, October.

Cardno Willing (2004) "Study on the Combined Effects of OSD and Rainwater Tanks on the Upper Parramatta River Catchment at Varying Sub-Catchment Scales, Additional Assessments". *Final Report*, prepared for the UPRCT, February.

Chanan, A. and Woods, P. (2006) "*Introducing Total Water Cycle Management in Sydney: a Kogarah council initiative.*" Desalination **187**, pp. 11-16.

Coombes P.J., Boubli, D. and Argue, J.R. (2003). "Integrated water cycle management at the Heritage Mews development in Western Sydney", *Proceedings*, 28th International Hydrology and Water Resources Symposium. 10-13 November, Wollongong.

Coombes, P.J. (2009) "The Use of Rainwater Tanks as a Supplement or Replacement for Onsite Stormwater Detention (OSD) in the Knox Area of Victoria", *Proceedings*, 32nd Hydrology and Water Resources Symposium, 30 November to 3 December, Newcastle, pp. 616-627.

Finnemore, E. J. and W. G. Lynard (1982). "*Management and control technology for urban stormwater pollution.*" Journal (Water Pollution Control Federation) **54**(7): pp. 1099-1111.

Goyen, A.G., Lees, S.J. and Phillips, B.C. (2002) "Analysis of Allotment Based Storage, Infiltration and Reuse Drainage Strategies to Minimize Urbanisation Effects", *Proceedings*, 9th International Conference on Urban Drainage, IAHR/IWA, 8-13 September, Portland.

Kannangara, D., Botte, M. and Thennakoon, A. (2013) "An Efficient and Sustainable Stormwater Management Approach to Infill Development Where Existing Drainage Runs 'At Capacity". *Proceedings*, 8th Intl Water Sensitive Urban Design Conference, 25-29 November, Gold Coast

Pezzaniti, D., Argue, J.R. and Johnston, L. (2002). "Detention/Retention Storages for Peak Flow Reduction in Urban Catchments: Effects of Spatial Deployment of Storages", *Australian Journal of Water Resources,* Vol 7, No. 2, pp. 131-137.

Phillips, B.C. and Yu, S. (2015) "How robust are OSD and OSR Systems?", *Proceedings*, 3rd International Erosion Control Conference and 9th International Water Sensitive Urban Design Conference, 20-22 October 2015, Darling Harbour

Phillips, B.C., Goyen, A.G. and Lees, S.J. (2005) “Improving the Sustainability of On-Site Detention in Urban Catchments”, *Proceedings*, 10th International Conference on Urban Drainage, IAHR/IWA, 21-26 August, Copenhagen.

Phillips, B.C., Smith, J. and Tickle, R. (2000) “Integrating Stormwater Quantity and Quality Control – The McGovern Basin and Wetland, Dubbo”, *Proceedings*, IPWEA Inaugural Conference, 30th April-3 May 2000, Penrith

Ruddell, N. (2005) “Conversion of a Dry Detention Basin to a Wet Basin to Achieve Water Quality Outcomes and Stormwater Harvesting in Bonnyrigg”, *Proceedings*, SIA NSW Conference, 20-21 April, Port Macquarie.

Scott, P., Santos, R. and Argue, J.A. (1998) “OSD versus OSR: Performance, environmental and cost comparisons in a residential catchment”, *Proceedings*, NOVATECH’98, 3rd international conference on innovative technologies in urban storm drainage May 4 - 6, 1998 Lyon, France.

Tennakoon, A. and Argue, J.R. (2011). “Managing Urban Regrowth with an 'at Capacity' Stormwater Infrastructure”. Proceedings, 34th IAHR Congress, 26 June -1 July, Brisbane.

Thompson, G.R., Goyen, A.G. and Phillips, B.C. (2004) “Evaluating the Interaction of On-Site Detention Tanks and Roof Water Tanks in Low Impact Developments at Varying Catchment Scales”, *Proceedings*, STORMCON 2004, 26-29 July 2004, Palm Desert, USA

van der Sterren, M. (2012). *Assessment of the impact of rainwater tanks and on-site detention on urban run-off quantity and quality characteristics*. PhD, University of Western Sydney.

van der Sterren, M. (2014) “Integrated Stormwater Management Design on a Lot Scale: The Rainwater Tank and Onsite Detention Debate”, *Proceedings*, Stormwater NSW Conference, 17-19 September 2013, Blue Mountains.

van der Sterren, M., Rahman, A. and Dennis, G.R. (2012) "Implications to Stormwater Management as a Result of Lot Scale Rainwater Tank Systems: A Case Study in Western Sydney, Australia." *Water Science & Technology*, **65** (8), pp 1475–1482.

van der Sterren, M.; Rahman, A.; Barker, G.; Ryan, G. and Shrestha, S. (2007) “Rainwater Tanks for On-Site Detention in Urban Developments in Western Sydney: An Overview”. *Proceedings*, 5th Intl Water Sensitive Urban Design Conference, 21-23 August 2007, Sydney.

Hydraulics

ASCE (1985) “*Stormwater Detention Outlet Control Structures*”, American Society of Civil Engineers, Reston VA.

Borghei, S.M., Jalili, M.R., and Ghodsian, M., (1999), “Discharge Coefficient for Sharp-Crested Side Weir in Subcritical Flow”, *Journal of Hydraulic Engineerin*g, 125 (10), pp.1051-1056.

Bos, M.G., (1978), “Discharge Measurement Structures”, 2th Edition, International Institute for Land Reclamation and Improvement/ILRI, Wageningen.

Brater, E.F., King, H.W., and Lindell, J.E., (1996), “Handbook of Hydraulics for the Solution of Hydraulic Engineering Problems”, Wei, C.E., Co-editor, McGraw-Hill Book Co. New York.

Bryant, D.B., Khan, A.A., and Aziz, N.M., (2008), “Investigation of Flow Upstream of Orifices”, *Journal of Hydraulic Engineering*, *ASCE*, pp.98-104.

Chanson, H., and Montes, J.S. (1998). "Overflow Characteristics of Circular Weirs: Effect of Inflow Conditions." *Journal of Irrigation and Drainage Engineering,* pp. 152-162.

French, R.H., (1986), "*Open-channel Hydraulics*", McGraw Hill Book Co., New York, pp.349-350.

FHWA, (2001), "*Urban Drainage Design Manual*", Hydraulic Engineering Circular No.22, 2nd Edition, Federal Highway Administration, Washington, DC.

Göğüs, M., Define, Z., and Özkandemir, V., (2006), "Broad-Crested Weirs with Rectangular Compound Cross Sections*", Journal of Irrigation and Drainage Engineering, ASCE*, 132(3), pp.272-280.

Govinda Rao, N.S., and Muralidhar, D., (1963), "Discharge Characteristics of Weirs of Finite Crest Width", La Houille Balanche, 5, pp.537-545.

Hager, W.H., (1987), "Lateral Outflow over Side Weirs", *Journal of hydraulic Engineering*, *ASCE*, 113(4), pp.491-504.

Jan, C.D., Chang, C.T., and Kuo, F.H., (2009), "Experiments on Discharge Equations of Compound Broad-Crested Weirs", *Journal of Irrigation and Drainage Engineering*, *ASCE,* doi:10.1061/(ASCE)IR.1943-4774.0000019.

Jan, C.D., Chang, C.T., and Lee, M.H., (2006), "Discussion of Design and Calibration of Compound Sharp-Crested Weir by J. Martinez et al.", *Journal of Hydraulic Engineering*, *ASCE*, 132(8), pp.868-871.

Johnson, M.C., (2000), "Discharge Coefficient Analysis for Flat-topped and Sharp-Crested Weirs", *Irrigation Science*, 19, pp.133-137.

Lea, F. C., (1942), "Hydraulics", 6^{th} ed., Longmans, Green, New York.

Martinez, J., Reca, J., Morillas, M.T., and López, J.G., (2005), "Design and Calibration of a Compound Sharp-Crested Weir", *Journal of Hydraulic Engineering*, *ASCE*, 131(2), pp.112-116.

Medaugh and Johnson, (1940), "Investigation of the Discharge and Coefficient of Small Circular Orifices," *Civil Eng.* (N.Y.), pp.422-424.

Ramamurthy, A.S., Tim, U.S., and Rao, M.V.J., (1987), "Flow Over Sharp-Crested Plate Weirs", *Journal of Irrigation and Drainage Engineering*, 113(2), pp.163-172.

Sargison, J.E., and Percy, A., (2009), "Hydraulics of Broad-Crested Weirs with Varying Side Slopes", *Journal of Irrigation and Drainage Engineering, ASCE*, 135(1), pp.115-118.

Shesha Prakash, M.N., and Shivapur, A.V., (2004), "Generalize Head-Discharge Equation for Flow over Sharp-Crested Inclined Inverted V-Notch Weir", *Journal of Irrigation and Drainage Engineering*, *ASCE*, 130(4), pp.325-330.

Steward, C.B., (1908), "Investigation of Flow through Large Submerged Orifices and Tubes, University of Wisconsin, Bulletin, No. 216.

Swamee, P.K., (1988), "Generalized Rectangular Weir Equations", *Journal of Hydraulic Engineering, ASCE*, 114(8), pp.945-949.

Swamee, P.K., Pathak, S.K., Mohan, M., Agrawal, S.K., and Ali, M.S., (1994), ''Subcritical flow over rectangular side weir.'' *Journal of Hydraulic Engineering. ASCE*, 120(4), 742–755.

USBR, (2001), "Water Measurement Manual", United States Department of the Interior Bureau of Reclamation, revised edition, Denver, CO

Van Haveren, and Bruce, P., (1986), "Water Resource Measurements*"*. American Water Works

Chapter 6
Pipe Network Design Method

Ben Kus

Chapter Status	
Book	9
Chapter	6
Date	27/11/2015
Content	Working draft
Graphs and Figures	Working draft
Examples	Working draft
General	Need to remove references to specific software Need to integrate with other chapters and check consistency.

6.1 Introduction

Computer Pipe Design Procedures

Various procedures have been developed to design pipe systems. In the past, without computers, these were based on simplifying assumptions, such as pipes being "full, but not under pressure".
Modern methods such as the procedure in DRAINS include more calculations and checks, and can apply unsteady flow hydraulic simulations through pipe systems. The 'medium' that is used to implement the calculations is electronic, not on paper. The amount of calculations is now so large that numerical checks are not possible, although 'sanity checks' can be made my comparing results from models with those from simplified procedures such as estimating flowrates per unit area. These will provide estimates that are different from those produced by computer models, but they should be in the same 'ballpark'.

The pipe design method applied in DRAINS is derived from the *Queensland Urban Drainage Manual*. It concentrates on pit inlets and requires that pits be arranged into 'families', with a range of pit sizes. (This has been done for DRAINS in sets of regional databases, for various states and territories).
In design calculations, DRAINS sets the pit size to the lowest value for its family, and then calculates the overflow from this pit, allowing for possible surface flows that may add to the flow as its overflow route passes through a downstream sub-catchment. A check is made to see whether the overflow's hydraulic characteristics exceed requirements such as those in Figure 1.20. If they do, the pit size is increased to the next level, and calculations proceed. The process continues until the method runs out of pit sizes, are a satisfactory flowrate is reached. Once pits and inflows are determined, pipes are sized, ensuring that requirements have been met.

6.2 Pipes

6.2.1 General

Peak flowrates and hydrographs calculated by rainfall-runoff models are entered into hydraulic models, which can calculate the flow characteristics (elevations, depths, widths and velocities) corresponding to the depths. Hydraulics is based more closely on physics than hydrology, and is more exact, requiring that the geometry of a system should be exactly defined. Key hydraulic concepts such as Continuity, Conservation of Mass, Energy, and the Bernoulli's Equation, are covered in Book 6 Chapter 2, along with Friction Equations including Darcy-Weisbach, the Manning Equation and the Colebrook-White Equation (Book 6 Chapter 2 Section 6), which all form an important part of pipe hydraulics.

Table 1.2. Hydraulic models for design and analysis of pipe networks and open channels.

MODEL TYPE	APPROPRIATE USE	DEGREE OF COMPLEXITY	MAJOR VARIATIONS	EXAMPLES
Individual link, steady flow, open channel flow peaks,	Design of simple pipe systems or systems on steep terrain. Approximate method of analysis.	Quite simple (computer optional)		Pipe design method in AR&R (1958). ILLUDAS pipe design method. Sizing of open channel sections by normal depths.
Connected link steady flow, mixture of open channel and pressure flow peaks (backwater or hydraulic grade line analysis).	Check on design. Design and analysis where flowrate peaks throughout system are predetermined.	Fairly simple (computer optional)		Pipe design methods in AR&R (1977), Argue (1986). Backwater analysis in open channels HEC-2 backwater program.
Pipe or channel network unsteady flow analysis (mixture of open channel and pressure flows).	Full analysis of complex pipe networks using hydrographs	Medium to very complex (computer essential)	Many methods, the main ones being: – Kinematic Wave routing; – Muskingum-Cunge Method; – Full solution to St Venant Equations.	WASSP Simulation program. EXTRAN block of SWMM. One and two-dimensional open channel flow models, eg CELLS quasi two-dimensional model.

As previously described in Table 1.2 in ARR87), various hydraulic models can be applied to pipe systems. These are shown later in Figure 6.5, with conditions illustrated by the hydraulic grade line (HGL) and energy grade line (EGL, also called total energy line, TEL). These grade lines are described in books on fluid mechanics and hydraulics and are a most useful tool for understanding flow phenomena.

6.2.2 The Hydraulic Grade Line (HGL) and Energy Grade Line (EGL)

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

The EGL is located a distance above the HGL equivalent to the velocity head V2/2g, V being the average velocity in the pipe and g the acceleration due to gravity. Its height represents the total energy (velocity + pressure + potential) available to the flow, expressed as a height in m, equivalent to flow energy per unit weight, in joules (or newton-metres) per newton.

Within pipe sections, grade lines slope downwards in the direction of flow. Their slope represents energy loss due to pipe friction. The two lines are parallel for steady flows. In closed conduits under pressure (with the HGL above the pipe), the grade lines generally have a different slope to the pipe. For steady, uniform open channel flows, the lines are parallel to the conduit, since friction loss equals the potential energy loss represented by the slope of the conduit.

Changes of conduit shape or direction cause turbulence and local losses, represented as sharp drops in the EGL. In stormwater drainage systems, significant energy losses are typically modelled at the center of pits. The HGL also drops at pits in most cases, but it can rise when the velocity downstream is significantly lower than that upstream, as illustrated in Figure 6.1.

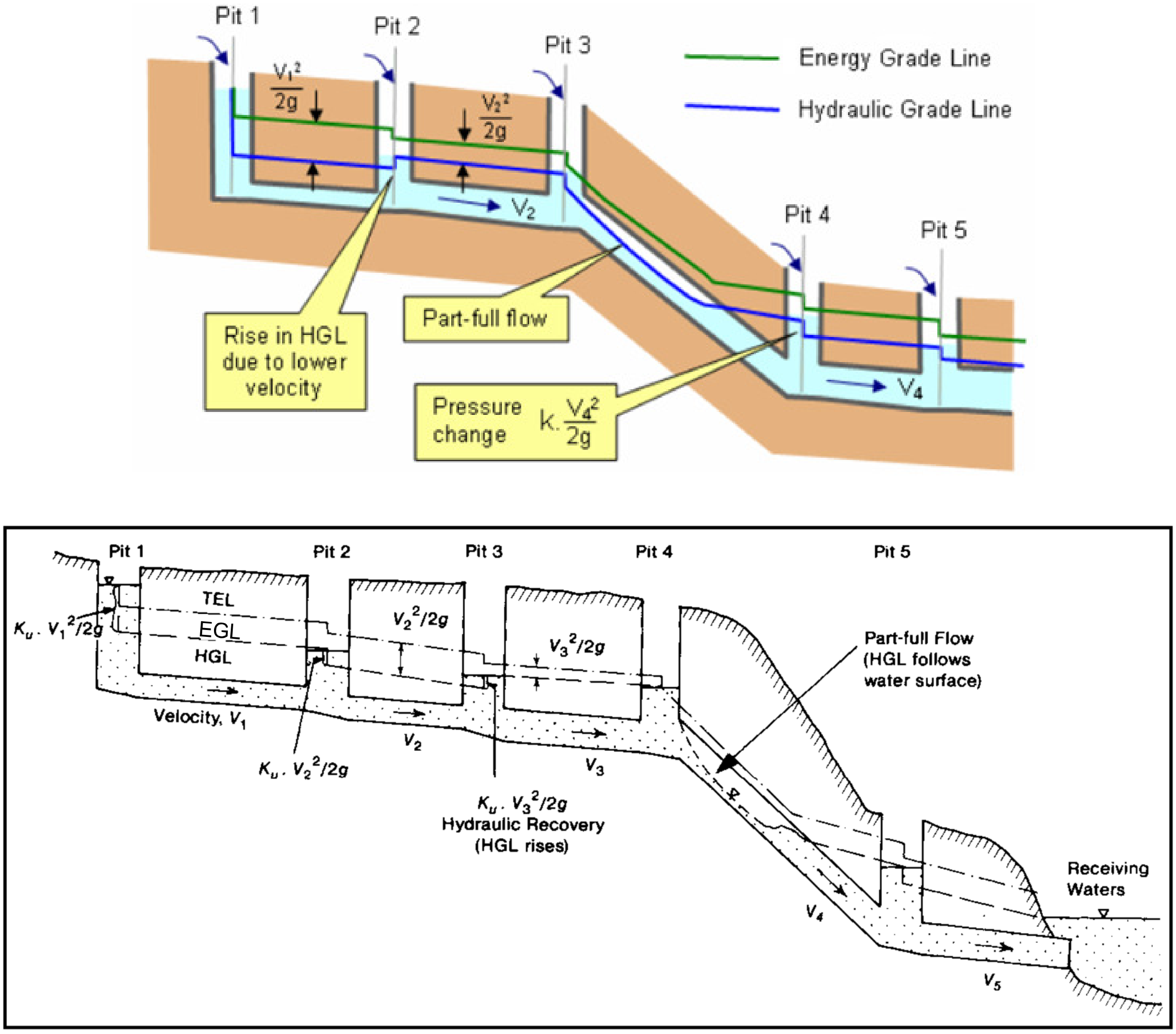


Figure 6.1 Flow behaviour in a surcharged pipe system showing Energy Grade Lines and Hydraulic Grade Lines

6.2.3 Flows through Pipe Systems

Local Losses

Changes of conduit shape or direction cause turbulence and local losses. These losses are represented as sharp drops in the EGL, and with stormwater drainage systems losses occur at locations such as entrances and exits to pits, pipe bends, and at contractions, expansions, junctions, and valves. Except for expansions and contractions, these losses are all in the form,

$$h = k.\frac{V^2}{2g} \qquad (1.10)$$

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

For a pipe entrance, the factor will depend on the geometry. A square-edged entrance will usually have a factor of k_e = 0.5 and a rounded entrance will be approximately 0.2. For a pipe exit, k_{exit} is usually 1.0, as it is assumed the entire kinetic energy of the flow will be lost as the pipe discharges into a larger body of water.

Bend losses depend on the bend radius, with a typical value being k_b = 0.5. Contractions, where the pipe diameter decreases, have low losses, with a typical k_c being 0.05. Expansions, where the diameter increases, have higher losses, dependent on the upstream and downstream velocities, with

$$h_L = \frac{K_{exp} \cdot (V_u - V_d)^2}{2g}$$

, k_{exp} being about 1.0 in abrupt pipe expansions.
Valves have variable k factors, which can become very large as valve closes.

Full-Pipe Flows

The estimation of flowrates through pipe systems flowing full is made by relating the available energy or head to the losses, all expressed in terms of the velocity head. The following calculation shows how a flowrate 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 6.2.

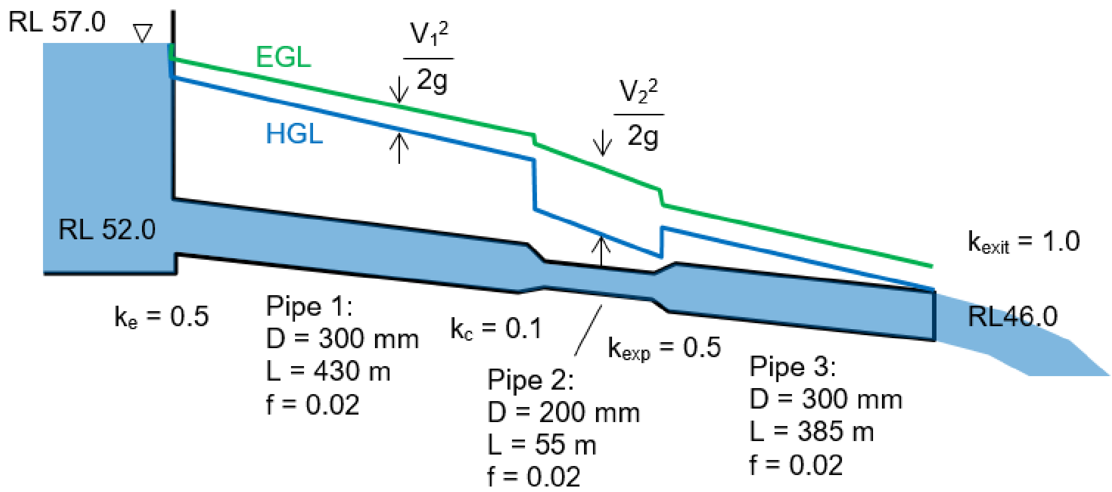


Figure 6.2 Full pipe flow

A reduction to a 200 mm pipe occurs in the middle of the pipe. For simplicity, the energy loss at the following expansion is assumed to be 0.5 times the velocity head in the downstream pipe, and all f values for the Darcy-Weisbach Equation are set at 0.02.

If the reservoir is at a level of 57.0 m, 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 $V_2 = V_1 . {}^{A_1}/_{A_2} = V_1 . ({}^{D_1}/_{D_2})^2$, the sum of the losses will be:

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

$$(0.5 + 0.02.{}^{430}/_{0.3} + 0.1 + 0.02.{}^{55}/_{0.2}.({}^{0.3}/_{0.2})^4 + 0.5 + 0.02.{}^{385}/_{0.3} + 1.0).\frac{V_1^2}{2g} = 11.0$$

$$\text{Thus, } 84.28\,.\frac{V_1^2}{2g} = 11.0, \quad V_1 = ({}^{11.0 \times 19.60}/_{84.28})^{0.5} = 1.60\text{ m/s}$$

$$\text{and } Q = V_1.A_1 = {}^{\pi}/_{4}.(0.3)^2\,.\,1.60 = 0.113\text{ m}^3\text{/s.}$$

The Manning Equation can also be used, with friction losses expressed by $(2g\,n^2\,{}^{L}/_{R^{4/3}}).\frac{V^2}{2g}$, since energy line slope $S = {}^{h_f}/_{L}$.

For systems of multiple pipes, equations can be set up that describe the state of the system, using conservation of mass, energy and momentum. When solved, these provide information on the pressures and velocities throughout a network, which can be visualised as positions of EGLs and HGLs. More complex, partial differential equations can deal with unsteady flows that change with time

Part-Full Flows

Part-full flows in stormwater pipes can be complex, as indicated in Figure 6.3, showing a classification developed by Yen (2003). ‘Surcharge’ here refers to full-pipe, flow under pressure. Although a pipe’s maximum flow capacity is actually achieved at less than full pipe flow, it is not good practice to design pipes under to this limit, as any slight disturbance will cause the free surface to transition to pressurized full pipe flow, possibly leading to surcharge.

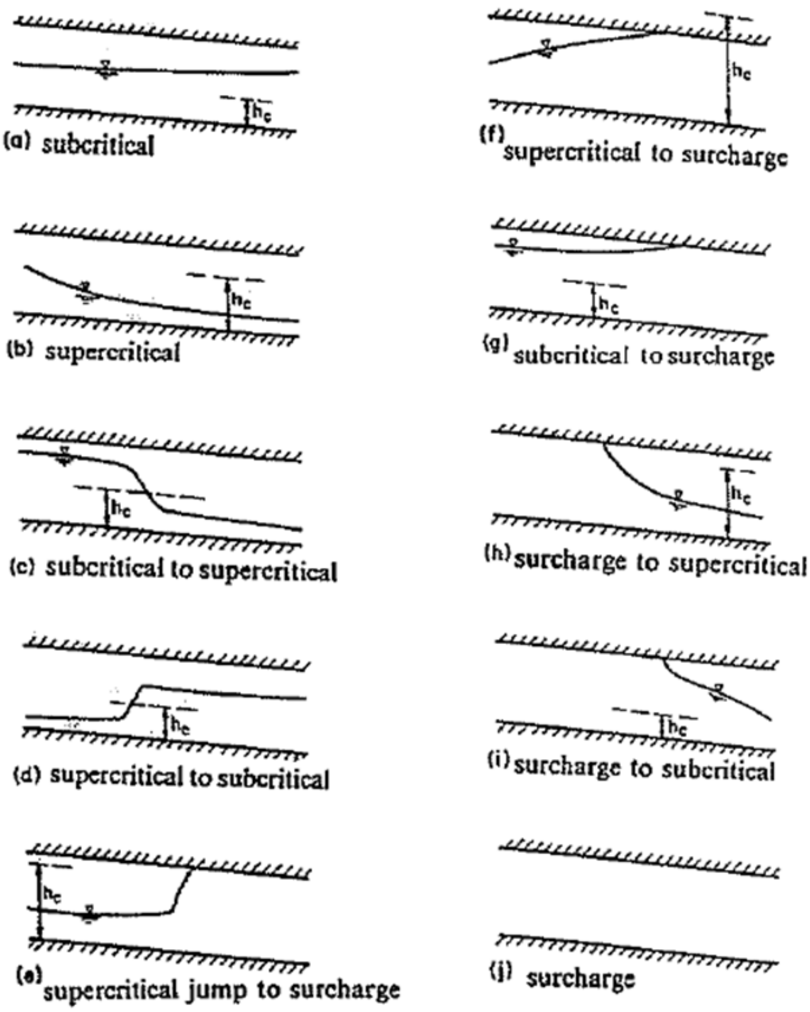


Figure 6.3 Classification of Flow in a Sewer Pipe (Yen 2003)

This assumes that flows are open channel flows, with atmospheric pressure at the surface. Further complications occur at the entrance and exit of the pipe, such as submergence at the entrance and tailwater levels affecting the outlet. In pipes flowing close to full, large air bubbles and air pockets can occur, and pressures in these can be above or below atmospheric pressure. Open channel theory is covered in Book 6.

Complex Procedures

A more complex and correct procedure is to apply partial differential equations of unsteady flow varying in space (the distance along a channel, x) and time t, defined in steps or intervals. These numerical models divide river 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, often requiring long computing times.

Although the calculation processes are quite different from water surface projection methods such as the ‘standard step’ procedure, the outputs are the same, the HGL levels at points along a conduit, at various times during a flow event. The equations allow for pipe friction and local losses, and pressure changes at pits and junctions can also be handled.

Modelling is typically carried out using computer based models, with different degrees of rigour or precision, and there is usually a trade-off between speed and accuracy. However, there are other important considerations such as stability, where iterative calculations may become unstable, giving impossibly high or low pressures, water levels, and flowrates. The usual way of achieving stable results with a computer model is to use a shorter time step, or to adjust factors affecting the relative time steps in space and time. Small errors in volumes of flow (typically < 1%) can be accepted in order to achieve fast running times.

Priessmann Slot

These methods must allow for the state of flow changing from part-full to full-pipe flow and back again. Unsteady modelling procedures employ the Priessmann Slot shown in Figure 6.5 so that they can effectively model a whole pipe system as a set of open channels. When the HGL rises above the obvert of a pipe and into a slot, the pipe is pressurised, but can still be modelled as an open channel.

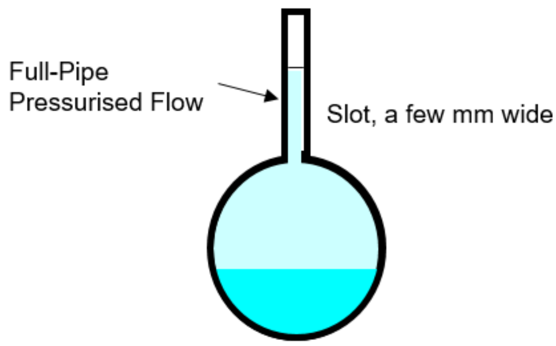


Figure 6.5 Priessmann Slot for Modelling Pipes as Open Channels

6.2.4 Hydraulic Models to Define Flow Characteristics

For the analysis of pipe networks, simple calculations based on energy must be replaced by more complex procedures. Rather than considering the flowrate that can be carried through a system, a set of inflows at entry points to a network are considered. The calculations then combine these inflows and move or route them through a system determining the water depths and velocities in the conduits.

Simpler methods or models can do this for steady flows, with unchanging flowrates, while more complex models can do this for unsteady, time-varying flowrates. HGLs and EGLs can be used to define flow depths, pressures and energies, and models must allow for overflows when water levels exceed limits, or pass over barriers.

The simple model in Figure 6.6(a) assumes steady flows occur in each pipe reach or link. These are peak flowrates derived from a hydrological model. The hydraulic grade line is assumed to run along the obvert (upper inside surface) of the pipe, so the flow condition can be described as “flowing full but not under pressure”. An allowance for local losses is sometimes made by providing a small drop (up to 90 mm, depending on change of direction) across the floor of pits. (This also serves to prevent sediment collecting in pits.)

Pipe capacities can be calculated easily, applying a friction formula such as the Manning Equation to the pipe slope. No allowance is made for surcharged conditions upstream or downstream, as the whole network is assumed to behave as a system of open channels.

The second model Figure 6.6(b) also assumes steady, peak flows occur, but as pressure flows, with the HGL located above or along the pipe obvert specific allowance is made for energy losses and pressure changes at pits, which are greater in this case than for open channel flows with levels below pipe obverts. Pipe capacity is dependent on downstream water levels which may exert a backwater effect.

When flowrates are determined by the Rational Method, or from peaks of hydrographs generated by a more complex model, it is assumed that peak flows occur simultaneously throughout the pipe network. Flowrates are constant within each pipe link, and the calculated HGLs and EGLs represent upper envelopes or loci. This model will usually estimate lower pipe capacities than unsteady flow models.

The last model Figure 6.6(c) deals with unsteady flows, represented by full hydrographs typically employed by computer models. Water levels rise and fall and flow characteristics change during a storm event simulation. Various combinations of full and part-full flows occur, along with dynamic effects such as fast-travelling waves generated by changes in flow conditions. This model must be applied by computer, due to the extent of the finite difference computations required. A steady flow system is independent of time, so that only one set of calculations is required, while unsteady model calculations must be repeated for many time steps. It may be necessary to divide pipe reaches into several sections and to perform calculations for each of these.

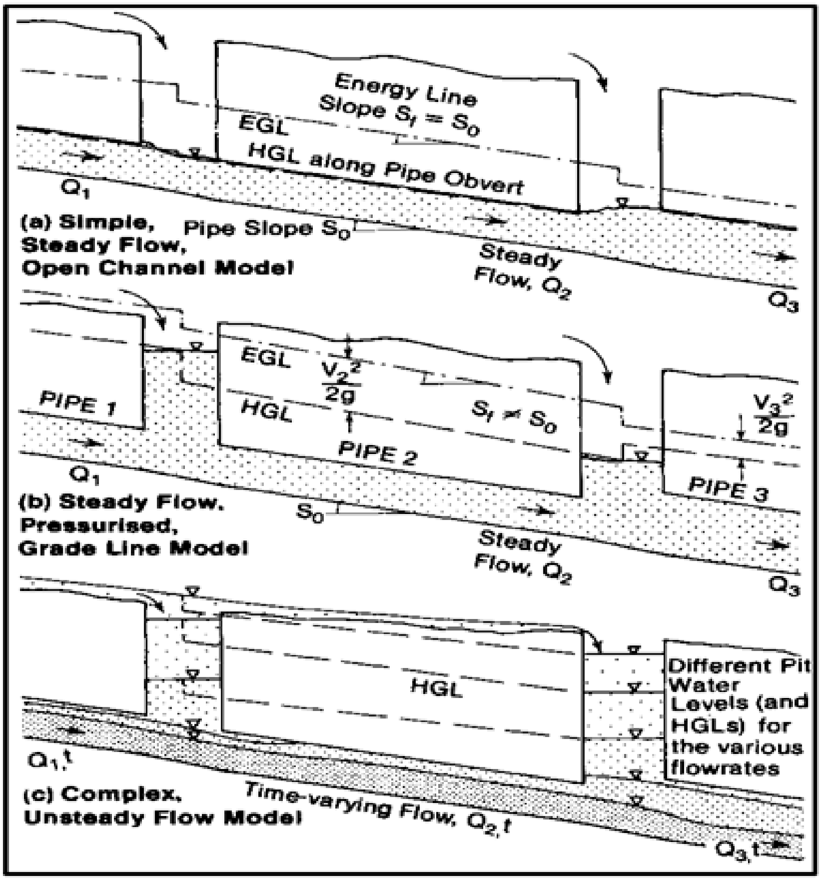


Figure 6.6 Three hydraulic models for pipe systems

All three models can be applied in design and analysis. The first and simplest can be used for design where downstream conditions may be varied, or later adjustments made, so that the system conforms to the assumptions of the model. In analysis of a fixed system this is not possible, and estimated pipe capacities may be incorrect. However, estimated capacities and overflows are usually close to those of more elaborate models and are suitable for preliminary studies.

The second model, which assumes steady flows and connects hydraulic grade line throughout a network, can be used both for basic design and analysis. Since it is better able to model real behaviour, and allows for surcharging of pits and pressure flows, it is likely to give more efficient designs. It may be used as a checking procedure, working backwards up a line from the receiving water level.

This model is presented in Section 6.4 as the preferred hand calculation based method for hydraulic design of simple pipe systems. Typically, calculations involve two passes through the pipe network, the first being top-down, accumulating the flows arriving at each pit or entry point, and allowing for possible bypass flows at pits. With the known flows through the pipe system, the sizes of pipes and the invert levels at their ends are determined, ensuring that HGLs do not rise above a limit, usually 0.15 m below the surface level of pits. This design procedure involves a series of trials with increasing pipe sizes from the available set of diameters. The smallest diameter that meets the design requirements is selected.

The second pass is bottom-up, starting at a set tailwater level, and projecting HGLs upward, allowing for the HGL slope due to pipe friction and the local pit pressure changes. Because flowrates, pipe diameters and submergence levels in pits are known, design charts such as the Missouri and Hare charts in the Queensland Urban Drainage Manual (Queensland Department of Energy and Water Supply, 2013) can be used to determine local pressure changes at pits which are covered later in Section 6.3.4. When the upwards trace reaches a pit where two pipe branches join, the calculations progress up the two branches separately.

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

While some designers of stormwater systems are still using the simple, steady flow procedures, the unsteady models are more accurate, and generate hydrographs and flow volumes, essential for modelling detention storages. Unsteady modelling is the preferred method for detailed analysis of complex pipe networks, where there are strict constraints and accurate modelling of system behaviour is needed. This is especially true for existing pipe networks, when trying to replicate an existing deficit or reproduce a known flooding problem. Examples of unsteady flow programs of this type are DRAINS, SWMM5, xpswmm, and MOUSE.

6.3 Hydraulics of Pits

6.3.1 Introduction

Apart from times of concentration in hydrological methods, two other problematic issues for designers of pipe drainage systems, as shown in Figure 6.7, are (a) the inflow of water through grates or kerb inlets, and (b) the energy losses or pressure changes that occur within pits.

Historically pit losses have been simplified to a single simple coefficient, when in reality there are entry losses to the pit, losses within the pit and exit losses from the pit. Because of the vast number of pit configurations, the simple single coefficient is generally used.

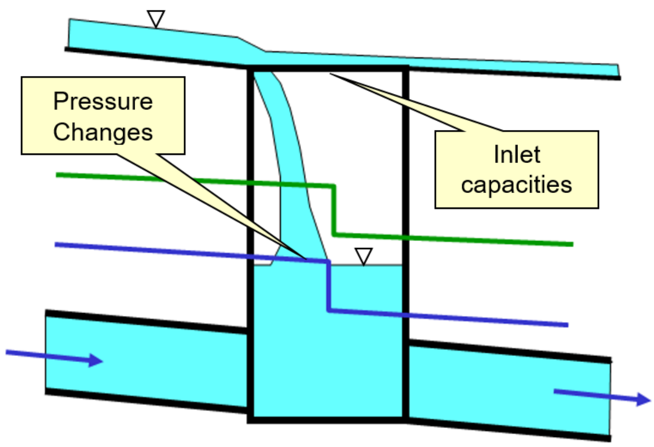


Figure 6.7 Idealised Pit Hydraulic Issues

Inlet capacity relationships are an essential part of the design of piped drainage systems, because they determine the magnitudes of bypasses. Designers are concerned that flow widths and depths are within appropriate limits, both upstream and downstream of a pit.

6.3.2 Pit Inlet Types

Most stormwater enters pipe systems through pits located in gutters and medians of roads. Pits can be classified on the basis of shape or configuration, but they are also defined in terms of their location, on a slope (on-grade pits) or in a depression (sag pits), as shown in Figure 6.8. These operate differently hydraulically, with the on-grade relationship linking inlet capacities to approach flowrates and resulting bypass flows, while sag pits use relationships between the inflow and the depth of ponded water over the pit which cannot escape without passing over a footpath or crown of a road.

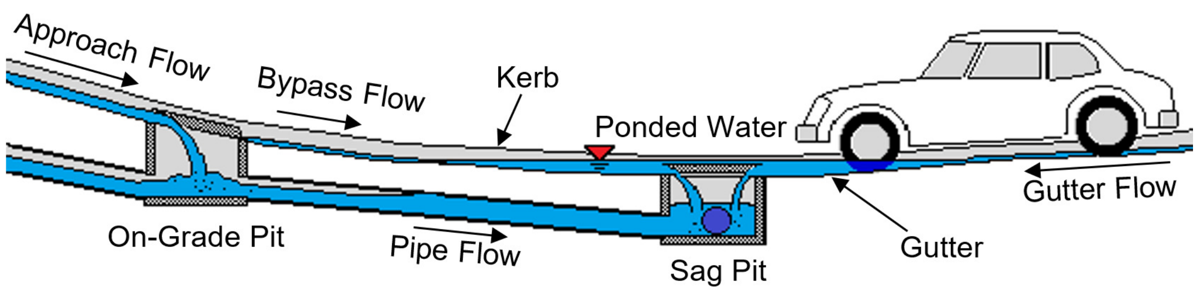


Figure 6.8 Basic Types of Pits

While it is desirable for a pit to collect as much stormwater as possible, this aim must be set against the safety and convenience of pedestrians, cyclists and motorists. An open pit provides the greatest inlet capacity, but is unacceptable in most environments. Openings must not be large enough to admit a child. Grates and depressions associated with inlets should not be hazardous to road users, particularly cyclists. Their use should be avoided on busy, narrow roads. This aspect of inlets has been studied by the U.S. Federal Highway Administration (Burgi and Gober, 1977) in its Bicycle-Safe Grate Inlets Study.

The usual pit entry devices employed are grates and kerb inlets (side entries), separately or in combination. Inlet capacities can be improved by providing extensions to kerb inlets, deflectors (ribs or grooves which direct water into an inlet), depressed grates and gutters, or to employ combinations of pits, such as two or three standard pits end-to-end.

For the great majority of pit types, no relationships are available. In the past several studies of pit entry capacities or “captures” have been made using hydraulic models. Among the most significant are those reported by Burgi and Gober (1977), the Australian Road Research Board (1979), the N.S.W. Department of Main Roads (1979) and Marsalek (1982). General discussions on entry capacities are given by Searcy (1969), Jens (1979), Marsalek (1982), Mills and O'Loughlin (1986), and Argue (1986).

In recent times however, inlet capacities are being developed experimentally using laboratory rigs such as those at Manly Hydraulics Laboratory, NSW, and at the University of South Australia. Unfortunately the relationships obtained from most tests do not extend far enough to model flowrates that may occur in extreme flood events such as 1% AEP or probable maximum floods, so relationships must be extrapolated.

The US Federal Highway Administration has published a general procedure for determining inflow capacities in their Hydraulic Engineering Circular No. 22 (2009) (HEC-22) which have been hydraulically tested by the Bureau of Reclamation for the Federal Highway Administration. Section 4.4 ‘*Drainage Inlet Design*’ gives consideration to the efficiency of various grate types and their impacts on inlet capacities over varying approach grades and velocities. In addition to grate and kerb inlets, slotted drain inlet capacities are also covered for locations where interception of wide sheet flow is desirable and low sediment and debris is expected. The HEC-22 pit inlet procedures are useful for entering into spreadsheets where inflow relationships can be quickly generated.

6.3.3 Pit Inlet Capacities

Sag Pit Inlets

The hydraulic behaviour of on-grade and sag pits is quite different. Sag pit inlet capacities are generally independent of the upstream gutter slopes as they are governed by weir and orifice equations, depending on the depth of ponding. The weir equations apply to flows that enter the pit at its edges, or the edges of bars in a grate. When water ponds above the inlet, usually at depths exceeding 0.2 m, orifice equations are applied. As the depth of ponding increases it eventually reaches a threshold level at which water will overflow as bypass flow, passing over a ‘weir‘ such as a road crown, driveway hump or wall.

Although the approach and cross-fall grades do not affect the pit inlet capacity when using the weir and orifice equations, they can however affect the availability of ponding storage volume surrounding the sag pit, which can indirectly affect the overall behaviour of the sag pit when considered as small detention systems during hydrodynamic analysis.

Sag pit inlets must have sufficient inflow capacity to accept the total approach flow to avoid undesirable ponding such as in intersections where turning traffic is likely to encounter ponding, onto footpaths, into adjacent low lying private properties or basement car parks, or over the crown of a road during a minor storm.

Basic calculations are provided below for determining approximate grated sag pit inlet relationships given by Searcy (1969). The HEC 22 procedures however should be used in preference to the sag pit Equations 6.1 and 6.2, especially when side entry inlet relationships are required.

For a grate,

$$Q_i = \text{BF x } 1.66Pd^{1.5} \quad \text{up to about } 0.12\ m \text{ of ponding } (d < 0.12) \qquad (6.1)$$

or

$$Q_i = \text{BF x } 0.67A(2gd)^{0.5} \quad \text{over } 0.43\ m \text{ of ponding } (d > 0.43) \qquad (6.2)$$

where Q_i is the inlet flowrate (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 9.8 m/s^2).

For depths between 0.12 and 0.43 m, the situation is unspecified, but generally the first equation is recommended.

Charts are provided by the U.S. Federal Highway Administration (Searcy, 1969) for depressed kerb inlets at sag points.

On-Grade Pit Inlets

Calculation of on-grade pit inlet relationships is more complex then sag-pit inlet relationships as several factors can change the inlet capacity. These factors include:

- the approach gutter (or channel) grade which will vary the approach velocity;
- the road cross-fall which impacts the flow width and consequently the maximum allowable flow depth at the inlet;
- the roughness of the gutter and road pavement (or channel);
- the efficiency of the grate; and
- the entry conditions leading into the pit chamber such as gutter depressions (Figure 6.9) and the angle of the throat (Figure 6.10).

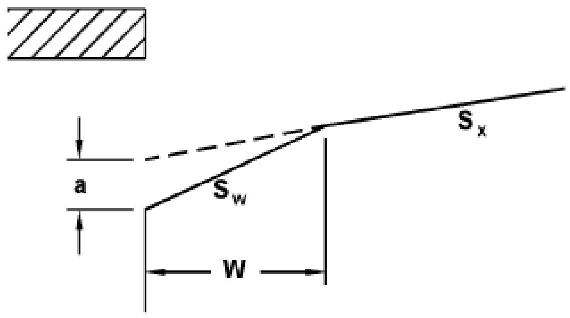


Figure 6.9 Kerb Inlet Gutter Depressions (inspiration from HEC-22 Figure 4.13)

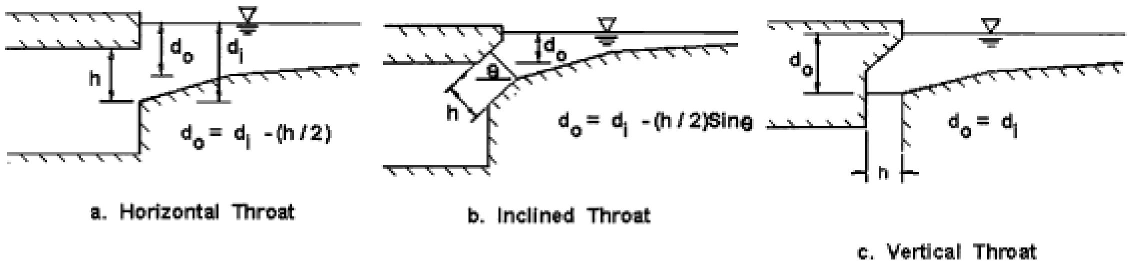


Figure 6.10 Kerb Inlet Throat Angles (inspiration from HEC-22 Figure 4.18)

Basic calculations are provided below for determining approximate on-grade pit inlet relationships for both grate, side entry and combination inlets given by Searcy (1969), however again Equations 6.3 and 6.4 should not be used in preference to the 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 kerb inlets on grade are shown in Figure 6.11 developed from a spreadsheet containing the HEC 22 procedures.

For an undepressed **kerb inlet,**

$Q_i = \text{BF} \times 1.66Ld^{1.5}$ for ponding up to about 1.4 times the height of the inlet,

h ($d = 1.4h$) (6.3)

or

$Q_i = 0.67A[2g(d-h/2)]^{0.5}$ (6.4)

where Q_i is the inlet flowrate (m^3/s),
BF is the blockage factor
d is the average depth of ponding (m)
L is the inlet width (m),
A is the clear area of the opening (m^2), and
g is acceleration due to gravity (m^2/s).

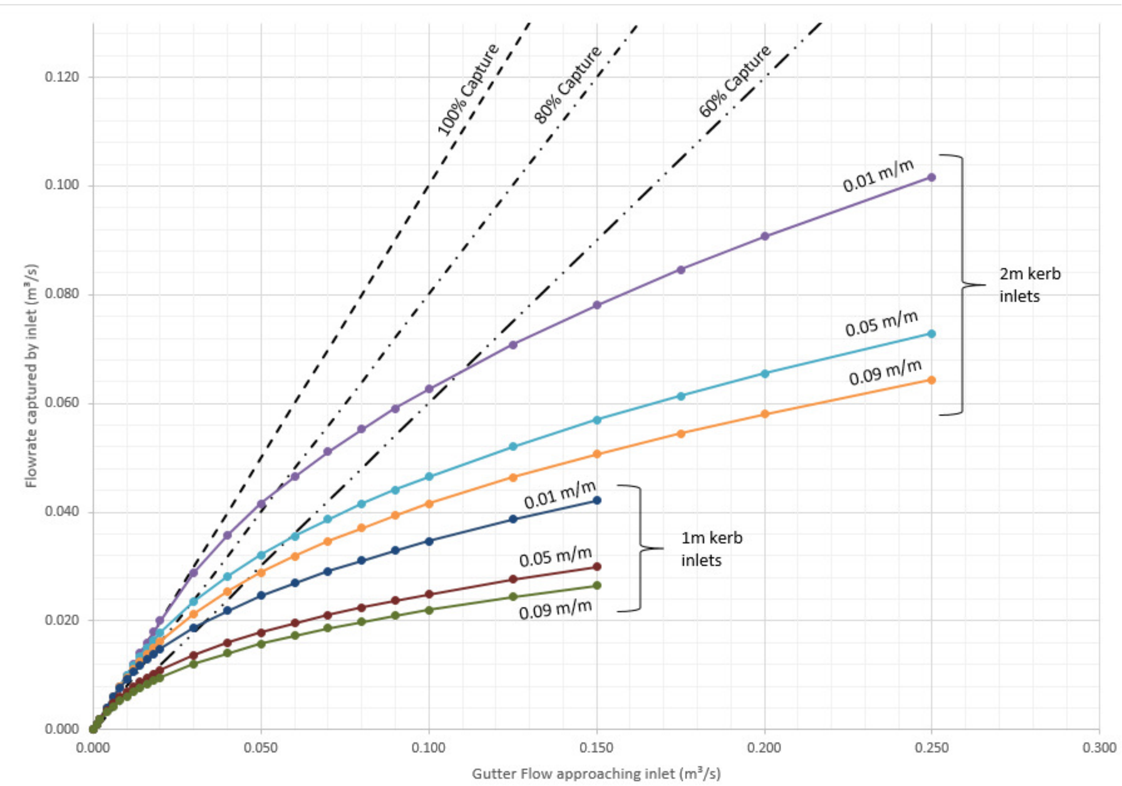


Figure 6.11 Entry capacities for kerb inlets on grade

Additional Information

It is recommended that relationships for the types of pits used locally should be employed. The range of pit types used across Australia is too great for comprehensive information to be provided here. In the absence of mandated equations provided by the local authority, preferences should be given to laboratory based methods.

Other manuals include those provided by VicRoads and Brisbane City Council, and older resources which include the National Capital Development Commission (1981), the Victorian Country Roads Board (1982) and the N.S.W. Department of Housing (1987).

In flow calculations, it may be inappropriate to employ the usual pit entry capacity relationships. Extraordinary flow depths, velocities and debris loads will occur. It is likely that pit entry capacities will be reduced.

In the absence of observations or experimental results, a major and minor blockage factor of 50% is generally applied for sag pits, while for on grade pits the blockage factor can vary between 0% and 20%, dependent on local conditions. A higher blockage factor is often applied for events rarer than the 1% AEP.

Ultimately inlet capacity relationships are an essential part of the design of piped drainage systems, because they determine the magnitudes of bypasses. Designers are concerned that flow widths, depths and depth × velocity ratios are within appropriate limits, both upstream and downstream of a pit. These factors can be controlled by locating pits at suitable places, and by limiting flowrates by providing inlets of sufficient sizes.

6.3.4 Pit Energy Losses

Summary of Pit Energy Losses

Significant pressure losses may occur at pits and junctions. In general, if open channel flows occur in pipes and benching or smooth transitioning is provided in pits, hydraulic losses are reduced. Once pipes are surcharged (i.e. pressurised) higher losses occur at pits. These are offset by the increased capacity of the pressurised pipes, so that as a whole the pressurised system can convey greater flowrates.
Pit energy losses are expressed as a function of the velocity V_0 in the outlet or downstream pipe:

$$h_L = k \, . V_0^{\,2} / 2\,g \qquad (6.5)$$

where

h_L is the loss in m,
k is a dimensionless energy loss coefficient, and
g is acceleration due to gravity (m/s^2).

This represents the change in the total energy line at the pit, as shown in Figure 6.12. The change in the hydraulic grade line is likely to be different, because of different pipe diameters and flowrates upstream and downstream. 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.
The pressure head change is given by

$$\Delta P/\gamma = k_u \, . \, V_0^{\,2} / 2\,g \qquad (6.6)$$

where

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

A similar relationship can be applied to pit water levels, which may be slightly higher than the HGL level, due to the conversion of some kinetic energy to pressure energy as flow crosses a pit

$$WSE = k_w \, . \, V_0^{\,2} / 2\,g \qquad (6.7)$$

where

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

This is also illustrated in Figure 6.12. For most pit configurations, k_u and k_w are very similar, and the water level in a pit can be assumed to coincide with the HGL level.

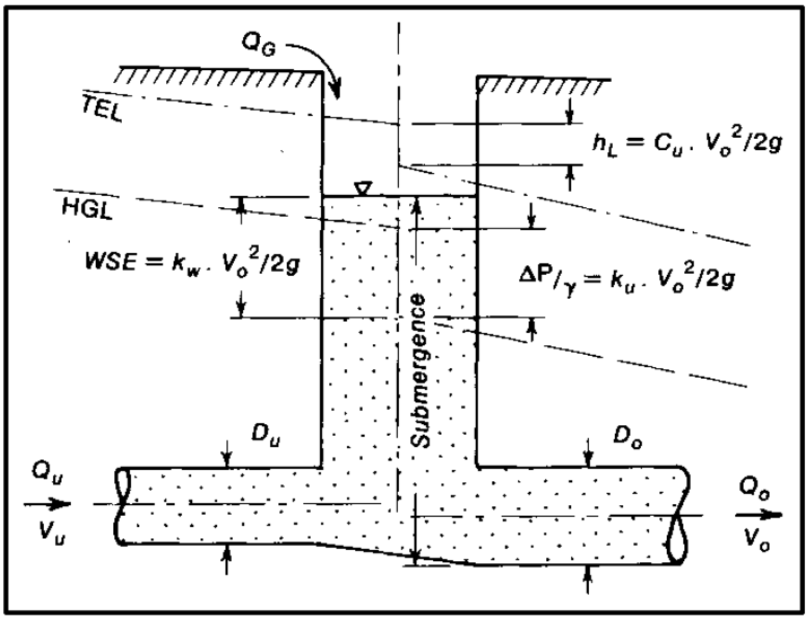


Figure:6.12 Idealised Grade Lines at a Pit.

The arrangement of grade lines in Figure 12 is an idealisation, with all changes assumed to occur at the centreline of the pit. Losses actually occur across the pit, and in the pipe immediately downstream. In tests, the convention is to project gradelines (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

Hydraulic model studies are the only means of deriving reliable values of energy losses and pressure changes for different types of pits and junctions. The most significant study has been the work at the University of Missouri by Sangster et al. (1958). This dealt with pipes flowing full and produced a set of design aids covering certain pit configurations, now termed the “Missouri Charts”. In Australia, Hare (1980, 1983) has produced information on other configurations. Most of the information in the Missouri charts are reproduced in QUDM (2013).

The charts are highly complex in nature with many possible geometric configurations available. Careful judgement is required in selecting the appropriate chart to use for a particular pit configuration, and in practice, iterative calculations will be required to converge to a suitable value.

For large stormwater networks this iterative process can be quite time consuming for the designer. Attempts have been made to replace dependence on charts by proposing 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 Associates Inc (1999) and Stein et al. (1999) has been developed. The FHWA HEC-22 procedure covered in Section 7.1.6 ‘Energy Losses’ has been developed through research and laboratory efforts improving the methodologies of the ‘Corrective Coefficient Energy-Loss Method’ –(FHWA research report by Chang and Kilgore, 1989) and the ‘Composite Energy Loss Method’ – developed in the research report ‘Energy Losses through Junction Manholes’ (FHWA-RD-94-080, November 1994). It is also the only method which covers part-full and full pipe flow, drops and other situations.

The later five of these semi-analytical methods were reviewed in a summary paper (O'Loughlin and Stack, 2002) which concluded that the results of the comparison of algorithms are inconclusive, with no single method being superior. However, the information obtained so far indicated that a viable algorithm can be developed, and that further testing and development is required in order for the methods to acceptably match the full range of pit configurations covered by the original Missouri Charts and the paper by Hare. The paper also found that although the FHWA algorithm appears to provide a significant advance in the determination of head losses and pit pressure changes in stormwater drainage and sewerage systems, comparisons with alternative algorithms and experimental data indicated that the other simpler methods still appeared to give results considered to be at least as good.

Determining Pit Pressure Losses in Practice

Determining pit pressure changes in practice is very complex because there are many possible geometric configuration which are influenced by an almost infinite number of configurations and factors.
Pit geometric configurations can vary according to:

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

While flow variances are impacted by:

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

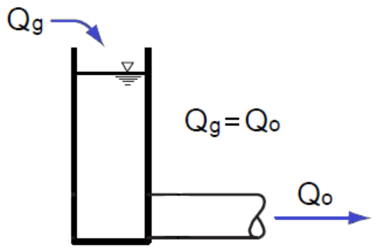
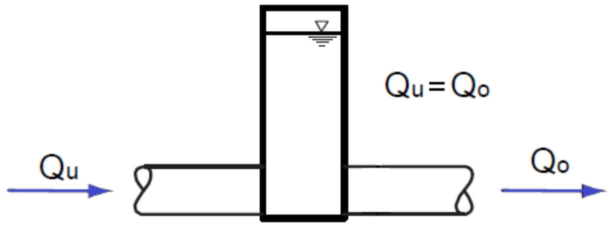
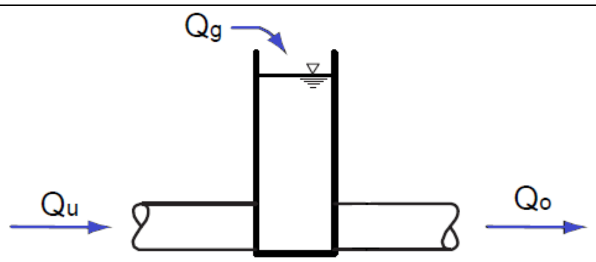
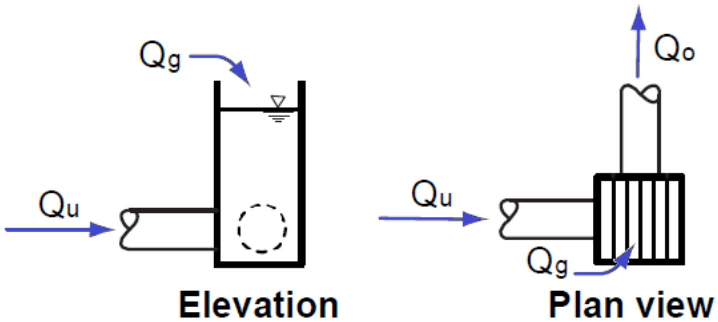
The design calculations typically need to be repeated up to 3 or 4 iterations before converging values are achieved. When designing to satisfy a freeboard requirement, revised coefficients may lead to pit and pipe inlet capacities being revised in a circular manner which requires the designer to intervene.

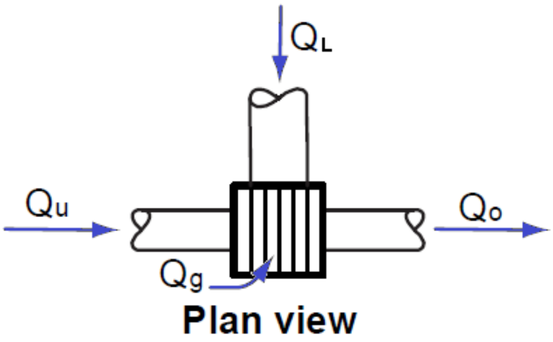
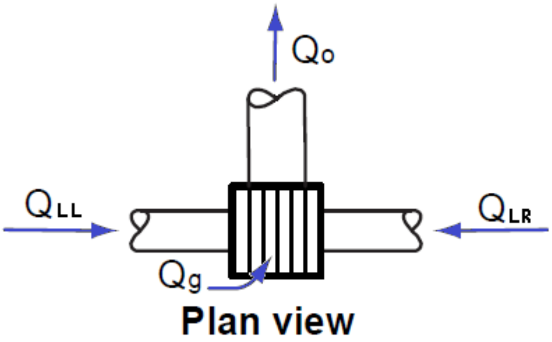
Initial Estimates of k_u before Commencing Iterative Calculations

In order to commence hydraulic grade line analysis of a pipe line, an estimated value of ku is required at each pit. Some government authorities may prescribe suggested values and experienced designers are likely to have developed 'rule-of-thumb' methods for determining these initial k_u estimates. Engineers are encouraged to continue using such methods where effective, to commence their hydraulic design.

If guidance is required for initial k_u estimates then Table 6.1 below provides possible values for a range of common pit configurations. These are not absolute or recommended values in any case for use as final analysis of a system, but only indicative starting points to hopefully reduce the number of iterations required to converge to a final value. Note that these estimations assume shallow pipes with typical minimum covers and no increases in outlet pipe diameters. Deeper pits may increase k_u values, while increases in outlet diameters may reduce k_u values.

Table 6.1 Approximate Pit Pressure Change Coefficients, 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	

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 it has been concluded that these simpler methods still appear to give results considered to be at least as good as more complex semi-analytical methods, further laboratory research and development was recommended so that they may acceptably match the full range of pit configurations covered by the original Missouri Charts and the paper by Hare. These simplified design methods may be considered for use during simple, non-critical pit and pipe network design, however preference should be given to the Missouri Charts and Hare's results.

Recommended Approach

With the Missouri Charts (Sangster et al. 1958) and Hare's results (1980) still widely accepted today, covering an estimated 85% of the enormous number of pit configurations possible.

Two of the charts from QUDM are shown in Figure 6.13. The first is derived from the original Missouri Chart 2 with modification from the Department of Transport, Queensland (1992), for a pit 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 chart is originally from Missouri Chart 4, with the inclusion of Hare's work. The pit comprises straight through flow for a submergence ratio S/D_0 of 2.5, with considerations for grate flow. Here k_u depends on the ratio D_u/D_o, with flow ratios Q_g/Q_o ranging from 0 to 0.5 provided. When the submergence ratio S/D_0 does not equal 2.5, a correction factor needs to be added from Table 6.2.

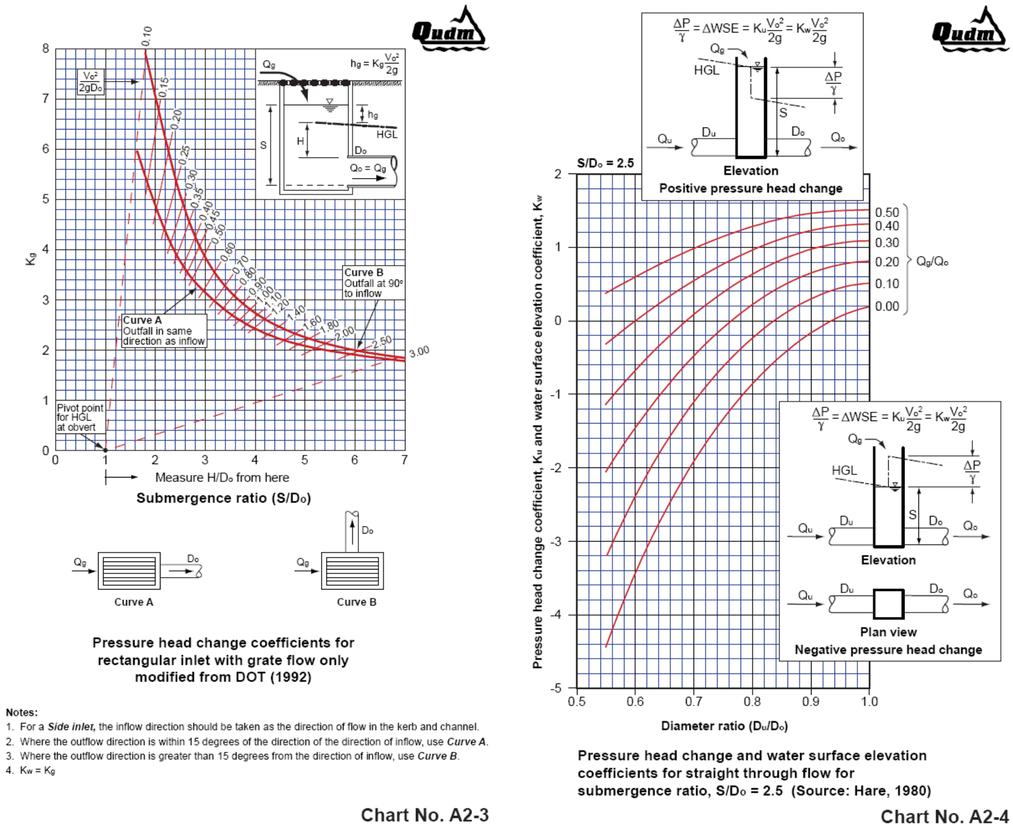


Figure 6.13 Pressure Change Coefficient Charts (Sourced from QUDM, 2013)

Table 6.2 Correction factor for k_u and k_w for submergence ratio (S/D_0) not equal to 2.5 (Sourced from 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	-0.60	-0.09	-0.12	-0.15
3.5	0.00	-0.04	-0.80	-0.12	-0.16	-0.20
4.0	0.00	-0.05	-0.10	-0.15	-0.20	-0.25

As configurations become more complex, additional influencing factors become apparent, such as interpolation coefficients for intermediate grate flow ratios, presence of deflectors, and additional lateral or sideline pipes.

It should also be noted that in the second chart shown in Figure 6.13, k_u can be negative where the outlet pipe is larger than the inlet, and "pressure recovery" occurs due to the downstream flow velocity being lower than that upstream.

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. Where possible, they should be directed into outlet pipes, with the projection of the inlet pipe wholly within the outlet pipe. Hare (1983) states that changes of 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 direction. Typical loss factors for such 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

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 Missouri Charts look up procedures based on the geometry and hydraulic results,
- semi-analytical algorithm based approaches,
- derived pure numerical methods.

6.4 Overland Flow

Overland flow can be conveyed as either sheet flow following the grade of the land or as an overland flow path within a prismatic type channel section. Sheet flow is typically produced during rainfall exceeding a catchment's infiltration capacity resulting in overland flows travelling towards a receiving watercourse or drainage inlet structure. If a drainage system or watercourse is under capacity however, sheet flow can also result as escaping floodwater.

Overland flow paths generally convey stormwater in excess of the minor drainage system capacity, sometimes as bypass between pit inlet structures along a kerb and gutter in the street, along swales in rural or grassed areas, or sometimes undesirably through private property. Overland flow path calculations are similar to open channels in that they can be made up of a number of channel sections, generally with a constant cross-section and slope. A key difference between overland flow paths and open channels however, is that overland flow paths are typically limited to shallower flow depths due to safe design criterion, while open channels typically convey the major storms at much greater depths and flowrates.

Where stormwater pollution is considered, buffer strips or vegetated swales may be combined with overland flow paths as a cost effective methods of stormwater management as they facilitate both flow attenuation and pollutant removal.

Flow Depth and Width Limitations

Limits may be placed on depths, widths and velocities, for safety and prevention of scour and other damage. Where a road cross-section is to be used to convey major and minor flows, various conditions may be applied, with the limiting factor being the criteria which is the most restrictive. These will depend on circumstances such as risks to pedestrians, particularly children, and the importance of the road. In the absence of guidance from the consent authority, the following conditions might apply:

- The depth at the kerb, d_g, should be limited particularly on the lower side of a street, to prevent uncontrolled overflows from entering properties. For 150 mm 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 driveway, plus an appropriate freeboard. This is provided the maximum width of flow is not 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 high 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, 0.6 to 0.7 m^2/s for stability of parked vehicles (depending on size), or as directed by the consent authority.
- In minor storms, or where flows are to be contained on one side of a street, flow depths should not exceed the height of the crown of the road. This includes ponding locations such as at sag pits. Depending on the importance of the road (local, collector, arterial) and problems of access, widths of flow may be limited to allow clear lanes in the centre of a road for passage of vehicles. Flow width limits of 2 to 2.5 m are typical or one traffic lane.
- For major overland flow paths not considered part of the trunk drainage system, and especially for new development areas, flow depths should ideally not exceed the height of the crown of the road by more than 50 mm where possible.

Dimensions of Flow

For trapezoidal style overland flow paths the Manning Equation can be applied. Sheet flow is commonly estimated with a version of the kinematic wave equation for distances up to at most 130m, after which sheet flows will have become concentrated into some form of gully or defined overland flow path (HEC-22, 2009).

Equations for gutter sections can be extended to cover flows along full road cross-sections during major events. For a given flowrate, 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 assumption of such conditions may not be entirely valid, the errors involved are generally acceptable.

Equation (6.8) may be used to prepare design charts which give flow capacities of roadway cross-sections. An example with a possible criterion is shown in Figure 6.14. Allowable zones are defined by the various, limiting conditions.

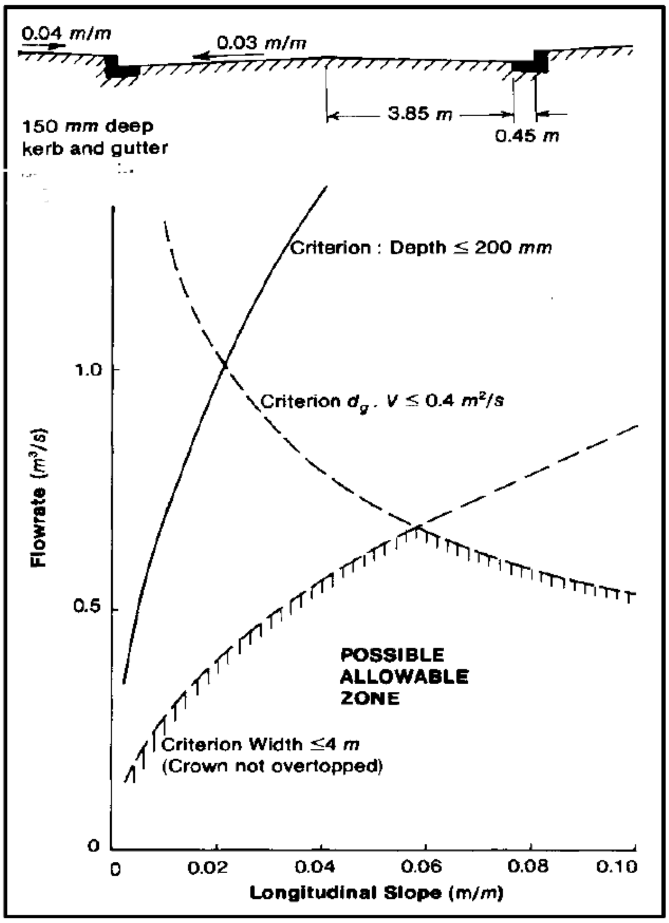


Figure 6.14. Flow capacity chart for one side of an 8 *m* carriageway with 3% cross-slope

Gutter and Roadway Flow Equation

For flows in streets, the following general equation is recommended. It is developed from relationships presented by the U.S. Bureau of Public Roads (Searcy, 1969). Referring to Figure 6.15(a),

$$Q = Q_{ABC} - Q_{DBF} - Q_{DEF} - Q_{GEH}$$
$$= 0.375F\left[\left(Z_g / n_g\right).\left(d_g^{8/3} - d_p^{8/3}\right) + \left(Z_p / n_p\right).\left(d_p^{8/3} - d_c^{8/3}\right)\right].S_0^{1/2} \quad (6.8)$$

where $Q(m^3)$ is the total flowrate, estimated by dividing the section as shown and applying the equation of Izzard (1946) for a triangular channel with a single crossfall or cross-slope:

$$Q = 0.375Fd^{8/3}S_0^{1/2}Z / n \quad (6.9)$$

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_0 is the longitudinal slope *(m/m).*

Where flows are contained in a gutter or on one side of a road, equation (6.8) can be applied in simplified form.

Clarke, Strods and Argue (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 6.15(a). These may be used in the absence of more precise information. Typical values of *n* are 0.012 for concrete, 0.014 for hotmix, 0.018 for flush seal and 0.025 for stone pitchers (Dowd et al.,1980).
Where the face of a kerb is relatively steep, it can be considered to be vertical. For "lay-back" kerbs with sloping faces, equation (6.8) can be applied, taking z_g to be equal to w/d_g as defined in Figure 6.15(b).

If the gutter is a lined or unlined drain or swale, an open channel flow equation such as the Manning Equation can be applied.

Flow depths and widths for a specified flowrate can be determined from equation (6.8). Velocities are estimated by dividing the flowrate by its corresponding flow area, and times of travel by dividing gutter length by velocity. With distributed, lateral inflows as shown in Figure 6.15(c), flowrates and characteristics such as width, depth and velocity vary along the gutter. The average velocity occurs at about 60% of the distance along the gutter towards the pit. Use of the total flow arriving at the pit in gutter flow calculations will overestimate velocities.

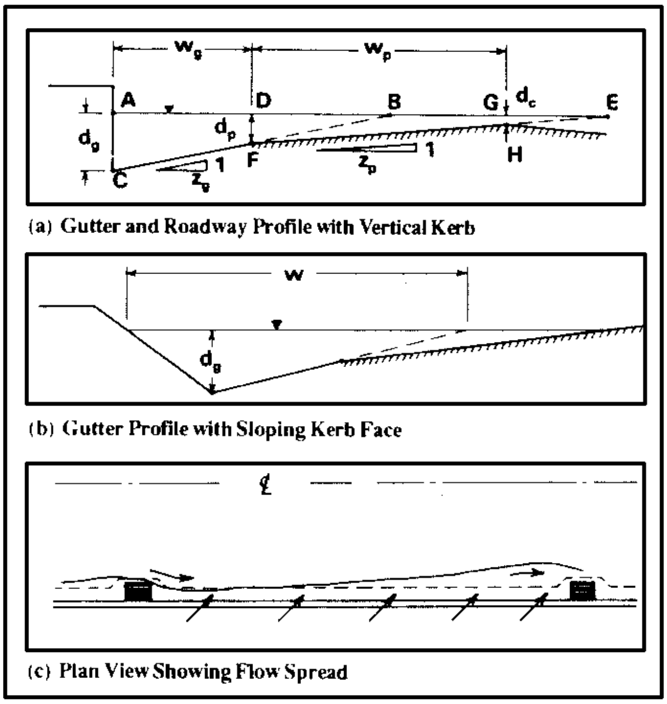


Figure 6.15 Gutter flow characteristics.

Other Considerations

Gutter flow times depend on flowrates, yet it is necessary to specify a time in order to estimate a flowrate. A set of iterative calculations are required. In these, a velocity or time must first be guessed, a flowrate calculated, and a check made to determine whether the total time of flow over overland and gutter flow paths agrees with that originally assumed.

If gutter flow times are to be calculated precisely, allowance must be made 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 flowrate must be estimated to calculate the average velocity and travel time.

Parked vehicles and driveways can interrupt and widen flows. The little experimental evidence available suggests that such effects are localised. Allowance for this factor may be made in streets where close parking of vehicles is likely, but no specific allowance appears necessary at other locations. Provision should be made for possible future alterations to gutter and road profiles, such as resurfacing. At locations where overflows may cause significant damage, effects of possible pit blockages should be assessed.

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

Lastly, it is often necessary to construct over minor overland flow paths, such as for property fencing, sound-control barriers and other major structures. When designing overland flow paths that may contain such structures it is important to consider the potential consequences of flows in excess of the nominated major storm.

6.5 Conveyance Networks

6.5.1 Design Overview

Figure 6.16 shows the general design process for drainage systems made up of components such as pits, pipes, open channels and storages.

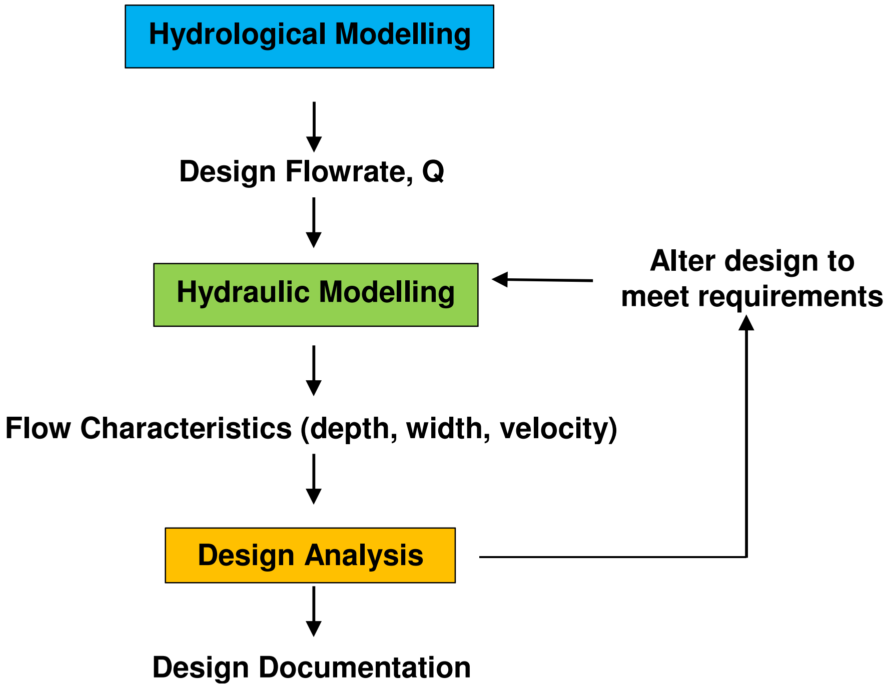


Figure 6.16 The Design Process

It involves a hydrological model that produces a design flowrate, a hydraulic model that converts the flowrate to a set of flow characteristics (depths, elevations, widths and velocities), and a design process that defines factors such as pipe diameters and invert levels.

The steps in the water engineering design process are generally to:

(a) define the design objectives and criteria;

(b) gather the information needed:

- survey information defining topography;
- geotechnical and soil information;
- climatic information;
- a plan of the development or facility to be designed; and
- constraints, such as easements and external drainage networks;

(c) define a trial layout of a drainage system made up of inlets, pipes, open channels, and storages;

(d) run the model to define the sizes and locations of components;

(e) run tests to determine that the system can meet the specified criteria;

(f) by trial and error, define a satisfactory system;

(g) prepare plans, specifications and design reports and provide essential instructions on how to build the drainage;

(h) have the design reviewed; and

(i) obtain approval from the required authorities and proceed with construction or implementation.

The following section focuses on Steps (c) to (f), with components sizing during Step (d) and trial design modelling of the system during Steps (e) and (f).

6.5.2 Pipe System Design with Computer Models

The development of design methods for urban stormwater drainage systems has a long history in Australia, with publications and methods going back to 1911. The rational method, which came into general use in the 1930s, has been gradually enhanced in the various editions of ARR in 1958, 1977 and 1987, and in the QUDM originally produced in 1992.

When access to computers increased, most of the design work for subdivisions and new piped drainage systems was performed by engineers and surveyors using computers with either dedicated drainage programs or through spreadsheet manipulation that typically implement the rational method with sizing based on peak flow estimation. With the increase in computing power design programs are now able to perform the same functions more accurately using hydrograph methods.

The design procedure in computer models is typically implemented much more easily and accurately than the simple design method. The main advantage is the ability of a computer model to perform a design procedure very quickly once a system is set up and the necessary data is entered. In addition, analysis is also possible of both minor and major storm events simultaneously to adequately size pits and pipes to ensure safe overland flow requirements are achieved first time through the design procedure.

6.5.3 Culverts

The simplest pipe system is the single-pipe culvert, which is a common component of highway and railway systems, located wherever an embankment crosses a stream of drainage path. Although there is typically only one pipe or barrel involved (or multiple in parallel), the hydraulics can be very complicated, as indicated by earlier in Figure 6.3. Culvert hydraulics are comprehensively described in the US Federal Highway Administration (FHWA) publication, Hydraulic Design of Highway Culverts, HDS-5 (Normann et al, 2005) which amalgamates and updates material from earlier FWHA manuals.

The treatment of culvert hydraulics (or headwalls) is divided by two flow conditions:

a) Inlet controls – dependent on the orifice effect at the culvert entrance, and

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

When multiple culverts are connected together by pits or junctions, they form a pipe network as discussed in Section 6.4.4 Pipe Networks.

Inlet Control

Inlet conditions are brought about by vena contracta effects, as shown in Figure 6.17.

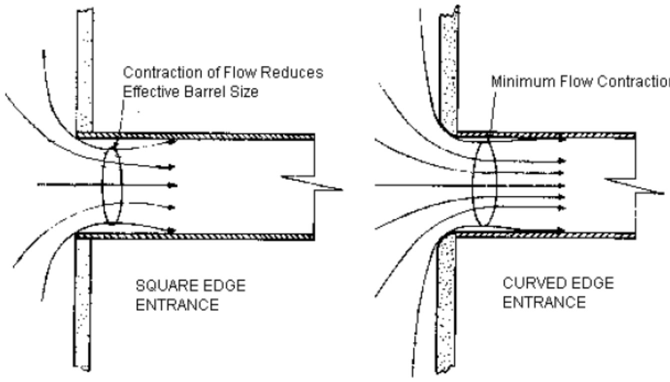


Figure 6.17 Vena Contracta or Contraction at a Pipe Entrance (HDS-5)

The streamlines of flows entering a pipe cannot turn abruptly, so that their curvature continues into the pipe, creating a jet with a diameter less than that of the pipe, thus reducing the available cross-sectional area of flow and the overall flowrate. 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 6.18.

Cc is the correction coefficient for the reduced area, while Cv 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/s2). The overall correction coefficient C = Cc.Cv.

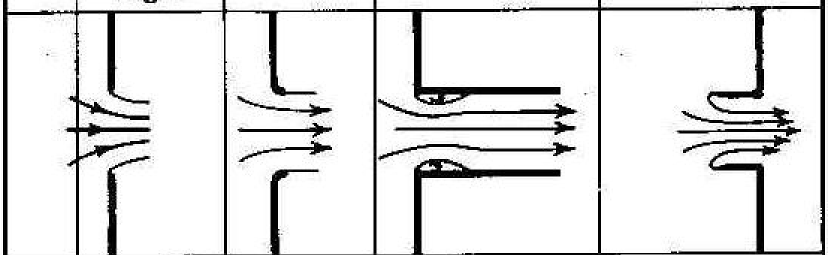
Orifices and their Nominal Coefficients				
	Sharp edged	Rounded	Short tube	Borda
				
C	0.61	0.98	0.80	0.51
C_c	0.62	1.00	1.00	0.52
C_v	0.98	0.98	0.80	0.98

Figure 6.18 Orifice Coefficients

The general case of inlet control is shown in Figure 6.19. From this figure, it is observed that the pipe barrel has a greater capacity than the entrance, as it is flowing part-full. As indicated in Figure 6.17, the capacity can be improved by modifying the entrance such as by rounding sharp edges, thus changing the streamlines. Often, these improvements are not provided during construction. They are useful in retrofit situations when additional capacity is required.

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

$$Q = A.V = C.\,\pi/4.D^2\,.(g.h)^{0.5} \qquad \text{... (6.10)}$$

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 (9.80 m/s^2).

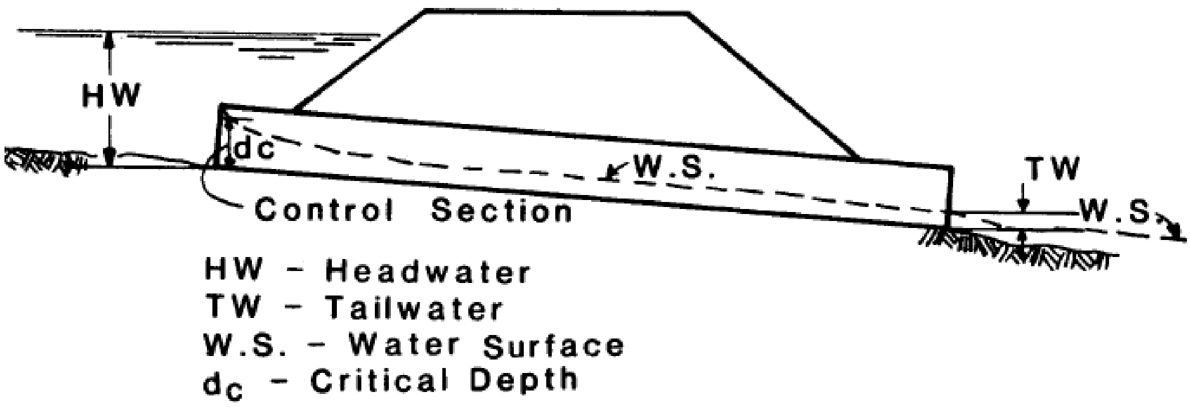


Figure 6.19 Inlet Control Situation (HDS-5)

The entrance hydraulics however are more complicated when the entrance to the culvert is not properly submerged. This involves three states depending on the headwater height above the invert, H and the culvert diameter or height, D:

- part-full flow for $H < 0.8$ D, a weir type flow, as water pours into the pipe.
- part-full flow with $0.8 < H < 1.2D$, type of flow akin to weir flow,
- full submerged inlet flow for $H > 1.2D$, an orifice flow.

The stated limits of 0.8D and 1.2D are approximate. These three zones lead to the behaviour shown in Figure 6.20, taken from HDS-5, where the inlet control relationship changes curvature depending on the headwater elevation. It is also possible to have two different flowrates at the same water elevation, depending on whether the culvert is operating as an inlet or outlet controlled system. The state can also depend on whether flows are increasing or decreasing.

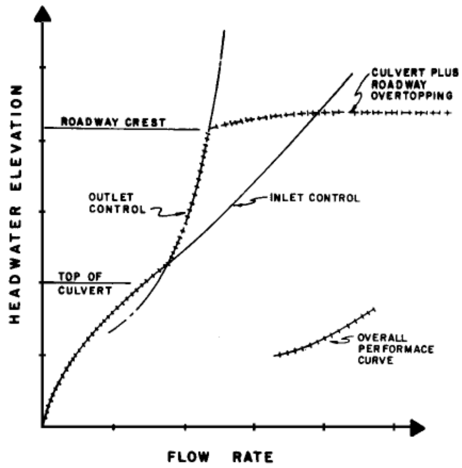


Figure 6.20 Inlet Control versus Headwater Elevation

Design aids are generally in the form of nomographs used to calculate headwater levels for various situations involving circular, box and other types of culverts. An example is shown in Figure 6.21. A better approach is to use available computer software to model culvert hydraulics.

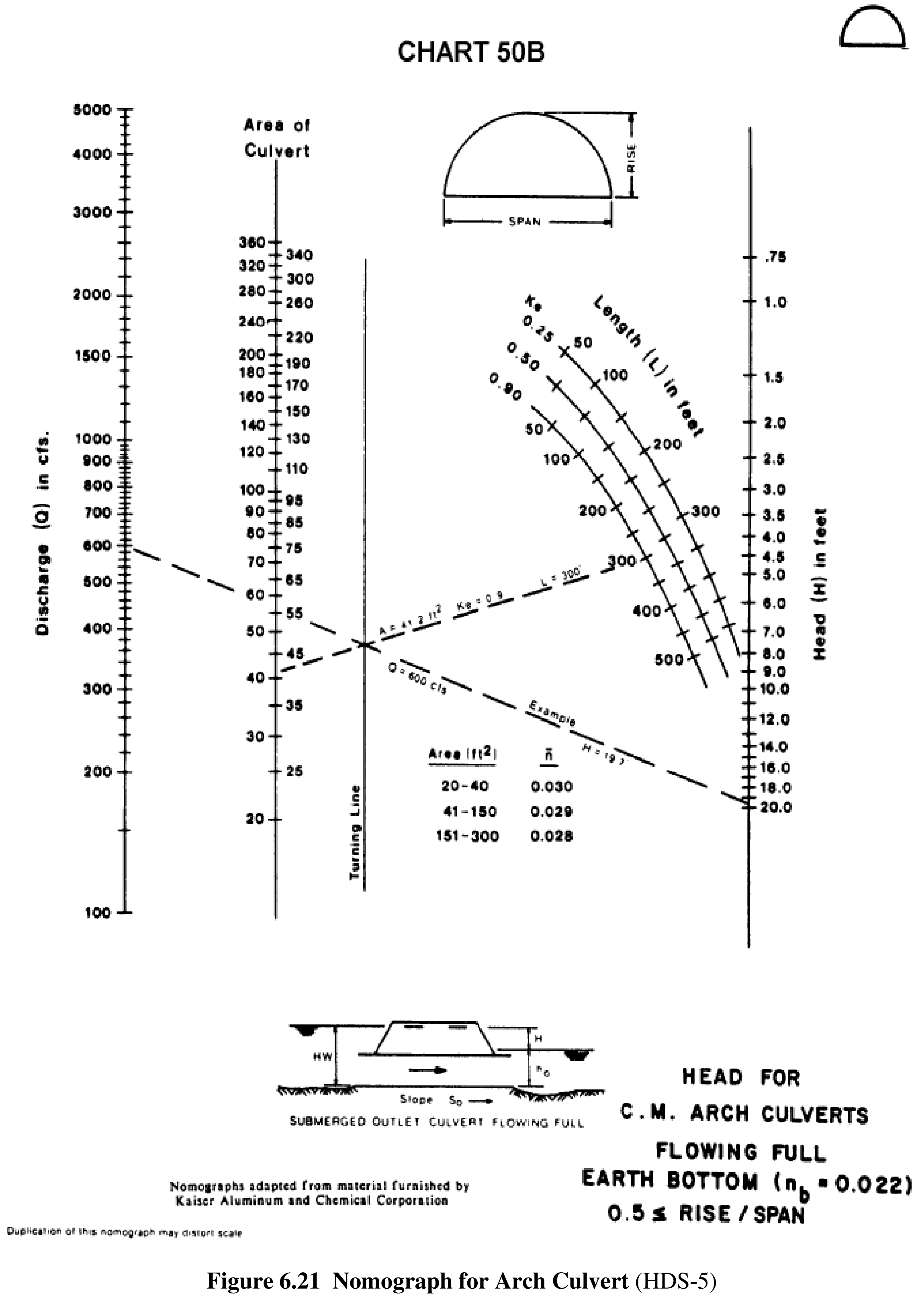


Figure 6.21 Nomograph for Arch Culvert (HDS-5)

Outlet Control

Outlet control occurs when the culvert barrel is not capable of conveying as much flow as the inlet can accept. The controlling section is generally at the barrel exit where subcritical or pressurised flow conditions are occurring or further downstream of the culvert due to tailwater conditions. Two outlet-controlled situations are shown in Figure 6.22. The difference between upstream headwater and the tailwater levels is what drives the water through the culvert. Energy losses are added and equated to the available head.

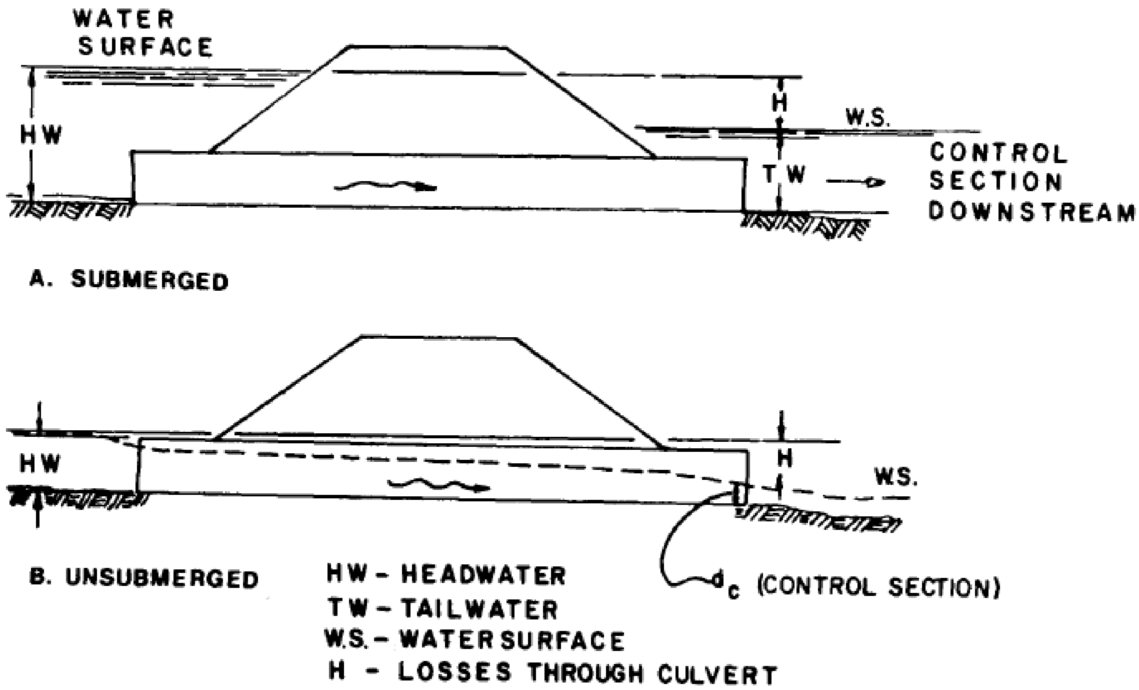


Figure 6.22 Outlet Control Situations (HDS-5)

The starting point for the backwards projection of the HGL is the tailwater level if this submerges the outlet. For a free outfall, different computer models make various assumptions. HDS-5 assumes 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. Other computer models however may assume that it is the lower of (a) the critical depth and (b) the normal depth.

To allow for overtopping of road embankments, a weir equation is applied:

Q = Cw, Lw. H1.5 ... (6.11)

where Cw is a weir coefficient, depending on the weir shape (Figure 6.23),
Lw 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).

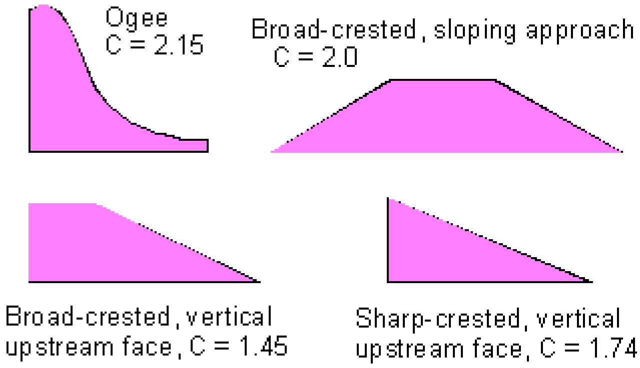


Figure 6.23 Weir Crest Shapes (from Laurenson et al., 2010)

The culvert and overflow weir outflows can be combined into a composite relationship, as shown in Figure 6.20. This needs to take account of inlet and outlet control and usually the most conservative relationship, giving the lowest flowrate for a given depth, is accepted.

The real behaviour or a culvert is more complex, involving a phenomenon called 'priming'. As upstream water levels rise, culverts tend to remain under inlet control until they run full. As upstream water levels drop, the culvert tends to 'stick' in a full-flow, outlet control configuration, until there is a sudden reversion to inlet control and drop of headwater level.

Since culverts are often used as outlets for larger detention basins. The relationships presented above can be applied to specify the elevation - discharge relationship needed for routing of flows through basins.

6.5.4 Pipe Networks

Networks of stormwater pipes acting under gravity are usually dendritic, or tree-like. Flows collected in a number of branches converge to junctions along main drains, and flow to an outlet. Inlets at the tops of branches and along branches:

- admit stormwater,
- provide a node where pipe diameters and directions can change,
- provide access for inspection and maintenance, and
- in some cases, provide a convenient overflow point.

In some cases, pits can be sealed with a bolt-down lid. These may be called manholes, junctions or junction boxes. Pits 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 more capacity. Pipes can therefore flow backwards, and this can also happen in dendritic networks under some circumstances.

For simplicity, pipes are laid straight and at a constant slope in almost all cases. They are available in a set of standard diameters supplied by the manufacturers. Plastic pipes typically start at 90 mm diameters and increase to about 600 mm, while reinforced concrete pipes may start at about 225 mm and increase to over 2 metres diameter. Road authorities usually specify a minimum size of 300 to 375 mm within the road reserve, for ease of maintenance.

Certain styles of pipe system layout are favoured at various locations. Figure 6.24 shows the types of systems used in New South Wales and Queensland. In the latter pipes are located under road centrelines and manholes are used as collectors from inlet pits. Differences in terminology also occur. In New South Wales and the US, 'kerb and gutter' is used, while in Victoria, Queensland and parts of New Zealand, the term is 'kerb and channel'.

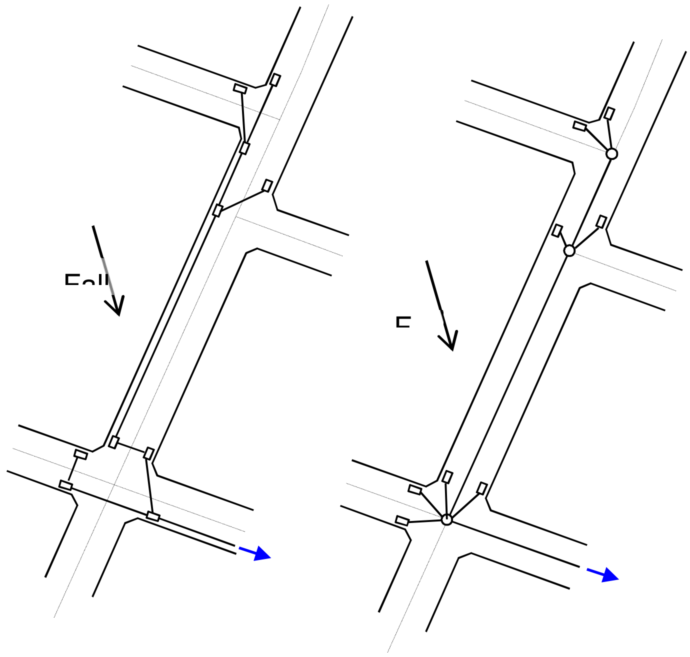
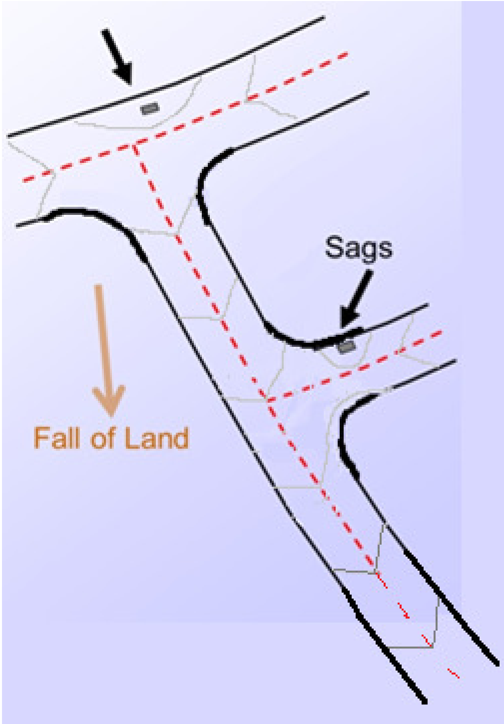


Figure 6.24 Pipe System Layouts in New South Wales and Queensland

It is vital that flow paths be provided for major flows. Ideally, these should be on roads or through open space and pedestrian paths. Flow paths through private property should be provided as a last resort and will require an easement (a legal instrument giving a party the right to drain stormwater through the site and for councils to enter the site for maintenance). Flows directed through sites are a hazard and inhibit the development of the property, as an easement cannot be blocked or built upon.

The next step is to set out a pipe system. This needs to take account of a number of constraints. Major ones will be the surface flows occurring at intersections of streets, where they can cause nuisance to pedestrians and motorists. Usually, a road design with profiles showing high and low points will be available for greenfield designs, and elevations of points along road centrelines and gutters can be determined from this.

The positions of sag pits in trapped low points will usually dictate where pipelines must go, 'joining up the dots' between such pits, as shown in Figure 6.25.



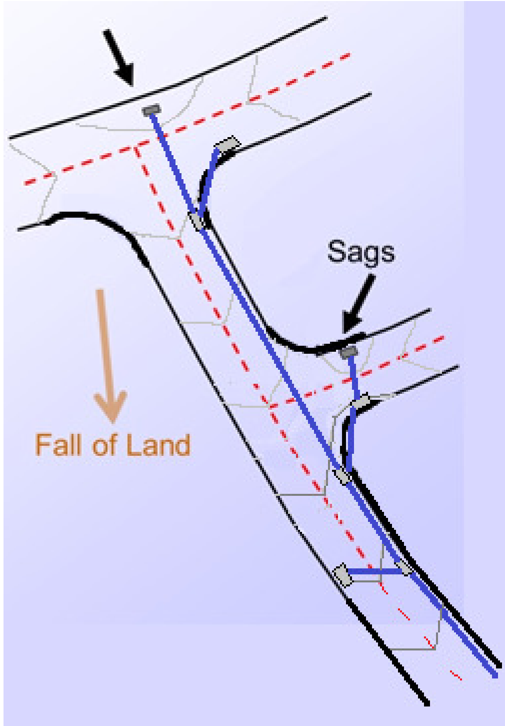


Figure 6.25 Typical Pipe System Layouts

In Figure 6.26, at the intersection to the right, a pit must be located upstream of the tangent point (Point E) to prevent excessive surface flows running round the kerb return. Bypass flows from this pit can be picked up by the pit at Point D.

The other pits at the intersection are located along the path of surface overflows to collect both minor and major overflows. The pit layout allows pedestrians to cross at the corners without being exposed to large widths of flows.

Figure 6.26 also indicates how pits at the top of a drainage system can be located. Since street gutters are flow components, it is desirable to use these as much as possible, and only provide more expensive pits and pipes when the flows cannot be adequately carried in gutters. This is usually taken to be when the width and depth of flows in gutters becomes excessive, preventing pedestrians from crossing streets and making conditions difficult and potentially dangerous for motorists. The Figure 6.26 example comprises a flow width criterion of 2m, however in the absence of guidance from the local authority, a width of 2 to 2.5 m is typical and has been discussed in Section 6.4. In addition to this, suitable pit locations may also be determined from percentages of flows captured, depth of flow in the gutter, and a velocity-depth ratio relationship.

For economy, a designer would like to collect all flows from the upper side of the street in Figure 6.26 with only the pit at Point D, without having to establish a side-line. To check whether this can be done, a trial point is defined, say at Point A where flows from the corresponding catchment are calculated, and the width of the gutter flow at Point A is estimated. The width will increase along the gutter length as the areas of contributing catchments increase. A pit must be located whenever any of the criterion limits are reached.

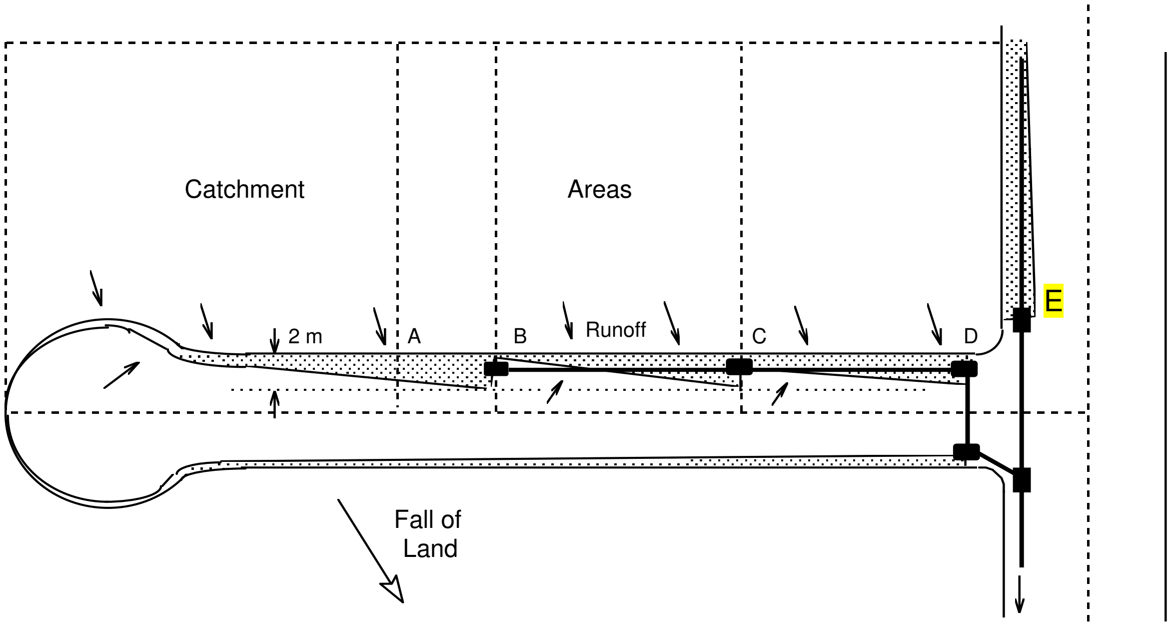


Figure 6.26 Location of Pits and Pipes at a Simple Street Location

If all the flow is captured by a pit at B, the flow will be reduced to zero just downstream, but will increase again along the gutter due to lateral inflows from the catchment. It is unlikely however, that on-grade pits will capture all flows during a minor storm, which will result in bypass flow downstream of the pit. This is shown for the pit at Point C where the width again reaches 2 m and is reduced due to the pit, however there will be bypass flow and some width of flow just downstream of the pit. The flow widths along the gutter will typically follow a saw-tooth pattern.

Positions of pits may also be set by the need to provide pits at significant locations, such as near a school with street crossings. There are also minor aspects of good practice such as the location of pits upstream of driveways, rather than downstream, or avoidance of clashes with other services. Another consideration is to allow for additional pipe connections from private property that are not always included with the street drainage system calculations. This may include directly connected pipes from sources such as inter allotment drainage, onsite detention systems, or from major commercial developments. What is usually considered as the first pit on the line, may actually be receiving considerable pipe flow from upstream private property.

The designer needs to decide on the density of the network, which will typically come down to meeting the guidelines set by the authority. For example, Figure 6.27 shows two arrangements of pits at an intersection that may be acceptable in two different scenarios.

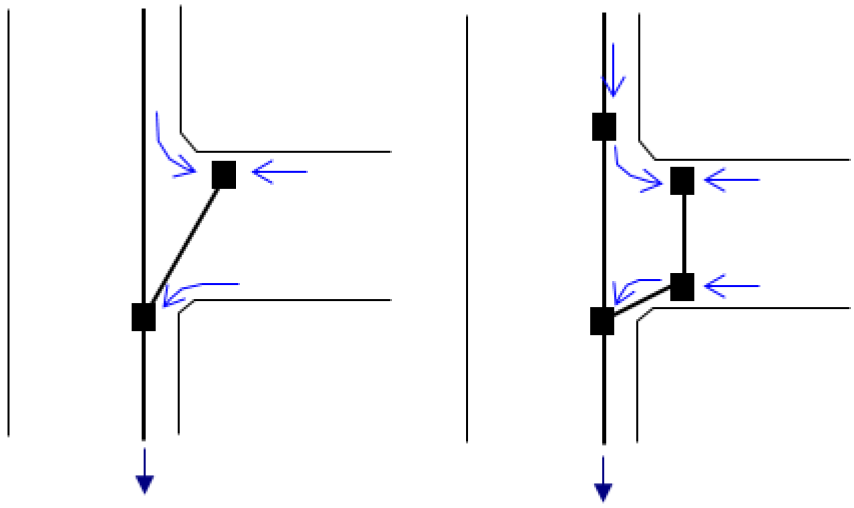


Figure 6.27 Alternative Pit Layouts

The arrangement on the left shows an intersection with two pits, while the other layout uses four pits. The decision of which is more appropriate to use depends on the magnitudes and consequences of the flows that may bypass the pits. Thus, in a densely-developed area, where overflows will cause nuisance and damage, the greater number of pits will be preferred. On a lower density development, where surface flows can be handled well and consequences of failure are small, fewer pits may be used.

6.5 Design Procedures

Stormwater design has been comprehensively covered in numerous guides, both nationally and internationally. These include, but are certainly not limited to QUDM, AUS-SPEC 'Handbook of Stormwater Drainage Design', the US FHWA HEC-22 and others. Each of the guides cover various aspects of stormwater design in different degrees of detail, often concentrating on key areas related to the overall design focus (for example subdivisions or main highways) or for problematic areas of concern related to the guide's general locality or several past events. Selection of a suitable design procedure is generally up to the user, as long as the final design and methods employed meet the requirements of the approval authority.

Following is an overview of possible design aspects covering design criteria and the general process that may be considered during the design of piped drainage systems. Although this section primarily focuses on hydraulics and hydrology, and the design safety requirements, there are other important aspects that should be considered during the planning and design of piped drainage systems. These include constructability, aesthetics, future maintenance, direct costs and other long term economic factors, and the liability of the system.
Design Criteria for Piped Drainage Systems

The overall hydraulic criterion is to define a pipe system that limits surface flows to safe limits. The primary requirement that applies within pipe systems is that pit water levels should be below the top of pit or invert of gutter level by a freeboard. This prevents pits filling to the brim under design conditions, inhibiting flows from entering. Freeboard is typically a factor set by the relevant consent authority, however, historically ARR87 assumed a freeboard level of 150 mm to be acceptable. In the absence of other guidance from the consent authority, 150 mm is a reasonable value to use. Some authorities may also specify maximum or minimum velocities.

The limits are intended to ensure that systems operate at given levels of service without causing flooding of properties, nuisance or hazards to pedestrians and cars on streets. The relevant consent authority should specify AEP's levels for both the minor and major storms for various types of land use. Designs usually involve both of these levels, the minor to size pipes or channels and the major to test that failures will occur safely. Figure 6.28 indicates the expected occurrences under different design assumptions.

In the absence of guidance from the consent authority, AEP levels should be selected to reflect the importance of the facility being designed and the consequences of its failure. Some examples are:

- Roof drainage systems – 5% to 1% AEP;
- Street drainage piped systems – 0.5 EY to 10% AEP for minor flows, 2% or 1% AEP for major flows;
- Trunk drainage systems – 1% AEP or higher, with checks on effects during PMP storms;
- Stormwater treatment and sediment control devices – 4 EY to 1 EY;
- On-site stormwater detention systems – the requirements vary, but as a minimum to match at least two AEP levels, typically the minor and major; and
- Detention systems that may endanger lives if failure occurs – probable maximum precipitation (PMP), the highest rainfall that can occur.

Both design and analysis involve modelling the operation of a system in a critical situation, defined by a set of rare storms that will test the system. Typically, a designed drainage system is shaped and sized to cater for critical storms of a certain magnitude, defined by an AEP. This approach recognises that:

- It is not practical or economic to design all systems to be free of failure. To do this for a normal pipe system would involve very large and expensive pit and pipe systems. These would occupy such a large space that it should be difficult and expensive to provide other infrastructure services, such as water pipes and electricity conduits;
- Failures will occur due to large or extreme rainstorms or to 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, accepting controlled failure;
- Ideally, the acceptable level of risk should be set by some economic analysis with public participation; and
- When failure does occur, its effects should be limited by providing a 'fail safe' system that does not fail disastrously.

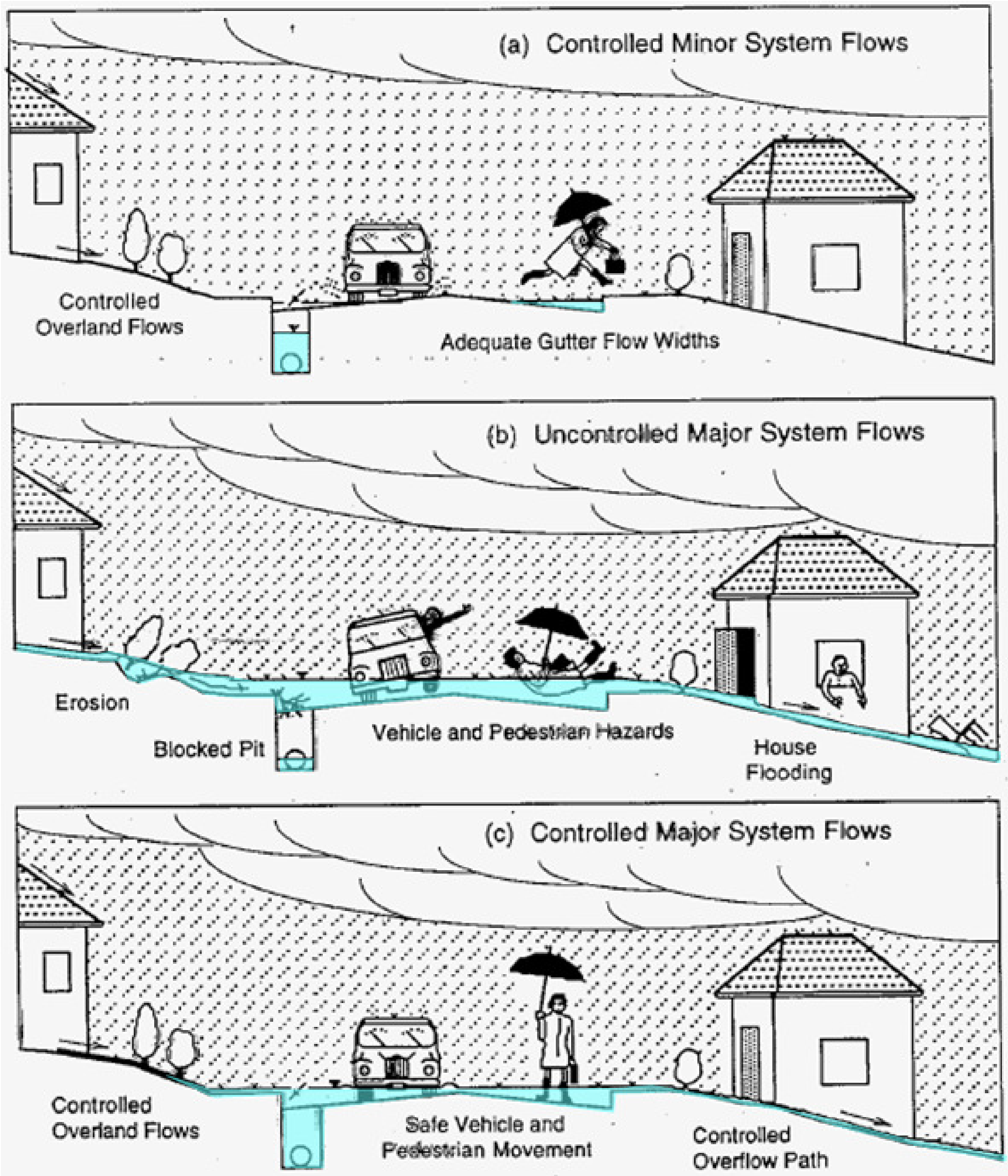


Figure 6.28 Operation of the Minor and Major Stormwater Drainage Systems

To ensure that damage and personal danger due to failures are limited, checks are required, using analysis techniques. Some failures of the system and overflows can be expected during major storm events, as shown in Figure 6.28, but the system should operate without causing safety hazards or large-scale property damage.

A summary of design steps has been provided below. Many of these steps have been presented previously in earlier sections, nonetheless some information may be duplicated here for clarity of the methodology process.

Preliminaries of Designing a Pipe Network:

Before starting the design and analysis of a stormwater drainage network, assuming that applicable road and subdivision layouts have already been established, there are a number of preparatory tasks that should be undertaken. These include the need to:

(P1) Decide on the stormwater design standards and methods to be adopted and to select suitable design rainfalls;

(P2) Ensure detailed survey and work as executed information on the topography of the land and any existing drainage systems is up to date, best confirmed by undertaking a site inspection. A services search is also recommended;

(P3) Obtain existing flood and stormwater information relevant to the site. This may include information such as:

- Existing flood study reports;
- Historical flood records and rainfall hyetographs from local Council or other sources; or
- Consultation with local authorities and community to gain an understanding about existing problematic drainage systems already causing issues.

(P4) Ensure considerations have been made during the planning stage for major system overland flow paths and WSUD (if required). When this has not been the case:

- If possible, make adjustments to the proposed road alignments and subdivision layouts early on, before committing valuable time and expense on design and analysis of a drainage system that may later prove difficult and expensive to achieve and construct a suitable outcome, or that may force expensive un-planned changes to a proposed development in the later stages of a project; or
- If roads and subdivision layouts are fixed (i.e. brownfield and infill development) identify probable constraint locations where major (and minor) system restrictions are likely to arise (such as trapped sags or other topographical constraints) so feasible solutions can be conceptualised early in the design process.

(P5) Determine which hydraulic and hydrology software package is best suited for the proposed system, or whether hand and spreadsheet calculations are acceptable.

Design – General Process of Designing a Pipe Network:

Design is the initial process of locating and sizing pits, pipes and overflow paths, along with the estimation of major and minor flows from sub-catchments so that later analysis can be carried out. The general sequence is:

(D1) Pits (refer Section 4.4.3.3):

- Determine suitable pit locations (e.g. kerb return tangent points, maximum spacing between pits, sag points)
- Specify pit surface levels & inlet type (on-grade, sag or sealed)
- Select an appropriate pit type (side entry, grated or combination) and allocate an inlet size, taking blockage factors into consideration.

(D2) Pipes (refer Section 4.4.3.2):

- Link the pits with pipes, noting the length and type of pipe (under road / not under road), and specifying minimum cover and grade requirements.

(D3) Overflow Routes (refer Section 4.4.3.4):

- Define location of overflow routes,
- Determine minor and major safety requirements, calculating safe maximum limits (flow widths, depths and velocity-depth ratios),
- Identify any critical locations where overland flows may lead to undesirable nuisance flooding, damage, or danger to life (footpath areas, driveway laybacks leading to low lying properties, or basement car parks).

(D4) Sub-Catchments

- Define and measure sub-catchments and apply appropriate characteristics

(D5) Estimation of Minor System Design Flow Rates

- Commence hydrological calculations for minor storm flow rates to determine necessary pit inlet sizing to satisfy safe overland flow route conditions, followed by determination of required pipe sizing to convey the underground network flows.

(D6) Calculation of Major System Design Flow Rates

- Perform calculations to determine major storm flow rates and compare with safe limit capacities of roadways and other major overland flow routes

(D7) Hydraulic Pipe Calculations

- Perform hydraulic pipe calculations to determine pipe sizes and to fix pipe inverts, with allowance for cover and minimum slope requirements.

(D8) Hydraulic Grade Line Analysis

- Perform possible hydraulic grade line analysis, working upstream from controlling or receiving tailwater levels.
- When tailwater levels are not available, past practise has been to adopt the pipe soffit level to commence HGL analysis. A conservative approach is to adopt the pit freeboard level, especially if draining into an existing downstream system which is assumed to have been designed to capacity.
- Provided HGL's are not steeper than the grade of the pipe (pressure flow), pit freeboard should be achievable when performing HGL check calculations upstream.
- If the downstream system is known to have limited capacity or flooding issues, or if the natural surface of the land is quite flat (e.g. less than 1% grade) then computational hydrodynamic modelling is recommended.

Note: Major flows may be estimated before minor flows or in parallel with them.

Analysis – General Process of Analysing a Pipe Network:

Analysis of pipe systems is more complex than design, and the steps involved depend on the nature of the problem. Generally, the sequence is:

(A1) Definition of the minor system and its characteristics, noting any problem areas where roadway flow widths are likely to be a limiting criterion under minor storm conditions;
(A2) Simulation of system behaviour, including major and minor flows, during design and historical storms;
(A3) Analysis of both major and minor results, including the identification of pipes and pits with inadequate capacities, and resultant overflows that exceed any major or minor safety requirements;
(A4) Development of an improved system with a large capacity. This may be achieved by one or more of the following options:

- increasing the size or number of inlets to the underground pipe network;

- increasing pipe diameters, or lowering the inverts to potentially lower the HGL;
- increasing overflow route capacities by altering carriageway or channel profiles to safely convey larger flowrates;
- adjusting road profiles / piped networks to divert flows where appropriate to an alternative overflow route or piped network with spare capacity, provided cross catchment flows do not occur, or unsafe conditions result elsewhere;
- consideration for detention storage to reduce peak flows; or
- consideration for WSUD, such as forms of infiltration or stormwater harvesting / retention, though this is unlikely to resolve capacity issues for the major and minor storms.

(A5) Simulate the behaviour of the improved system, then assess if further improvements are required.

Resolution of Common Drainage Design & Analysis Issues:

During the design of a new drainage network, the overriding objective is to ensure that overland flow paths are safe in minor and major storms. Unless there are specific constraints such as avoiding other infrastructure, cover, or needing to match into existing systems, pipes can generally be amplified, duplicated, or augmented, while pit inlets can be increased or additional pits can be added to the network. Overland flow paths are generally harder to alter as they must follow the grade of the land, and are typically set by a standard road cross section profile, or are limited to an easement when flowing through property. With these constraints in mind, following are two possible consideration that may be used to resolve unsafe overland flow paths:

(R1) When upstream approach flow results in an unsafe overflow path approaching a pit:

(a) If the associated catchment flow rate is significantly contributing to this unsafe condition, then consider making the catchment smaller by relocating / shifting the pit upstream or adding additional pits upstream (splitting the catchment), and / or

(b) If significant upstream bypass flows are contributing to this unsafe condition, then consider making upstream pit inlets larger to reduce the amount of accumulating bypass flows approaching the subject pit.

(R2) When downstream bypass flow results in unsafe overflow route conditions leaving a pit:

- Increase pit inlet capacity to reduce the amount of bypass flow. This may include adding or increasing grate and lintel size, constructing a custom larger pit inlet structure if feasible. If this does not solve the unsafe condition, then consider solutions from (R1) above until safe overflow conditions have been achieved.

Simple Rational Method Example

An example has been prepared that presents design flowrate estimation by the Rational Method. Hydrological calculations are set out in three design sheets in Sheets A to C, with related hydraulic calculations presented in Sheets D and E. A detailed description of the sheets has been provided in Tables 6.3 and 6.4.

Design calculations are performed for the stormwater drainage system shown in Figure 6.29, assumed to serve a hypothetical 10 lot residential subdivision, with a 7.2 m wide access way with barrier kerb grading down to a cul-de-sac head. The upstream external catchments are limited and there are no detention basins as part of the analysis. The proposed drainage system contains five stormwater drainage pits and pipes, and discharges from a headwall into a watercourse. The system is predominantly one single pipeline with one short sideline to reduce overland flows along the opposite side of the carriageway.

The land-uses involved are residential with an assumed 60% impervious area, roads with an assumed 80% impervious area and for sub-catchments representing both residential and road areas, an assumed 70% impervious area has been applied. A blockage factor of 20% and 50% has been applied for on-grade and sag pits respectively.

The pipeline is affected by a 1% AEP flood level from the watercourse at 74.2 m AHD at the point of discharge and is above the 0.5 EY (20% AEP) flood level. The system has been designed to safely convey the 0.5 EY (20% AEP) for the minor storm and the 1% AEP for the major storm. Due to the nature of the site and close proximity to the watercourse, the 1% AEP overflows have been allowed to pass through the private property within a table drain and an easement.

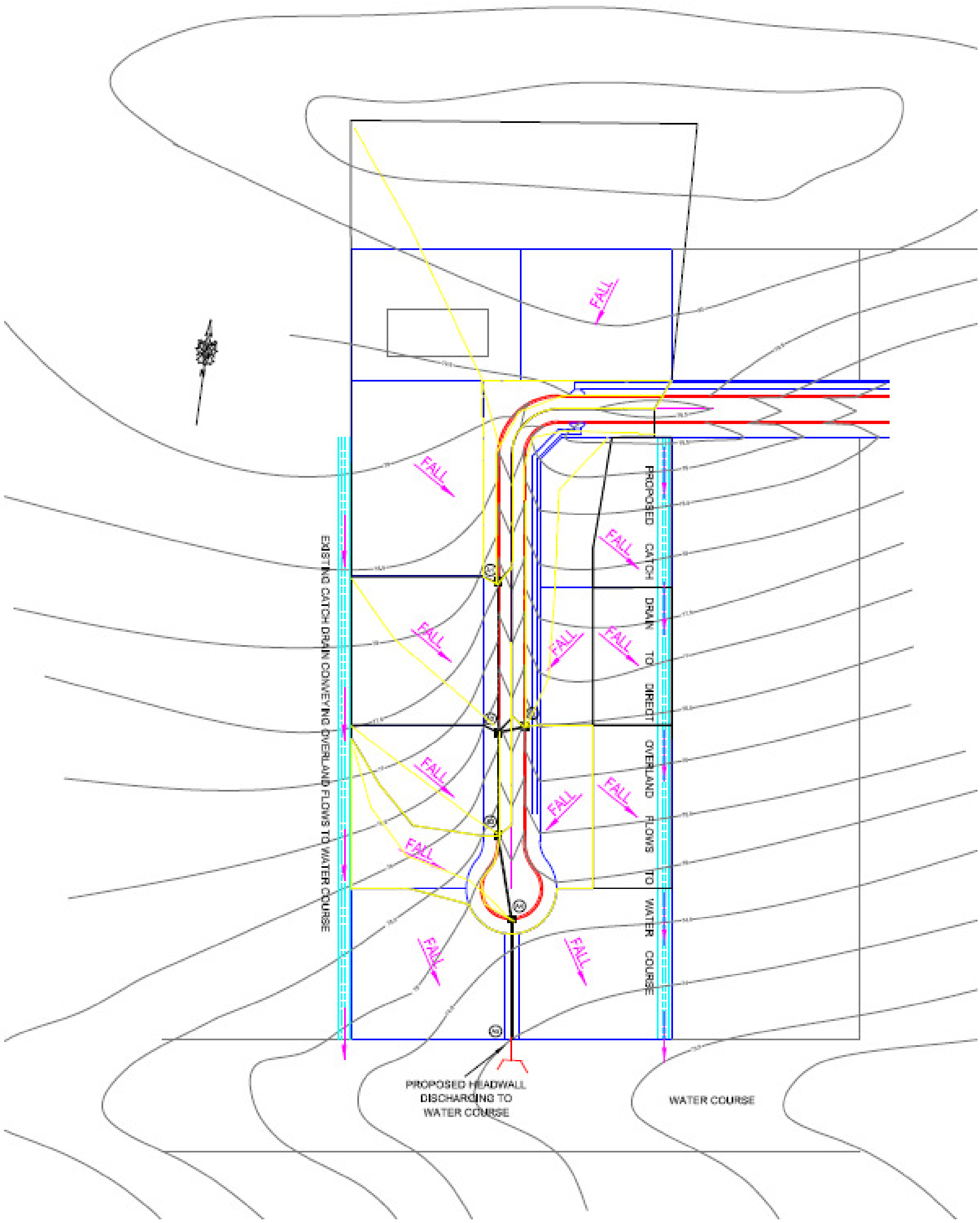


Figure 6.29 - Stormwater drainage system example to serve a 10 lot residential subdivision

The overflow safe limits applied for this example include:

- limiting velocity-depth ratio to 0.4 within the road,
- minor overflows are not to pass through private property,
- major overflows are allowed to pass through private property leaving a sag point provided that:

  - flows are safely conveyed within a dedicated swale that does not exceed 0.3 m flow depth or a velocity-depth ratio limit of 0.3 within the property, and
  - flows follow the general direction of the piped drainage network.
- flow depths are limited to 0.135 m in the 7.2m wide roadway for minor storms (to avoid crossing the crown) and 0.285 m in the major storm (to avoid crossing private property boundaries).

Maximum flow widths in the road have not been considered for this example due to the short length, and non-critical nature of the access way. If a 2.5 m flow width was to be applied on a road with 3% crossfall, the maximum allowable depth in the gutter would be nominally 102 mm.
(2.05 m x 3% grade) + 0.040 m (450 mm gutter depth)

Most design work for subdivisions and new drainage systems is now performed using computer simulation models, however the use of spreadsheets for hydraulic analysis using the rational method may be considered suitable for certain drainage networks such as shown in the example. Reasons why the Rational Method is not always suitable has been discussed in detail within Section XX. Some key reasons for not using the Rational Method include when:

- travel times and/or locations for the minor drainage system is significantly different from that of the major overflows;
- there are rapidly changing hydrographs;
- the drainage network becomes complex in nature, for example where there are channel networks where flow splits are not well defined or relief drainage works incorporate split pipe flows;
- the surface gradient is relatively flat resulting in flat pressurised pipes;
- the catchment contains significant on-site detention systems that require modelling or when overland flow paths pass through detention basin storage structures such as parks or ovals; or
- the drainage system is greatly impacted by flooding, high tailwater levels or HGL’s above the surface level.

Hydrology Calculations by Spreadsheet

For small developments where a designer may choose not to use a dedicated computer simulation model, and where the modelling of water pollutants or detention basins is not required, following is a method of major/minor system design by spreadsheet. The proposed spreadsheet method of pipe design is similar to that presented in ARR87, however due to advancements in computing power and spreadsheets some variations and additions have been applied. Following is a brief description of each sheet:

Hydrological Design Sheet A: defining minor flowrates arriving at pits, pit inlet types and capture ratios, and calculating safe overflows approaching and leaving a pit.

Hydrological Design Sheet B: defining design flowrates for various pipes in a system based on the capture ratio calculated from Sheet A.

Hydrological Design Sheet C: estimating major flowrates at all locations assuming the pipe network is limited to the minor capacity, then calculating safe overflows approaching and leaving a pit.

Hydraulic Design Sheet D: determining trial pipe diameters and invert levels based on design flow rates from Sheet C, minimum pipe grades and covers, pit pressure losses and a downwards HGL calculation.

Hydraulic Checking Sheet E: tracing hydraulic grade lines upwards from a specified tailwater level, determining whether HGLs meet freeboard requirements.

The pipe system shown in Figure 6.29 has been located with the aim of providing adequate drainage with as little pipework as possible. This has involved some trial and error, but typically only the final arrangement is presented here. Design flowrates for pit entrances are calculated in Sheet A, and types of entrance are defined. Due to the nature of the catchments only full-area flows were considered. Partial-area estimates should be considered on flat catchments, or ones with a different style of property drainage. Bypass flows are estimated, and directed to downstream pits with safe condition checks carried out for both the approaching flows and downstream flows leaving a pit due to the potential of changes in longitudinal grades. The accumulative capture ratio has been calculated at each pit to determine the contributing catchment area within the pipeline for later calculations. Since the flowrates are small under minor flow conditions, the times of concentration have not been adjusted.

The Rational Method which is not designed to provide continuity, but to instead provide a method of designing the pipe size leaving the pit. Full-area and partial-area estimates are made in Sheet B then multiplied by the accumulative capture ratio from Sheet A to determine design pipe flows. Equivalent impervious areas are accumulated, working down the pipe system, and these are multiplied by rainfall intensities corresponding to the time of concentration for each pipe. As flows are only admitted to pipes at their upstream ends, the times of concentration to these points are used. (Where flows are admitted through connections along a pipe, the time of travel in the pipe should be included in the time of concentration.)

The definition of catchments is relatively simple for full area flows. The time of concentration of the furthest upstream area is selected, and times of travel along flow paths and pipes are added, while corresponding sub-catchment areas are accumulated. Where two pipe branches meet, the longer of the two times of concentration is selected, to encompass the total combined catchment. The major storm intensities give shorter overland flow times with the kinematic wave equation and as such should be assessed independently of the minor storm times as completed in Sheet C.

Calculating the safe overflows for the major storm can become quite tedious to estimate by hand or spreadsheet, particularly since the minor piped network is likely to be surcharging during a major storm event.

The sub-catchments for each pit in the drainage system are shown in Figure 6.29. Allowance is made for the effect of property boundaries on flow paths. The longest flow paths can be defined and times of concentration calculated using methods presented previously. Gutter flow characteristics are determined from Section 6.3 and pit entry capacities from the HEC-22 pit inlet procedures as discussed in Section 6.1.

Minor Storm - Hydrological Design And Overland Safety Check - Sheet A

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]		[15]	[16]	[17]	[18]	[19]	[20]	[21]	
					[3] /1000 / (π/4 x [4]')	[6]' / 19.6				[9] x [10]		[8] x [12] / 0.36			[13] + [14]							
		Flow Times					Runoff									Approach Flow Rate Safety Check Immediately U/S					Approach Limits for Safe Overland Flow Rates Immediately U/S	
Pit Name	Land Use Type	Flow Length (m)	Slope (m/m)	Mannings "n"	Time tc (min)	Total Time (min)	Intensity I (mm/hr)	Runoff Coeff. C	Area A (ha)	C.A (ha)	ΣC.A (ha)	Q = C.I.A (L/s)	Bypass Flow (L/s) from Pit ( )		Total Approach Flow Rate (L/s)	U/S Overflow Route Profile	U/S Gutter Slope (m/m)	U/S Flow Width (m)	U/S Flow Depth (m)	U/S Flow V.D	Maximum Allowable Flow Rate (L/s)	Limiting Factor
A1	Residential	85	0.05	0.04	12	12.400	88	0.65	0.791	0.514	0.569	139	-	0	139	Std 7.2 m	0.033	2.270	0.094	0.16	421	Depth > 0.135 m
	Road	40	0.033	Gutter	0.4			0.75	0.073	0.055						Wide Road						
																Barrier Kerb						
A2	Residential	55	0.036	0.04	11	11.000	93	0.7	0.168	0.118	0.118	30	A1	60	90	Std 7.2 m	0.035	1.840	0.081	0.13	434	Depth > 0.135 m
																Wide Road						
																Barrier Kerb						
A3	Residential	48	0.052	0.04	10	10.000	96.6	0.7	0.102	0.071	0.071	19	A2	31	51	Std 7.2 m	0.035	1.390	0.068	0.10	434	Depth > 0.135 m
																Wide Road						
																Barrier Kerb						
B1	Residential	40	0.042	0.04	8	8.000	106	0.7	0.198	0.139	0.139	41	-	0	41	Std 7.2 m	0.035	1.240	0.063	0.09	434	Depth > 0.135 m
																Wide Road						
																Barrier Kerb						
A4	Residential	64	0.041	0.04	11	11.000	92.8	0.7	0.205	0.144	0.144	37	A3	11	54	Std 7.2 m	0.033	1.440	0.069	0.10	421	Depth > 0.135 m
													B1	7								
																Wide Road						
																Barrier Kerb						

[22]	[23]	[24]	[25]	[26]		[27]	[28]	[29]	[30]	[31]	[32]	[33]	[34]	[35]	[36]	[37]	[38]	[39]		[40]
						[12] & [30]	Σ[27]	[25] / [15] x [28]	[26] / [15] x [28]	[29] + U/S [31]	[12] + U/S [32]	[31] / [32]								
Pit Inlet						Pit Capture Ratio							Bypass Flow Rate Safety Check Immediately D/S					Bypass Limits for Safe Overland Flow Rates Immediately D/S		Remarks
Inlet Type	Sag Pit Properties Allowable Ponding Depth (m)	Actual Ponding Depth (m)	Inlet Capacity (L/s)	Bypass Flow (L/s) to Pit ( )		Approach C.A (ha)	Total Approach C.A (ha)	Capture C.A (ha)	Bypass C.A (ha)	Total Capture C.A (ha)	ΣC.A (ha)	Captured Ratio (%)	D/S Overflow Profile	D/S Gutter Slope (m/m)	D/S Flow Width (m)	D/S Flow Depth (m)	D/S Flow V.D	Maximum Allowable Flow Rate (L/s)	Limiting Factor	
On-Grade			79	60	to A2	0.569	0.569	0.323	0.246	0.323	0.569	57%	Std 7.2 m	0.035	1.500	0.071	0.11	434	Depth > 0.135 m	Minor overflow is safe
1m Kerb Inlet													Wide Road							
Plus Grate													Barrier Kerb							
						0.246														
On-Grade			59	31	to A3	0.118	0.363	0.237	0.126	0.676	0.825	82%	Std 7.2 m	0.035	1.050	0.058	0.08	434	Depth > 0.135 m	Minor overflow is safe
1m Kerb Inlet													Wide Road							
Plus Grate													Barrier Kerb							
						0.126														
On-Grade			40	11	to A4	0.071	0.198	0.156	0.041	0.832	0.897	93%	Std 7.2 m	0.020	0.630	0.045	0.04	328	Depth > 0.135 m	Minor overflow is safe
1m Kerb Inlet													Wide Road							
Plus Grate													Barrier Kerb							
On-Grade			34	7	to A4	0.139	0.139	0.115	0.023	0.115	0.139	83%	Std 7.2 m	0.020	0.460	0.040	0.03	328	Depth > 0.135 m	Minor overflow is safe
1m Kerb Inlet													Wide Road							
Plus Grate						0.041							Barrier Kerb							
						0.023														
Sag	0.150	0.118	54	0	to A5	0.144	0.208	0.208	0.000	1.040	1.040	100%	Not Allowed	-	-	-	-	0	Overflow Not	Minor overflow is safe
																				*50% blockage factor applied
1m Kerb Inlet					Water														Allowed Through	for sag pit
Plus Grate					Course														Private Property	

Sheet A. Calculation of minor flowrates at pits. (Separate Excel provided)

Hydrological Minor Design Sheet B

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]
							[6] x [7] / 0.36				Sheet 1: Σ[29]	[6] x [7] / 0.36				[10] x [11] / 0.36	Greater of [8] & [12]	Sheet 1: [33]	[13] x [14]	Remarks
			Full Area					Full Area (in pipe)					Partial Area							
Pipe Name	Pipe Length (m)	Est. Pipe Time (min)	Catchment Time tc (min)	Total Time (min)	Intensity I (mm/hr)	ΣC.A (ha)	Q = C.I.A (L/s)	Catchment Time tc (min)	Total Time (min)	Intensity I (mm/hr)	ΣC.A (ha)	Q = C.I.A (L/s)	Total Time (min)	Intensity I (mm/hr)	ΣC.A (ha)	Q = C.I.A (L/s)	Adopted Flow Rate (L/s)	Ratio in Pipe (%)	Q pipe (L/s)	
A1-A2			12.4	12.4	88.0	0.569	139	12.4	12.4	88.0	0.323	79	5.0	126.2	0.263	92	139	57%	79	0.32 ha estimated to contribute in partial area time of 5 minutes, C=0.65 + Road C.A of 0.055
	40	0.4																		
B1-A2			8.0	8.0	106.0	0.139	41	8.0	8.0	106.0	0.115	34	5.0	126.2	0.105	37	41	83%	34	0.15 ha estimated to contribute in partial area time of 5 minutes, C=0.7
	7.5	0.1																		
					+	0.569				+	0.323			+	0.263					
					+	0.118				+	0.115			+	0.077					
A2-A3			11.0	12.8	86.7	0.826	199	11.0	12.8	86.7	0.676	163	5.1	125.3	0.445	155	199	82%	163	0.11 ha estimated to contribute in partial area time of 5.1 minutes, C=0.7
	27	0.3																		
					+	0.071				+	0.071			+	0.063					
A3-A4			10.0	13.1	85.8	0.897	214	10.0	13.1	85.8	0.832	198	5.4	122.8	0.508	173	214	93%	198	0.09 ha estimated to contribute in partial area time of 5.4 minutes, C=0.7
	23	0.3																		
					+	0.144				+	0.144			+	0.091					
A4-A5			11.0	13.4	78.9	1.041	228	11.0	13.4	78.9	1.040	228	5.7	120.5	0.599	200	228	100%	228	0.13 ha estimated to contribute in partial area time of 5.7 minutes, C=0.7

Sheet B. Calculation of minor flows for pipes. (Separate Excel provided)

Major Storm - Hydrological Design And Overland Safety Check - Sheet C

[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]		[15]	[16]	[17]	[18]	[19]	[20]	[21]	
					[3] /1000 / (π/4 x [4]³)	[6]³ / 19.6		Minor x 1.2/0.95		[9] x [10]		[8] x [12] / 0.36			[13] + Σ[14]							
		Flow Times					Runoff									Approach Flow Rate Safety Check Immediately U/S					Approach Limits for Safe Overland Flow Rates Immediately U/S	
Pit Name	Land Use Type	Flow Length (m)	Slope (m/m)	Mannings "n"	Time tc (min)	Total Time (min)	Intensity I (mm/hr)	Runoff Coeff. C	Area A (ha)	C.A (ha)	ΣC.A (ha)	Q = C.I.A (L/s)	Bypass Flow (L/s) from Pit ( )		Total Approach Flow Rate (L/s)	U/S Overflow Route Profile	U/S Gutter Slope (m/m)	U/S Flow Width (m)	U/S Flow Depth (m)	U/S Flow V.D	Maximum Allowable Flow Rate (L/s)	Limiting Factor
A1	Residential	85	0.05	0.04	9	9.400	172.9	0.82	0.791	0.649	0.719	345	-	0	345	Std 7.2 m	0.033	3.360	0.127	0.25	844	V.D > 0.4
	Road	40	0.033	Gutter	0.4			0.95	0.073	0.069						Wide Road		Flows still lower then				Not by depth > 0.285 m
																Barrier Kerb		crown (0.135 m)				
A2	Residential	55	0.036	0.04	8	8.000	184.9	0.95	0.168	0.159	0.159	82	A1	266	348	Std 7.2 m	0.035	3.320	0.125	0.26	830	V.D > 0.4
																Wide Road		Flows still lower then				Not by depth > 0.285 m
																Barrier Kerb		crown (0.135 m)				
A3	Residential	48	0.052	0.04	7	7.000	194.9	0.95	0.102	0.097	0.097	52	A2	289	341	Std 7.2 m	0.035	3.300	0.124	0.25	830	V.D > 0.4
																Wide Road		Flows still lower then				
																Barrier Kerb		crown (0.135 m)				
B1	Residential	40	0.042	0.04	6	6.000	206.4	0.95	0.198	0.188	0.188	108	-	0	108	Std 7.2 m	0.035	1.990	0.086	0.14	830	V.D > 0.4
																Wide Road						
																Barrier Kerb						
A4	Residential	64	0.041	0.04	8	8.000	184.9	0.95	0.205	0.194	0.194	100	A3	301	474	Std 7.2 m	0.033	3.810	0.140	0.30	844	V.D > 0.4
													B1	74		Wide Road						
																Barrier Kerb						
																		Flows approaching cul-de-sac from two directions				

[22]	[23]	[24]	[25]	[26]		[27]	[28]	[29]	[30]	[31]	[32]		[33]
Pit Inlet						Bypass Flow Rate Safety Check Immediately D/S					Bypass Limits for Safe Overland Flow Rates Immediately D/S		Remarks
Inlet Type	Minor Sag Pit Properties Allowable Ponding Depth (m)	Actual Ponding Depth (m)	Minor Inlet Capacity (L/s)	Bypass Flow (L/s) to Pit ( )		D/S Overflow Profile	D/S Gutter Slope (m/m)	D/S Flow Width (m)	D/S Flow Depth (m)	D/S Flow V.D	Maximum Allowable Flow Rate (L/s)	Limiting Factor	
On-Grade			79	266	to A2	Std 7.2 m	0.035	2.960	0.115	0.22	830	V.D > 0.4	
1m Kerb Inlet						Wide Road						Not by depth > 0.285 m	Major overflow is limited here by V.D > 0.4 due to the grade of the road
Plus Grate						Barrier Kerb							
													Overflows were still lower then crown level (0.135 m), therefore no splitting of overflows to other pits
													Major overflow is safe
On-Grade			59	289	to A3	Std 7.2 m	0.035	3.080	0.118	0.23	830	V.D > 0.4	Major overflow is safe
1m Kerb Inlet						Wide Road							
Plus Grate						Barrier Kerb							
On-Grade			40	301	to A4	Std 7.2 m	0.020				984	V.D > 0.4	Major overflow is safe
1m Kerb Inlet						Wide Road						Not by depth > 0.285 m	
Plus Grate						Barrier Kerb							
On-Grade			34	74	to A4	Std 7.2 m	0.020				984	V.D > 0.4	Major overflow is safe
1m Kerb Inlet						Wide Road							
Plus Grate						Barrier Kerb							
Sag	0.150	0.120	54	420	to A5	Trapazoidal	0.020	4.50	0.125	0.11	1320	V.D > 0.3	
1m Kerb Inlet					Water	Channel							Overflows approaching from two directions (Catchment A4+A3 and B1), both leading to the sag pit, therefore no splitting over crown required.
Plus Grate					Course	2m base							
						1 in 6 batter							
													Major overflow is safe

Sheet C. Major flow calculations. (Separate Excel provided)

Table 6.3 Comments on hydrological design sheets.

SHEET A (MINOR PIT FLOWS)

This sheet is used for calculation of minor flowrates draining to pits from each sub-catchment, and for determination of required pit entry capacities. Columns are numbered and values in them are designated by the column number in square brackets, eg Column 12 is referred to as [12].

Column 1 – pit identification; various nomenclatures can be used. (A note can be made as to whether calculations are for the full area of the catchment or for a partial area.)

Column 2 – land-use types present in the sub-area. Each should be set out on a separate line.

Columns 3 to 7 are used to calculate times of flow for different segments of the longest flow path. Usually these can be placed beside the related land-use type given in [2], but it may be necessary to put information on different lines when different flow segments (say with different slopes) occur on the same land-use type, or when two modes of flow occur on the same area of land use (for instance, sheet flows collected in a catch drain). When flows from part of the catchment enter pipes directly through underground connections, the pipe travel time must be included; this will be short and can be approximated.

Column 3 – length of flow path (*m*).

Column 4 – slope of flow path (*m/m*).

Column 5 – estimated surface roughness*, "n"*.

Column 6 – time of travel (*minutes*), calculated from the formulae and aids given in Section X

Column 7 – total flow time (*minutes*), the sum of the relevant values from [6].

Column 8 – the design rainfall intensity (*mm/h*) corresponding to time [7].

Column 9 – the runoff coefficient for each land-use present.

Column 10 – the area of each land-use type (*ha*). In partial-area calculations, this is the area draining in the time required for the directly connected impervious area to contribute fully to flows.

Column 11 – the equivalent impervious area for each land-use type present, *C.A* (*ha*) obtained by multiplying [9] by [10].

Column 12 – the total equivalent impervious area for the sub-catchment Σ *C.A* (*ha*), the sum of the values for the contributing land-uses in [11].

Column 13 – the calculated flowrate from the Rational Method formula (*L/s*), [8] x [12] / 0.36. (If flowrates are being calculated in m^3/s, [8] x [12] is divided by 360.)

Column 14 – any bypass flowrates from upstream (*L/s*), with the identification for the pit of origin being noted in curly brackets {..}.

Column 15 – the adopted flowrate (*L/s*), being the greater of any applicable full-area and partial-area estimates in [13] together with any bypass flows in [14].

Columns 16 to 21 are used to calculate flow conditions approaching the pit inlet and have been calculated using procedures from Section 6.4 (Overland Flow).

Column 16 – description of upstream overflow route profile

Column 17 – gutter slope at the approach section to the pit (*m/m*).

Column 18 – flow width at the pit (*m*), depending on gutter geometry and longitudinal slope [17] and the flowrate [15]. Flow depth (*mm*) may also be noted. Where flow approaches a sag pit from two directions, the flow has to be reduced accordingly.

Column 19 – flow depth at the pit (*m*), depending on gutter geometry and longitudinal slope [17] and the flowrate [15].

Column 20 – velocity depth ration check.

Column 21 – maximum flow limits for safe overland flow conditions approaching the pit, with a note on what condition led to the limiting factor. If [15] is less than [21] then the approach flowrate can be considered safe.

Column 22 – a description of the pit inlet type and dimensions.

Column 23 – maximum allowable pond depth if [22] is a sag pit.

Column 24 – resulting pond depth based on approach flow of [15] if [22] is a sag pit, calculated from relationships of the type discussed in Section 6.3.

Column 25 – inlet flowrate (*L/s*) for pit type [22] and flowrate [15], calculated from relationships of the type discussed in Section 6.3.

Column 26 – bypass flow (*L/s*), if any, together with identification of destination pit, in brackets {..}.

Columns 27 to 33 are used to calculate pit capture ratios that are required for pipe flow calculations in Sheet C.

Column 27 – the equivalent approach areas for the sub-catchments *C.A* (*ha*) (as calculated in [12]) including equivalent upstream bypass sub-catchment areas [30].

Column 28 – the total equivalent approach area for the sub-catchment approaching the pit Σ *C.A* (*ha*) Σ [27].

Column 29 – the equivalent approach area captured by the pit [25] / [15] x [28].

Column 30 – the equivalent bypass area not captured by the pit [26] / [15] x [28].

Column 31 – the total equivalent captured area within the pipe network [29], plus upstream captured areas Σ U/S [31].

Column 32 – the total equivalent sub-catchment area [12], plus total of upstream sub-catchment areas Σ U/S [32].

Column 33 – the capture ratio calculated by dividing [32] from [31].

Columns 34 to 39 are used to calculate flow conditions leaving the pit inlet and have been calculated using procedures from Section 6.4 (Overland Flow).

Column 34 – description of downstream overflow route profile

Column 35 – gutter slope at the downstream section to the pit (*m/m*).

Column 36 – flow width at the pit (*m*), depending on gutter geometry and longitudinal slope [35] and the bypass flowrate [26]. Flow depth (*mm*) may also be noted. Where flow approaches a sag pit from two directions, the flow has to be reduced accordingly.

Column 37 – flow depth at the pit (*m*), depending on gutter geometry and longitudinal slope [35] and the bypass flowrate [26].

Column 38 – velocity depth ration calculation.

Column 39 – maximum flow limits for safe overland flow conditions approaching the pit, with a note on what condition led to the limiting factor. If [26] is less than [39] then the approach flowrate can be considered safe.

Column 40 – comments on unusual points or cross-references to other sheets.

SHEET B (PIPE FLOWRATES)

This sheet enables full- and part-area flowrates to be estimated and compared. The larger is adopted. Equivalent impervious areas and travel times are accumulated along the pipe system, and flowrates are calculated for each pipe by the Rational Method formula multiplied by the capture ratio from Sheet A [33].

Column 1 – pipe identification; various nomenclatures can be employed.

Column 2 – pipe length *(m).*

Column 3 – estimated pipe time *(min).*

Column 4 – sub-catchment time of concentration (*minutes*). (As per Sheet A [7].

Column 5 – full-area time of travel (*minutes*), derived from times calculated for pit entry flowrates [4], and accumulated travel times in pipes [3]. Full-area calculations always use the longest travel time; this may lead to some anomalies when a sub-area with a longer travel time is encountered.

Column 6 – rainfall intensity (*mm/h*) corresponding to time [5].

Column 7 – accumulated values of equivalent impervious areas, Σ *C.A* (*ha*) for sub-catchments draining to each pipe.

Column 8 – full-area flowrate (*L/s*), calculated from [6] x [7] / 0.36.

Column 9 – partial area time of travel (*minutes*). To derive this, a suitable top sub-catchment is chosen, its partial-area time selected (typically starting with 5 minutes), and pipe travel times are added to this, as calculations proceed down the line.

Column 10 – rainfall intensity (*mm/h*) corresponding to time [9].

Column 11 – accumulated equivalent impervious areas for partial area sub-catchments.

Column 12 – partial-area flowrate (*L/s*), calculated as [10] x [11] / 0.36.

Column 13 – adopted total flowrate (*L/s*), the greater of [8] and [12].

Column 14 – total pipe capture ratio from Sheet A [33].

Column 15 – adopted pipe design flowrate (*L/s*), [13] multiplied by [14].

Column 16 – comments or cross-references.

SHEET C (MAJOR SYSTEM FLOWS)

The Rational Method can be employed to estimate total major event flowrates along the drainage system. The procedure selected here is a simplified method that assumes that the piped drainage network is at full capacity during a minor storm, resulting in the total overflow being the difference between the total major flow and the minor pit inlet capacity. Other methods have also been discussed below this table.

All sections of this sheet are similar to those presented for the minor storm in Sheet A except that the minor pit inlet capacities have been retained resulting in increased overflow and bypass in the major storm, and any design and safety checks are carried out with respect to the major storm limits.

More judgement is required for partial-areas. An upstream area must be selected and a partial-area time determined (5 minutes in this case). Pipe flow times are then added to this base time, and partial areas contributing flows to the pipe system within the total time are accumulated. For example, at Pipe A2-A3 it is assumed that 0.11 ha of the 0.168 ha of the equivalent area draining to Pit A2 will contribute in 5.1 minutes out of the full concentration time of 11 minutes. The contributing area consist of the road pavement and footpath area, the roof and impervious areas of the property, along with any pervious areas that would drain to Pit A2 within the 5.1 minutes.

The greater of the two flow estimates is selected. In this case, the full-area estimates are greater. Depending on the characteristics of the catchment (such as if there are large pervious areas at the top of the catchment), partial-area estimates may become dominant as the calculations proceed down the system due to the influence of upper catchments becoming less important.

Calculations for the major system are given in Sheet C. In ARR87 these were only undertaken as hand calculations in the form of checks at critical points. Due to the proficiency in using spreadsheets, these checks are quickly carried out at all locations. The Rational Method is used to calculate flowrates at each point, for appropriate times of concentration. An adjustment of (1.2/0.95) is made to equivalent impervious areas to allow for the 0.5 EY (20% AEP) and 1% AEP runoff coefficient factors.

There are a number of methods to performing these checks. The first is to assume that the roadway takes all of the major flow. If it can do this satisfactorily, no calculations concerning pipe system capacity need be performed. Where the roadway capacity is inadequate, an estimate can be made of the flowrate that the pipe system can carry under major flow conditions. Keep in mind that full HGL analysis is required for the major storm based on the assumed pit and pipe capacities to ensure compliance with pit freeboard requirements and that the resulting major overflows have not been underestimated. If HGL analysis later reveals that assumptions were overestimated for pit inlet capacities then revision is required followed by reassessment of the overflow safety checks. Although this method may provide the most economic sizing of the system, it can be quite tedious and is better suited to computer simulation modelling software.

A simplified approach, as demonstrated in Sheet C, is to assume that the piped drainage network is at full capacity during a minor storm, resulting in the total overflow being the difference between the total major flow and the minor pit inlet capacity. In reality there is likely to be some capacity available in parts of the piped network during a major storm. This approach is only suitable provided that overflow paths mirror the alignment of the piped drainage network, otherwise computer simulation modelling methods are recommended.

Hydraulic Calculations

A flow situation is visualised by grade lines as previously shown in Figure 6.2 (shown earlier in Figure 6.1 Section 6.2 (pipes)). Phenomena such as pressure recovery (Pit 3) and part-full flows in steep pipe links (Reach 4) are shown. The system is surcharged (or pressurised), but there are no overflows. Positions of grade lines are defined by flowrates and energy losses throughout the system, and the backwater influence from the receiving waters.

This is the manner in which a pipe system typically behaves under design conditions. For lesser flows, the grade lines are lower and most pipes flow part-full. For higher flowrates, HGLs may rise to the surface and overflows may occur.

In designing a new system, it is necessary to fix pipe sizes and locations by setting invert levels at each end of the pipe. When The Rational Method approach assumes steady flow in each pipe at the peak flowrate. Target freeboard levels are set at each pit, and pipes are sized and positioned so that water levels do not rise above these.

The aim is to provide sufficient capacity to carry flows of a given design AEP, corresponding to a “convenience requirement”, while preventing unsafe overflow. Additional requirements are adequate cover on pipes to cushion surface loadings, and pipe slopes sufficient to prevent siltation and blockage. Deep pipes must have adequate structural strength, in the form of pipe material and wall thickness, combined with suitable bedding, to withstand earth loads.

Calculations can be performed from the top of a system down to the receiving waters, or in reverse. Both methods have their particular advantages. With both it may be necessary to backtrack and alter previous calculations. The example given works from the top in Sheet D, and then Sheet E makes a backwards pass through the system as a check.

Example of Hydraulic Design Calculations for Pipe Systems

A set of design calculations is presented in Table 3.4 for the system shown previously in Figure 6.29. The flowrates used are taken from the Rational Method calculations in Sheet B. Pipe friction is calculated by the Manning Equation (with wall roughness n = 0.013) and pressure change coefficients for pits are determined from the Missouri Charts and Hare (1983) as discussed in Section 6.3.4.

A minimum pipe diameter of 375 mm is selected. Pipe wall thicknesses range from 34 mm for 375 mm internal diameter pipes to 44 mm for 600 mm pipes. The calculations make allowance for cover depths (taken as 0.6 m), and for alignment of pipe inverts. A fall of 0.03 m across each pit is specified to prevent sediment accumulation. Required freeboard levels in pits are set 0.15 m below surface gutter levels.

Figure 6.30 shows a pipe reach on which features are identified and linked to the various columns of the calculation sheet in Table 6.4. Columns [1] to [7] present the basic design information, while columns [8] to [17] are for calculation of the hydraulic grade line position. The remaining columns are used to determine pipe invert levels, allowing for hydraulic considerations, cover and positions of upstream pipes. Pipe slopes are calculated to check for sedimentation problems. More detailed comments on the procedure are given in Table 6.4.

In this example, minimum cover depths are set in advance and pipes are positioned to allow for these. An alternative procedure is to vary the class of pipe, and hence the allowable cover, and aim to keep pipes reasonably shallow. This may be advantageous in rocky or difficult ground.

As a check the procedure shown in Sheet D can be reversed as demonstrated in Sheet E, with hydraulic grade lines being projected upwards from receiving waters, and allowance being made for pressure changes as each pit is encountered. Following this, HGLs can be plotted graphically.

For estimating water levels in pits, kw factors (equation (6.7 – referenced in Section 6.3.4 (pits))) should be used where available, while ku factors (equation (6.6)) must be employed when tracing an HGL along a main line of pipe reaches. Where special factors (kL) are available for lateral or side lines, these should be used to trace HGLs. These factors may differ from those originally assumed in design calculations, as more information is now available about pit submergence ratios, which can influence coefficients. kw factors may be significantly larger than ku values. Thus the situation may arise where the pit water level rises above ground level even though the HGL is still below the required freeboard. This may be acceptable where there is no risk of local flooding.

Where an HGL projected upwards from a pit intersects a steep upstream pipe, a hydraulic jump occurs in the vicinity of the intersection point, and flow upstream of this is part-full. Nevertheless, surcharging may occur in the upstream pit.
Pipe systems can be separated into a main-line which dominates the flow situation (usually the longest and deepest line) and side-lines. The design procedure assumes that downstream water levels can be controlled. For minor systems, the tailwater level for a pipe side-line can be taken as the higher of (a) the main-line HGL in the junction pit, and (b) the obvert level of side-line at the point where it enters the main-line. The same selection can be made where the terminal reach in a pipe network joins a trunk drain or discharges directly into receiving waters.

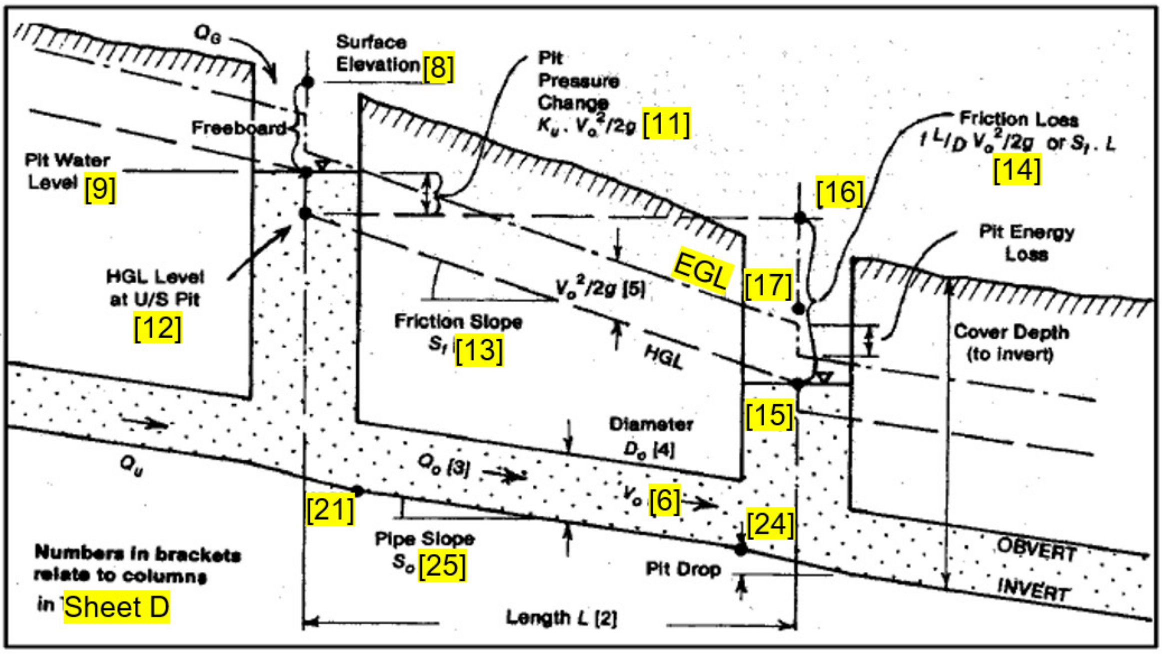


Figure 6.30. Pipe reach showing features identified in calculations.

			Cover Depth [m]	0.6	[C]																						
			Manhole Drop [mm]	30	[D]																						
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]		[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]		[18]	[19]	[20]	[21]	[22]	[23]	[24]	[25]	[26]
		Sheet 2: [15]			[3] / 1000 / (π/4 x [4]	[6]^2 / 19.6		Min of ([8]-[D]) or matching U/S [17]			[7] x [10]	[9] - [11]	(([6]x(A)/([4]/4)^(2	[2] x [13]	[12] - [14]		Min of [15],[16] or Tail Water Level		[12] - [4]	[8] - [5] - [4] - [C	U/S [24] - ([D]/1000)	Min of [18],[19],[2	[17] - [4]	[16] - [5] - [4] - [C]	Min of [22],[2	([21] - [24]) / [2]	
																			U/S Invert Levels [m]				D/S Invert Levels [m]				
Pipe Name	Length [m]	Design Minor Flowrate (Q) [L/s]	Trial Pipe Diameter [m]	Pipe Thickness [mm]	Full Pipe Velocity [m/s]	Velocity Head [m]	U/S Surface Level [m]	Max U/S Pit Water Level [m]		Pit Pressure Coeff. [ku]	Pit Pressure Losses [m]	HGL at U/S Pit [m]	HGL Slope [m/m]	Pipe Friction Loss [m]	HGL at D/S Pit [m]	D/S Pit Surface Level [m]	D/S Pit Water Surface Level [m]		Hydraulic	Cover	U/S Pipe	Adopted Invert	Hydraulic	Cover	Adopted Invert	Pipe Slope [m/m]	Remarks
pA1	40	79	0.375	0.034	0.715	0.026	77.900	A1	77.150	4.4	0.115	77.035	0.00203021	0.081	76.954	75.900	A2	75.750	76.660	76.291		76.291	75.375	74.891	74.891	0.0350	
pA2	27.4	163	0.375	0.034	1.476	0.111	75.900	A2	75.750	0.7	0.078	75.672	0.008642951	0.237	75.435	74.950	A3	74.800	75.297	74.891	74.861	74.861	74.425	73.941	73.941	0.0336	
pA3	22.6	198	0.375	0.034	1.793	0.164	74.950	A3	74.800	1.28	0.210	74.590	0.012759149	0.288	74.302	74.600	A4	74.302	74.215	73.941	73.911	73.911	73.927	73.591	73.591	0.0142	
pA3	22.6	198	0.45	0.036	1.245	0.079	74.950	A3	74.800	1.15	0.091	74.709	0.004829	0.109	74.600	74.600	A4	74.450	74.259	73.864	73.911	73.864	74.000	73.514	73.514	0.0155	
																									73.591	0.0142	450 mm adopted
pA4	51.4	228	0.45	0.036	1.434	0.105	74.600	A4	74.302	1.11	0.116	74.186	0.003915296	0.201	73.985	74.000	A5	73.850	73.736	73.514	73.561	73.514	73.400	72.914	72.914	0.0117	
	51.4	228	0.525	0.041	1.053	0.057	74.600	A4	74.302	1.38	0.078	74.224	0.002010652	0.088	74.136	74.000	A5	73.850	73.699	73.434	73.561	73.434	73.325	72.834	72.834	0.0117	
																									72.914	0.0117	525 mm adopted
pB1	7.6	34	0.375	0.034	0.308	0.005	75.950	B1	75.800	5	0.024	75.776	0.000376049	0.003	75.773	75.900	A2	75.750	75.401	74.941		74.941	75.375	74.891	74.891	0.0066	

Sheet D Example of hydraulic calculations for pipe design. (Separate Excel provided)

Table 6.4 Comments on columns in hydraulic calculation sheet.

Sheet D is used to perform HGL calculations from the top of a system down to the receiving waters. As a check the procedure shown in Sheet D is reversed in Sheet E, with hydraulic grade lines being projected upwards from receiving waters. The column descriptions and general formulae are relevant for both sheets.

SHEET D (DOWNWARDS)

Columns in Sheet D are designated by number, eg Column 2 as [2]. Their contents are:

Column 1 – the pipe link or reach. Various nomenclatures can be used.

Column 2 – pipe length (*m*).

Column 3 – design flowrate (in *L/s* or perhaps m^3/s) derived from hydrological calculations such as the Rational Method or a computer model.

Column 4 – a trial pipe diameter *(m)*. This may be changed if calculations indicate that energy losses are too high, and the pipe is too deep, or where there is interference with other services.

Column 5 – the pipe wall thickness *(mm)* as provided from manufacturers specifications.

Column 6 – the velocity obtained by dividing the flowrate in [3] by the area of a pipe with the diameter given in [4]. Consistent units must be used to give a velocity in *m/s* units

Column 7 – $V^2/2g$ calculated with the V from [6].

Column 8 – upstream pit surface level (*m* AHD [Australian Height Datum]).

Column 9 – allowable water level in the upstream pit (*m* AHD). For the first pit in a line, it is the surface level [7] minus a freeboard such as 0.15 *m*. For other pits it is the lower of this value and the levels set in [15] during calculations for the pipe reach(s) immediately upstream.

Column 10 – a pressure change or pit water revel coefficient (as defined in equations (6.6) or (6.7) – referenced in Section 6.3.4 (pits))), appropriate to the pit geometry, relative pipe sizes and flowrates. Refer to Section 6.3.4. Subsidiary calculation sheets or notes may be needed.

Column 11 – the pressure change for the pit (*m*), calculated as [10] x [7].

Column 12 – assumed HGL position for flows leaving the pit, obtained by subtracting [11] from [9], (*m* AHD).

Column 13 – HGL slope (*m/m*), calculated from equations or charts for the given flowrate, pipe size and roughness.

Column 14 – energy loss due to pipe friction (*m*) as [13] x [2]. The HGL drops by this amount over the length of the pipe.

Column 15 – the HGL level at the downstream pit (*m* AHD), as [12] – [14].

Column 16 – surface level of the downstream pit (*m* AHD).

Column 17 – allowable water surface level, the lower of (a) the level found by subtracting a freeboard from [16], (*m* AHD) and (b) the HGL level calculated in [15], or for a known tailwater level. This should be adopted as the level in Column [9] of subsequent calculations for the pipe downstream of this pit.

Column 18 – a pipe invert level for the upstream pit based on hydraulic requirements (*m* AHD), found by subtracting [4] from [12].

Column 19 – an upstream pit invert level (*m* AHD) obtained by subtracting a cover depth (depth from surface to pipe crown + pipe thickness + diameter) from the surface level in [7].

Column 20 – an upstream pit invert level (*m* AHD), equal to the invert level of the lowest upstream pipe entering the pit, minus any allowance for a slope or drop across the pit.

Column 21 – adopted upstream pit invert level (*m* AHD), being the lowest of [18], [19] and [20].

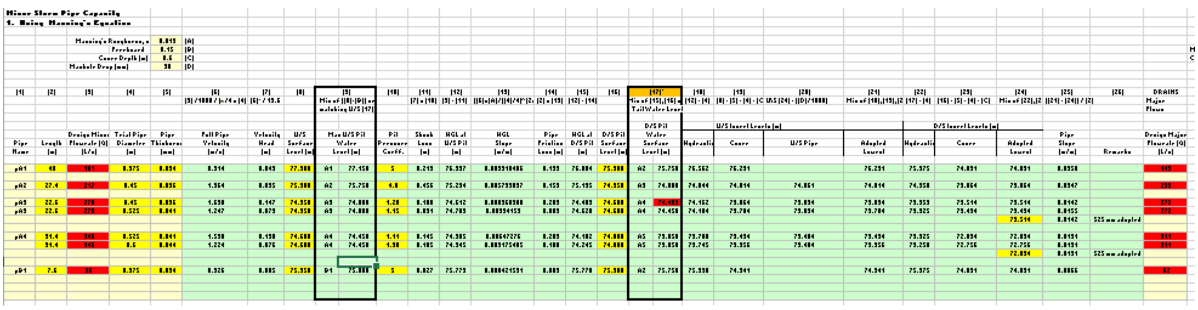
Column 22 – a downstream pit invert level based on hydraulic considerations (*m* AHD), obtained by subtracting diameter [4] from level [17].

Column 23 – a downstream invert level (*m* AHD) based on cover considerations, obtained by subtracting cover depth and wall thickness [5] from the surface level [16].

Column 24 – adopted downstream pit level (*m* AHD), the lower of [22] and [23].

Column 25 – pipe slope (*m/m*) calculated from adopted invert levels, as ([21] – [24]) / [2]. This must be checked and pipe inverts or sizes altered if it is unacceptable.

Column 26 – remarks on any unusual features of design, or cross-references to notes or other design sheets.



Sheet E. Calculations for backwards check through pipe system. (Separate Excel provided)

6.6 References

Balkham, M., Fosbeary, C., Kitchen, A. and Rickard, C. (2010) *Culvert design and operation guide*, CIRIA C689, London

Laurenson, E.M., Mein, R.G. and Nathan (2010) *RORB - Version 6 Runoff Routing Program, User Manual*, Monash University Department of Civil Engineering and Sinclair Knight Merz

Normann, J.M., Houghtalen, R.J. and Johnston, W.J. (2005) *Hydraulic Design of Highway Culverts*, Hydraulic Design Series HDS 5, 2nd Edition, Federal Highway Administration, Washington, DC

Chapter 7
Urban Network Analysis

Chapter Status	
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Chapter	7
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Examples	In preparation
General	