Water Sensitive Urban Design Impediments and Potential: Contributions to the Urban Water Blueprint (Phase 1)

Task 3: The Potential Role of WSUD in Urban Service Provision

Myers, B, Pezzaniti, D, Kemp, D, Chavoshi, S, Montazeri, M, Sharma, A,
Chacko, P, Hewa, GA, Tjandraatmadja, G, and Cook, S

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Enquires should be addressed to: Goyder Institute for Water Research
Level 1, Torrens Building
220 Victoria Square, Adelaide, SA, 5000
tel: 08-8303 8952
e-mail: enquiries@goyderinstitute.org

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Executive Summary

The project *Water Sensitive Urban Design Impediments and Potential: Contributions to the SA Urban Water Blueprint,* funded by the Goyder Institute, aimed to identify the factors impeding Water Sensitive Urban Design (WSUD) uptake in South Australia. This report represents the findings of Task 3 of this project, which examined the potential contribution of WSUD to flow management in urban South Australia with a particular focus on the minor drainage system. The findings of this report were based on six studies including:

1. The development of a matrix of WSUD solutions suitable for different development scales.
2. Development of a methodology to examine the impact of WSUD approaches on runoff characteristics in urban catchments.
3. The application of this methodology to examine the impact of existing WSUD at case study sites in Burnside, SA (B-Pods) and Mile End, SA (Rain gardens)
4. The application of this methodology to examine the impact of generic retention and detention based WSUD approaches on flow characteristics of larger infill and greenfield catchments.
5. The application of the 'SUSTAIN' optimisation tool for the selection of appropriate WSUD strategies for infill catchments and to develop an understanding of SUSTAIN's benefits and limitations for broader use by the profession.
6. Preliminary assessment of the 'MUSIC' model for estimating flows when examining WSUD strategies for stormwater management plans from developed urban catchments in South Australia.

Based on a review of WSUD guidelines from across Australia in Task 3, a matrix of WSUD solutions was produced with respect to development scale. The matrix indicated there is much agreement on the applicability of constructed WSUD tools with respect to development scale. This information was used in developing WSUD scenarios for case studies of urban infill and greenfield developments later in this task.

A review of existing studies was undertaken to explore methods which have been used to indicate the success or otherwise of WSUD to manage peak flow rates, flooding and runoff volumes in urban catchments. Existing studies generally applied design storm techniques when considering flow volume and peak flow rates. Continuous simulation studies tended to be short term. Those which were not short term only considered runoff volume, not peak flow rates. A methodology was therefore developed for this study based on continuous simulation and partial series analysis of catchment flow rates. The methodology was considered suitable for predicting the impact of WSUD on the average recurrence interval of peak flows and flooding volumes at any point in a catchment.

The methodology was first applied to explore the effectiveness of City of Burnside B-Pods and City of West Torrens rain gardens in small urban catchments (2.3 Ha and 3.4 Ha respectively). The results indicated that B-Pods had a minor impact on peak flow rates and runoff volumes in the design conditions. Rain gardens were shown to have reduced the peak flow rate of runoff from the catchment area, but had little impact on runoff volume due to the practical limitations on storage size and the presence of impermeable liners preventing infiltration.

In consultation with local governments and the Goyder Institute WSUD project reference group, concerns were identified regarding the impact of infill development (sub-divisions) on peak flows and associated 'minor' system flooding in urban areas of Adelaide (the 'minor' system represents the bulk of infrastructure investment in urban stormwater drainage). The concern has increased with the greater emphasis on infill developments proposed in the *30 Year Plan for Greater Adelaide*. In response, the impact of WSUD tools in the form of retention (e.g. rainwater tanks, on-site infiltration storage) and detention (e.g. on site detention tanks) installed in conjunction with infill development (sub-divisions) was explored using calibrated catchment models of two catchments: the 44.7 Ha Frederick Street catchment in Glengowrie, SA and the 76 Ha Paddocks catchment in Para Hills, SA. The study findings indicated that retention and detention on the new homes of subdivided allotments, with typical impervious area connections (100 m^2 roof), contributed to but could not restore the pre-infill development flow regime of a catchment. However, ensuring all existing and new properties had 100 m^2 of roof area connected to a detention/retention measure did restore the flow regime to pre-infill development levels. There was little difference between the peak flow and flood reduction benefits achieved by on-site retention and detention storages, however it should be noted that retention systems may be considered to offer additional benefits based on their ability to reduce flow volume. Street scale rain gardens, which were assumed connected to all upstream impervious area, were effective at restoring the flow regime to a limited extent of infill development, but their effectiveness was restricted by their storage capacity. It is recommended that further research is undertaken to examine the cost of implementing on-site retention, on-site detention and street scale rain gardens to determine which option presents a least cost option to achieve flow regime benefits. Further research should also be undertaken to ensure that these results apply to larger catchment areas (e.g. greater than 100 Ha). It is also recommended that options to maximise the connectivity of new impervious areas to on-site retention and detention are explored. Opportunities also exist to improve street scale rain garden design specifically for flow management benefits. It is also worth noting that the current study was based on a simulation of the impact of infill on runoff flow rates. This modelling should be validated with observed flow data. Suitable data may be available for the Frederick Street catchment in 2013/2014.

Local government also expressed a concern about the impact of greenfield development on remnant natural streams in urban areas. This study explored the contribution of WSUD to maintaining the pre-development flow regime in a creek channel exposed to a 16 Ha catchment of greenfield development in Flagstaff Hill, SA. Development was shown to cause changes to the pre-development flow regime in the creek with increases in the total annual flow and peak flow rates, and alterations in the flow duration curve. The inclusion of retention or detention tanks on all homes constructed was ineffective at maintaining the pre-development flow regime in the creek. Street scale bioretention systems were also unable to restore pre-development flows. This indicates that the potential for single on-site or street scale WSUD measures to maintain the pre-development flow regime of a greenfield catchment was limited.

The application of optimisation tools to explore WSUD alternatives in a catchment was explored using the USEPA SUSTAIN optimisation software. Five subcatchments of the Paddocks catchment in

Para Hills were selected for the case study. The tool was used to identify the most cost effective arrangement of on-site retention or detention to maintain pre-infill development peak flow rates. SUSTAIN successfully produced a runoff time series from the urbanised catchment pre- and post-infill development, and provided optimal solutions for the distribution of retention and detention based scenarios in some cases. Recommendations were not consistent however, with retention or detention recommended in various arrangements when identical optimisation runs were repeated. This may be because the case study catchment was too small. There were several difficulties encountered in the application of SUSTAIN. The most significant included a generally unstable operating environment and the requirement for an out of date operating system and ArcGIS software which inhibit recommendations for wider application at this stage. For research purposes, it is recommended that SUSTAIN be applied to larger catchment areas to determine whether it can produce consistent advice on the optimal placement of retention or detention based WSUD.

Finally, the suitability of the MUSIC model as a tool for stormwater quality assessment in South Australia was investigated to provide guidance on its application for identifying the broad effectiveness of WSUD strategies. This study applied the model to identify suitable parameters for South Australian urban catchments based on the known parameters of the Frederick Street and Paddocks catchments. The results indicated that MUSIC provided a good estimate of the flow volume and peak flows with input parameters derived from calibration. The default parameters provided in MUSIC produced an error in flow volume estimation in the order of 70%. The analysis found that different input values were required for the two catchments and a further investigation is necessary to assess input data for the use of MUSIC in SA. This will be necessary to develop guidelines for practitioners to apply the model with more confidence to assess pre- and post-development flow conditions in ungauged catchments.

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1 Introduction

1.1 Project Background

The Goyder Institute for Water Research approved the project titled “Water Sensitive Urban Design Impediments and Potential: Contributions to the SA Urban Water Blueprint (Phase 1)” in October 2012. The overall project was proposed with three main tasks. This report describes the background, goals, aims, methodology and results of Task 3 of the project.

Water management is a key priority for South Australia. According the *SA Strategic Plan* (Government of SA, 2011), South Australia is a world leader in wastewater, irrigation, stormwater and groundwater management and the *SA Strategic Plan* has targets which aim to increase stormwater and wastewater reuse within SA by 2025. The *SA Strategic Plan* (Target 68) and the *30 Year Plan for Greater Adelaide* (SA DPLG, 2010) also indicate that while new housing in the Adelaide metropolitan area is currently being produced in a 50:50 ratio of infill to fringe (greenfield) development, it is intended to change this to 70:30 by 2040. This poses challenges to infrastructure in the existing urban environment as dwelling density and demand increases at a faster rate than in the past.

Based on these challenges, the research in Task 3 of the Goyder Institute WSUD project was broadly undertaken to examine the effectiveness of water sensitive urban design (WSUD) in the South Australian urban environment such that future development may be considered ‘water sensitive’. It is recognised that one of the areas that will influence the mainstream acceptance of WSUD in South Australia is an understanding the multiple benefits that can be delivered by WSUD systems. The value proposition for WSUD has traditionally focussed on its role in addressing water quality objectives for urban developments, however as noted in the recent WSUD document from the South Australian Department for Environment, Water and Natural Resources (DEWNR, 2013) there is a potential for WSUD to provide other benefits to urban development such as:

- a reduced demand for mains (imported) water through sustainable water harvesting and use
- flood mitigation and flood volume reduction by restoration of the natural flow regime
- frequent flow management for bank stability control, and

Understanding the full potential of WSUD for urban development in South Australia will assist in assessing and, where appropriate, building a strong business case for WSUD elements to be adopted.

1.2 Project Goals

The broad aim of Task 3 of the “Water Sensitive Urban Design Impediments and Potential: Contributions to the SA Urban Water Blueprint (Phase 1)” project was proposed to the Goyder Institute as follows:

To understand the potential of WSUD in South Australia to promote water conservation, reduced flooding risk, reduced impacts of frequent flow from development on watercourses, the development of green space and water quality impacts on Gulf St Vincent

The findings of this report were based on six studies including:

1. The development of a matrix of WSUD solutions suitable for different development scales.
2. Development of a methodology to examine the impact of WSUD approaches on runoff characteristics in urban catchments.
3. The application of this methodology to examine the impact of existing WSUD at case study sites in Burnside, SA (B-Pods) and Mile End, SA (Rain gardens)
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6. Preliminary assessment of the 'MUSIC' model for estimating flows when examining WSUD strategies for stormwater management plans from developed urban catchments in South Australia.

2 The role of various WSUD Approaches

2.1 Background

There are various guidelines which have been developed by local, state and federal governments in Australia which advise on the implementation of WSUD, from concept planning to detailed design. These guidelines include general recommendations regarding the suitability of WSUD features with respect to development type and scale. However, a review of these documents indicated that there were inconsistencies in recommendations with respect to development type and scale. Further difficulties were apparent because their terminology is not consistent when scale and development type are mentioned. The main goal of this task was to document a review of Australian WSUD guideline recommendations regarding the role of various constructed WSUD approaches at different scales of urban development (domestic, precinct and development wide). It is intended that the reviewed literature can be compiled into a single summary of recommendations. The findings of this chapter will be used to select suitable WSUD scenarios in the modelling undertaken for this research and in subsequent studies.

2.2 Methodology

The role of various WSUD approaches in the provision of urban water services at different development scales was explored by:

(a) reviewing literature on the services provided by WSUD components at the domestic, precinct and development scale. This was undertaken using published literature in South Australia and interstate.
(b) producing a summary table of WSUD options suitable for identifying appropriate WSUD with respect to this literature review.

2.3 Review of Services Provided by WSUD

Information on the services provided by WSUD is typically provided as part of WSUD guideline documents, which have been produced for much of Australia at the State Government or Local Government level. Almost all state level guideline documents in Australia have recommendations regarding the applicability of WSUD based on development scale. For example, South Australian guidelines for WSUD (SA DPLG, 2009, p.1-31) provide an overview of WSUD measures in Chapter 1 including suitable scales of application. Other literature has also provided recommendations for the selection of appropriate WSUD for a particular need or location. A review of existing information on service provision by WSUD with respect to development scale and type is provided in the following sections. The review is based on Australian and selected international literature.

2.3.1 Australia - National

Guidelines for WSUD policy and implementation were produced by BMT WBM (2009) on behalf of the Joint Steering Committee for Water Sensitive Cities as part of the National Water Initiative. The guidelines were intended to be a 'comprehensive national reference'. The guideline provides a means to evaluate WSUD options to suit the needs of a development. The guidelines included an overview of development types and suitable WSUD initiatives. These are reproduced in Table 2-1.

Table 2-1 – Suitability of WSUD options for various types and scales of development (adapted from BMT WBM, 2009, p.3-20)

Option		Household	Medium density	High rise	Commercial and industrial	Subdivision	Urban retrofit
Potable water demand production techniques	Water efficient appliances	Y	Y	Y	Y	Y	?
	Water efficient fittings	Y	Y	Y	Y	Y	Y
	Rainwater tanks	Y	Y	Y	Y	Y	Y
	Reticulated recycled water	N	N	Y	Y	Y	N
	Stormwater harvesting and reuse	N	N	?	Y	Y	Y
	Greywater treatment and reuse	Y	Y	Y	?	Y	Y
	Changing landscape form	N	?	N	N	Y	N
	Water use education programs	Y	Y	Y	Y	Y	Y
Stormwater management techniques	Sediment basins	N	N	N	N	Y	N
	Bioretention swales	?	Y	N	Y	Y	N
	Bioretention basins	Y	Y	N	Y	Y	Y
	Sand filters	N	?	N	Y	Y	Y
	Swales and buffer strips	Y	Y	N	Y	Y	?
	Constructed wetlands	N	N	N	?	Y	?
	Ponds and Lakes	N	N	N	?	Y	?
	Infiltration systems	?	?	N	Y	Y	Y
	Aquifer storage and recovery	?	?	N	?	Y	?
	Porous pavements	Y	Y	?	Y	Y	?
	Retarding basins	N	N	N	?	Y	N
	Green roofs/roof gardens	Y	Y	Y	Y	N	Y
	Stream and riparian vegetation rehabilitation	N	N	N	?	Y	Y
	Water quality education programs	Y	Y	Y	Y	Y	Y
Y = Potentially suitable, ? = possibly suitable, N = Generally not suitable							

2.3.2 South Australia

The South Australian *Water Sensitive Urban Design Technical Manual* (SA DPLG, 2009) provided descriptions of suitable development types and scale for a range of WSUD techniques in Chapter 1. These descriptions are summarised in Table 2-2. It should be noted that there is some inconsistency in the wording used to describe 'suitable development' within this document which makes it difficult to interpret in a consistent manner.

Table 2-2 – Suitable site conditions for WSUD measures (adapted from Chapter 1 of SA DPLG, 2009)

WSUD Measure	Focus		Suitable site conditions	Unsuitable conditions
	Water quality	Water Quantity		
Demand reduction	Low	High	Residential, commercial and industrial sites.	Where water quality does not meet end use requirements.
Rainwater tanks	Low	High	Proximity to roof. Suitable site for gravity feed. Need to incorporate into urban design.	Non-roof runoff treatment. Where tank water is not used on a regular basis.
Raingardens	Medium	High	Allotment scale.	Reactive clay sites. Near infrastructure.
Green roofs	Medium	Medium	Flat roofs, slopes up to 30 degrees.	Roofs not structurally suitable.
Infiltration Systems	High	Medium	Precinct scale.	Non-infiltrative soils. High groundwater levels.
Pervious pavements	High	Medium	Allotments, roads and car parks.	Severe vehicle traffic movement and developing catchments with high sediment load.
Urban Water harvesting	Medium	High	Residential, commercial and industrial, generally more viable for precinct scale sites.	Locations where demand is limited or adverse impacts to downstream users.
Gross pollutant traps	High	Low	Site and precinct scales.	Sites larger than 100 ha. Natural channels. Low lying areas.
Bioretention systems	High	Low	Flat terrain	Steep terrain. High groundwater table.
Swales	Low	Low	Mild slopes (< 4%)	Steep slopes.
Buffer strips	High	Low	Flat terrain	Steep terrain.
Sediment basins	High	Medium	Need available land area.	Where visual amenity is desirable.
Constructed wetlands	High	Medium	Flat terrain. Need available land area.	Steep terrain. High groundwater table.
Wastewater management	Medium	High	Where adequate treatment and risk management can be ensured.	

2.3.3 Queensland

The most current guidelines which provide guidance on the conceptual design of WSUD services in Queensland are those produced by Water by Design (2009), specifically written for South East Queensland. These guidelines are intended to give information on all aspects of the planning stage for a WSUD project. A reproduction of the suitability of various WSUD initiatives to development in urban areas and the rural fringe is reproduced in Table 2-3. Discussion is provided for specific WSUD measures from each realm of WSUD (conservation, wastewater reuse and stormwater management) within the document. As an example, Table 2-4 shows a reproduction of the scale and performance effectiveness of WSUD tools for stormwater management in South East Queensland.

Table 2-3 - Potential contribution of WSUD strategies and application in urban environments (adapted from Water by design, 2009, p.54)

WSUD Tool	**WSUD Strategy**			**Urban Core**	**Urban centre**	**Suburban**	**Peri-urban (rural)**
	Water conservation	**Wastewater minimisation**	**Stormwater management**				
Demand management							
- Internal	Y	Y		Y	Y	Y	Y
- External	Y					Y	Y
Roofwater harvesting	Y		Y	Y	Y	Y	Y
Stormwater harvesting	Y		Y		Y	Y	Y
Wastewater treatment and reuse	Y	Y		Y	Y	Y	Y
Gross pollutant capture devices			Y	Y	Y		
Sedimentation basins			Y			Y	Y
Grassed/vegetated swales	Y		Y			Y	Y
Sand filters			Y	Y	Y		
Bioretention systems	Y		Y	Y	Y	Y	Y
Constructed wetlands			Y		Y	Y	Y
Porous pavements			Y	Y	Y	Y	
Infiltration measures	Y		Y			Y	Y

Table 2-4 - Scale of WSUD measure application and performance effectiveness (adapted from Water by design, 2009 p.25)

	Scale			Runoff quality and quantity management effectiveness		
	Allotment scale	Street scale	Precinct or regional scale	Quality treatment*	Peak flow attenuation	Reduction in runoff volume
WSUD Measure						
Gross pollutant capture devices			Y	L	L	L
Sediment basins			Y	M	M	L
Grassed/vegetated swales		Y	Y	M	M	L
Sand filters	Y	Y		M	L	L
Infiltration systems	Y	Y		N/A	L	H
Bioretention systems	Y	Y	Y	H	M	L
Constructed wetlands		Y	Y	H	H	L
Rainwater tanks	Y			L	M	H[a]
Porous pavements		Y		L	L	M/H

* Effectiveness in removing key environmental pollutants such as TSS, TP and TN
[a] With reuse
L = Low, M = Medium, H = High

The Water by Design (2009) information tends to provide water quantity and water quality ratings to devices in a similar fashion to those presented in Table 2-2 from the SA DPLG (2009) guidelines. However, some ratings are different when the high, medium and low ratings for water quality and quantity management are compared for each device. In addition, the Water by Design (2009) guideline has more consistent and wide ranging advice for selecting WSUD approaches to suit development scale. The consistency is based on the use of categories in Table 2-4, as opposed to the general description provided by SA DPLG (2009), which contains useful information but has inconsistent terminology.

2.3.4 New South Wales

The NSW catchment management authority produced an interim reference guideline document (NSW CMA, 2012) to guide practitioners on appropriate WSUD for development in Sydney. This document is directs the reader to the *South East Queensland Concept Design Guidelines* (Water by design, 2009) for selecting appropriate WSUD measures.

2.3.5 Victoria

A key champion for WSUD practice in Victoria is Melbourne Water, a State Government owned entity responsible for water supply, wastewater treatment and major drainage system management in the Greater Melbourne area. The guidelines for WSUD practice recommended by Melbourne Water[1] are those published by CSIRO (2005), which provide guidance on the design of WSUD measures for stormwater management in Victoria. However, there is little broad advice on selection of measures to suit site requirements provided by CSIRO (2005).

In response to this, there have been several guidelines produced by Local Government in Victoria which provide advice on WSUD selection based on development scale. For example, guidelines from the City of Yarra (2007) have been reproduced in Table 2-5. The recommendations from City of Yarra (2007) have a wider and more consistent approach to recommending WSUD features based on scale when compared to the South Australian guidelines summarised in Section 2.3.2.

Table 2-5 – Guide to treatment options and their application (adapted from City of Yarra, 2007, pg. iii)

		Scale			
Treatment type	**WSUD element**	**Small**	**Medium**	**Large**	**Broad**
Water use reduction	Appliances	Y	Y	Y	
Waterway rehabilitation	Local action	Y	Y	Y	Y
Rain water reuse	Tanks – general		Y	Y	Y
	Tanks – household	Y			
	Design	Y	Y	Y	Y
Grey water reuse	Overview			Y	Y
	Diversion	Y			
	Subsurface wetland	Y	Y		
	Biological processes		Y	Y	Y
	Recirculating media filter		Y		
	Depth filtration		Y	Y	Y
	Membrane filtration			Y	Y
	Water disinfection	Y	Y	Y	Y
Blackwater reuse	Overview	Y	Y	Y	Y
	Biological processes		Y	Y	Y
	Recirculating media filter		Y		
	Depth filtration		Y	Y	Y
	Membrane filtration		Y	Y	Y
	Water disinfection		Y	Y	Y
Stormwater treatment and discharge	Rain gardens	Y			
	Water quality		Y	Y	Y
	Gross pollutant traps		Y	Y	Y
	Sedimentation		Y	Y	Y
	Lakes and ponds			Y	Y
	Swales and buffer strips		Y	Y	Y
	Bioretention systems		Y	Y	Y
Stormwater treatment and reuse	Bioretention systems		Y	Y	Y
	Wetlands		Y	Y	Y
	Water disinfection		Y	Y	Y

[1] See http://wsud.melbournewater.com.au/

2.3.6 Tasmania

The Derwent Estuary Program produced a WSUD engineering procedures manual (DEP, 2005) which provides a guide to WSUD planners on the selection of appropriate WSUD measures for their site of interest. Factors considered include the available area for WSUD measures, the total catchment area, cost, a list of suitable/unsuitable site conditions and applicable development types. The analysis is reproduced in Table 2-6 and Table 2-7. Where water quality is being considered, there is guidance on selecting WSUD tools for particular target pollutant types. Like the recommendations of City of Yarra (2007) and Water by design (2009), the DEP (2005) recommendations are generally more consistent with terminology than the South Australian guidelines in Section 2.3.2 and allow a wider scope of sites to be assessed. However it should be noted that the DEP (2005) guidelines are focussed on the stormwater runoff quality realm of WSUD. There is little consideration for managing stormwater runoff volume, flow rate, reuse, mains water demand or wastewater.

Table 2-6 – Rating of WSUD measures and their benefits and costs (adapted from DEP (2005), p.2-4)

	Treatment of:				Other factors:			
Design element	Sediment	Hydrocarbons	Metals	Nutrient loadings	Maintenance	Space required	Treatable catchment area	Cost
Sedimentation basin	3	1	1	1	3	3	3	3
Bioretention swale	3	3	3	3	1	2	2	2
Bioretention basin	3	3	3	3	1	2	2	2
Sand filter	3	2	2	1	3	1	1	2
Swale buffer	2	1	1	2	1	1	1	1
Constructed wetland	3	3	3	3	3	3	3	3
Ponds	2	2	2	2	2	3	3	3
Infiltration	3	3	3	3	2	3	2	1
1 = Not high, 2 = Fair, 3 = High/large								

Table 2-7 – Treatment application for development types (adapted from DEP (2005) p.2-5)

Location	Sedimentation basin	Bioretention swale	Bioretention basin	Sand filter	Swale buffer	Constructed wetlands	Ponds	Infiltration measures
New streets in large or small development areas								
- On slopes < 4%	Y	Y	Y	N	Y	Y	Y	Y
- On slopes > 4%	Y	N	Y	Y	N	N	N	N
Existing streets and roads where drainage or pavements to be upgraded or road duplicated								
- On slopes < 4%	Y	Y	Y	Y	Y	Y	Y	N
- On slopes > 4%	Y	N	Y	Y	N	Y	N	N
Public land including open space where land area and use allow additional features to be installed	Y	Y	Y	Y	Y	Y	Y	Y
New residential development, low, medium and high density	Y	Y	Y	Y	Y	Y	Y	Y
Existing residential development, low, medium and high density	N	N	Y	Y	N	N	N	Y
Commercial and industrial properties	N	Y	Y	Y	Y	N	N	Y
Car parks, public or private	N	Y	Y	Y	Y	N	N	Y
Y = Highly suitable, N = moderate to low suitability								

2.3.7 Northern Territory

In support of the Darwin Harbour WSUD Strategy, McAuley and McManus (2009) produced recommendations on ideal WSUD targets for key development types, and appropriate WSUD options with respect to development type. These recommendations are reproduced in Table 2-8 and Table 2-9. The guidelines from McAuley and McManus (2009) are among the few guidelines which provide advice on adopting WSUD measures to achieve WSUD goals beyond the context of stormwater runoff management. However it should be noted that these goals are more relevant to the specific policy settings for achieving the goals for Darwin Harbour.

Table 2-8 – Recommendations on WSUD objectives for different development types (adapted from McAuley and McManus, (2009) p.6)

Development type	**WSUD Objective**	
	Stormwater quality	**Potable water conservation**
Single allotment	Not recommended	Recommended
Medium and high density residential	Recommended in medium term	Recommended
Large residential subdivision	Recommended	Recommended
Commercial and industrial	Recommended	Recommended
Government buildings	Recommended in the short/medium term	Recommended
Infrastructure	Recommended in the short/medium term	Not applicable

Table 2-9 – Recommended WSUD measures for different development types (McAuley and McManus (2009) p.14)

					Public sector			
WSUD measures		**Single, detached dwellings and low density attached dwellings**	**Medium and high density residential development**	**Commercial and industrial development**	**Buildings**	**Open space**	**Transport infrastructure**	**Major subdivisions**
Potable water conservation	Water efficient fittings and appliances	3	3	2	3	1	1	1
	Water efficient landscaping	3	2	2	1	3	3	2
	Rainwater tanks	3	1	3	2	1	1	1
	Water recycling	1	1	2	1	1	1	3
	Stormwater harvesting and reuse	1	1	1	1	3	1	3
Stormwater quality	Gross pollutant traps	1	1	2	1	1	1	3
	Swales	1	1	1	1	2	3	3
	Bioretention systems	1	1	1	1	3	3	3
	Wetlands	1	1	1	1	3	1	3
	Infiltration	2	1	1	1	1	1	3
1 = not practical for this development, 2 = recommended measure for this development, 3 = ideal WSUD measure for this development								

2.3.8 Western Australia

The Government of WA Department of Water has produced a number of brochures which describe WSUD features. These documents are limited to the stormwater management realm of WSUD, but include advice on suitable goals with respect to stormwater runoff management using structural WSUD (Table 2-10) and which measures should be considered with respect to development scale (Table 2-11).

Table 2-10 – WSUD measures in relation to peak flow frequency and recommended objectives (adapted from Government of WA DoW, 2011)

ARI	**0 to 1 Years**	**1 to 5/10 years**	**Greater than 5 or 10 years**
Objectives	Source control	Runoff control	Safe conveyance and discharge
	Capture or prevent runoff from impervious surfaces and manage water quality	Retain, detain and convey stormwater, manage stormwater quantity for serviceability and reduce erosion	Convey, protect from flooding
Structure	Rainwater tanks Pervious paving Soakwells Biofilters Tree pits Litter/sediment traps Hydrocarbon management	Overflow pipes Swales and buffer strips Infiltrations basins Infiltration trenches Dry or ephemeral detention areas Living streams Constructed wetlands	Major system conveyance by overland flow along roads and floodways

Table 2-11 – Recommended WSUD measures for different development types (adapted from Government of WA DoW, 2011)

	District	**Precinct (subdivision)**	**Street**	**Lot**
Bioretention		Y	Y	Y
Constructed wetlands	Y	Y		
Dry or ephemeral detention	Y	Y		
Infiltration basins and trenches	Y	Y	Y	
Litter/sediment traps		Y	Y	Y
Living streams	Y	Y		
Pervious paving			Y	Y
Rain storage and reuse			Y	Y
Swales and buffer strips	Y	Y	Y	Y

2.4 Analysis of Literature Review

Using an informed judgement of the recommendations in the review, a summary table was produced to cross match the recommendations of the Australian guidelines with respect to development scale and WSUD feature. All features mentioned in more than two guidelines with respect to scale were included. The results are shown in Table 2-12. Development scale was simplified into three categories, namely the individual allotment (small), the street scale (medium) and the subdivision/suburb (large). Where a guideline recommended implementation was 'possible' but not recommended, a 'half' recommendation was recorded. Using this approach, Table 2-12 gives an indication of how strongly each WSUD item was recommended with respect to scale.

Table 2-12 – Summary of WSUD features mentioned by guidelines and the number of recommendations with respect to scale

	Allotment*	Street*	Precinct or subdivision*
Demand reduction	3 of 3	3 of 3	3 of 3
Rainwater tanks	6 of 6	0 of 6	0 of 6
Infiltration Systems	3.5 of 5	3 of 5	2 of 5
Pervious pavements	1 of 4	3 of 4	2 of 4
Urban Water harvesting	0 of 3	1 of 3	3 of 3
Gross pollutant traps	1 of 5	2 of 5	5 of 5
Bioretention systems	4 of 6	4 of 6	6 of 6
Swales	1.5 of 5	4 of 5	5 of 6
Buffer strips	2 of 3	2 of 3	3 of 3
Sediment basins	0 of 4	1 of 4	4 of 4
Constructed wetlands	0 of 5	3 of 5	5 of 6
Recycled wastewater/reuse	0 of 2	1 of 2	2 of 3
Greywater reuse	2 of 2	1 of 2	2 of 2
Ponds	0 of 2	0 of 2	2 of 2
Detention basin	0 of 2	0 of 2	2 of 2
Sand filter	2 of 3	2 of 3	0 of 3

* NOTE: "x of y" indicates that this feature was explicitly mentioned with respect to scale in y guidelines, and recommended for this development scale by x guidelines.

All guidelines recommended rainwater tanks for allotment scale only. This was generally true, with few examples of their application at larger scale in the South Australian WSUD site catalogue presented by Tjandraatmadja et al (2014).

There was disagreement on the adoption of infiltration systems with respect to scale. This however may be a disagreement over the definition of an infiltration system. For example, larger scale infiltration basins are applicable at the medium scale, while allotment scale infiltration systems have been implemented as a policy in areas of Australia such as City of Gosnells, WA (Tennakoon et al., 2011).

Pervious pavements also tended to be recommended for precinct and larger scale adoption. However they are typically recommended in design literature to be installed as a ratio of the area of pervious paving to the contributing impervious area of approximately 1 to 1, depending on slope and location (Argue, 2004). In this circumstance, the allotment scale may still be suitable. However precinct scale may be referring to the installation of pavements in municipal car parking areas (e.g. multiple pavements with limited scale catchments).

Similarly, bioretention systems were most recommended for large catchment scales, but this may be referring to the installation of multiple bioretention systems over large areas. For example, small scale bioretention systems have been installed across the City of West Torrens and City of Salisbury are using multiple engineered biofilter systems in a central location to treat stormwater for harvesting. This illustrates the ability of bioretention to adapt to scale and need.

Overall, the findings of this chapter will be of use in selecting WSUD scenarios to explore in the greenfield and infill development scenarios in subsequent research. It is recommended that future updates to WSUD guidelines consider the findings of this review for the provision of advice on selection of WSUD techniques with respect to development scale. A better definition of development type and scale, where referred to, would also be beneficial when producing such guidelines. It may also be beneficial to supplement this review in future with information on the effectiveness of each WSUD feature with respect to aims. For example, depending on circumstances, a feature that has runoff quality benefits may be more suitable to certain needs that one with runoff quantity or reuse benefits. For example, if stormwater runoff is a priority and quality is important, a rain garden may be expected to provide more benefit than a detention system. Similarly, if water demand reduction is important, then the application of rainwater retention systems is of greater benefit than infiltration or detention measures.

3 Identification of a Methodology to Assess the Impact of WSUD on Catchment Flow Characteristics

3.1 Background

According to *South Australia's Strategic Plan* (Government of SA, 2011) and the *30 Year Plan for Greater Adelaide* (SA DPLG, 2010, p.17), the ratio of infill development to fringe development in metropolitan Adelaide will gradually shift from the current 50:50 until about 70% of all new housing is being built within existing urban areas 'to create an efficient urban form'. Infill development poses challenges to State and Local Governments with regard to the management of stormwater runoff. This is because increasing the density of existing urban catchments tends to result in increasing levels of impervious area. During storm events, this increased impervious area leads to an increase in the volume of runoff , an increase in the peak flow rate of runoff, a reduction in the time to peak flow compared to the existing catchment (Jacobsen, 2011) and an increase in flood frequency (Moscrip and Montgomery, 1997). Urbanisation also has impacts on the amenity and natural function of downstream ecosystems (Booth and Jackson, 1997; Fox et al., 2007). All of these impacts have been acknowledged by the South Austrlian Government in a recent WSUD statement produced by DEWNR (2013).

Since exisitng stormwater management systems are designed for urban development with a lower impervious area than that which is likely to exist following the proposed shift to more infill development proposed in *South Australia's Strategic Plan* (Government of SA 2011) and the *30 Year Plan for Greater Adelaide* (SA DPLG, 2010), increased flows may be beyond the capacity of the exisiting drainage system. This has implications for risk management because of a greater frequency of flood occurrence due to the now under capacity stormwater drainage system. The current design flow rates also increase, such that a rain event which occurs (say) every 5 years at present may be expected to produce a more severe flood in future due to greater runoff rates and volumes than that which the existing stormwater drianage system was originally designed. This has several implications. For example, cost estimates for flood damage incurred for a flood event following a 5 year ARI storm event of critical duration in the City of Holdfast bay and City of Marion were $1,200,000 with current development levels. However, should current development trends continue (with additional dwelling subdivision accounting for almost all development in the area of study) the cost of the resulting flood will increase to $4,900,000 due to the increased flooding and dwelling numbers impacted (Tonkin, 2013). There are several options available to manage the increase in peak flow:

Option 1 **Accept the reduced capacity** of the system to manage flooding at the desired frequency. For example, if increased urban density reduces the capacity of the drainage system from one designed to manage a 1 in 5 year storm event to one that can only manage a 1 in 2 year storm event, then accept this reduced capacity and the associated increase in flood frequency and cost.

Option 2 **Increase the capacity of the stormwater drainage system**. This involves the construction of new drainage systems in addition to the existing pipe and channel network to enable it to carry greater flow rates and volumes of stormwater to a point of safe discharge.

Option 3 **Implement a policy of increased floor levels for new development sites.** Such measures overcome any lower level flooding issues which arise from failure of the drainage system.

Option 4 **Implement a programme of progressive on-site flow management measures** for new re-development sites. Such measures can include detention tanks, extended detention tanks, retention tanks, infiltration systems and any other measures which withhold water on a site prior to discharge, infiltration or reuse.

Option 1 may be the cheapest alternative in the immediate future, but comes at the risk of accepting more frequent flooding and associated costs for stormwater flow events beyond the capacity of the drainage system. When flooding becomes apparent, by observation or by community feedback, Option 2 tends to be undertaken by catchment managers to manage flooding. However, the upgrade of stormwater systems is expensive. In addition to material and construction costs, stormwater management systems tend to follow transit corridors, and such works may be expected to interrupt day-to-day activity in the area. This requires traffic management and/or road closures which can cause safety concerns and disruption. In addition, the presence of other services such as electricity, gas, mains water and wastewater pipelines also requires design and construction crews to adequately plan and implement a drainage solution which does not interfere with other services. Undertaking an upgrade to the stormwater drianage system also moves the issue of larger stormwater volume and increased flow rates downstream which can not only impact on the performance of downstream infrastructure,but also downstream ecosystems. Increasing the volume and peak flow rates into downstream ecosystems can have detrimental impacts on overall ecosystem health, including water quality degradation and erosion of natural systems (Booth and Jackson, 2007; Fletcher et al., 2013). Option 3 measures, where an increase in flood frequency is accounted for by adjusting building regulations to increase floor levels is also an option to protect new housing from increased flooding. However, such a solution fails to protect existing residents and businesses from increased flooding.

Option 4 measures represent an alternative measure to address flow regime change due to development, and it is currently being implemented in South Australia. According to the SA Building Code (ABCB, 2013), it is currently mandatory for all new dwellings and some home additions to include an alternative water supply to the mains water supply system. In the absence of a recycled water source (such as recycled wastewater or harvested stormwater from a municipal scale scheme) this is typically achieved by implementing a minimum one kilolitre rainwater tank. In addition, local government such as the City of Tea Tree Gully and City of Mitcham have detention policies for new developments in addition to this requirement. Such policies require new development to detain stormwater on-site using detention tanks in such a way that runoff is collected and stored by a tank of specified size and allowed to drain away through an orifice of specified size. These tanks are designed to capture the initial volume of runoff during a rainfall event to reduce the peak flow rate of stormwater runoff from development sites up to a particular design storm interval. As such, these measures may be considered to be steps toward achieving the runoff quantity performance principle in the WSUD document published by DEWNR (2013): helping to manage flood risk by limiting the rate of runoff to downstream areas and attempting to preserve the 5 year ARI and 100 year ARI peak flow rates.

However, the implementation of Option 3 measures has tended to be based on design techniques centred on 'design storm' approaches, which require assumptions in the design process. In this section of the report, methodologies to assess the impact of WSUD on the runoff volume and peak flow rate are reviewed, and a methodology developed for application in subsequent sections of the report.

3.2 Aims

The aim of this research is to determine a preliminary methodology to investigate the contribution that WSUD may have on the reduction of minor drainage system peak flow rates and runoff volumes in urbanised catchments subject to infill development.

The minor system was the focus of the research as it represents the majority of drainage infrastructure expenditure in urban developments to protect the community from unecessary hazards due to frequent flooding (O'Loughlin and Robinson, 1999). On this basis is was considered that preserving the existing capacity of the minor drainage system is a priority for local government.

3.3 A Review of Existing Methodologies for the Impact of WSUD on Runoff Flow and Volume

3.3.1 Literature Review

Several studies have investigated the impact of WSUD features to manage flow rate, volume and flooding from developed catchments. A summary of recent studies is provided in the following paragraphs.

Roldin et al (2012) studied the impact of using infiltration systems in suitable areas across a 300 Ha catchment in Copenhagen, Denmark, to reduce combined sewer overflow frequency. In the study by Roldin et al. (2012), combined sewer overflows (CSOs) occur when the combined wastewater and stormwater flows 'overflow' from catch drains, which is analogous to the occurrence of flooding at a point in the current study for Adelaide. Roldin et al. (2012) presented a novel way of assessing the performance of infiltration systems to reduce CSO frequency in urban areas. In their case, the CSOs resulted in a spill of combined stormwater and wastewater into a local stream (Harrestrup Stream) from three structures, one of which provided 95% of the total CSO volume into the stream. Modelling of the entire catchment was undertaken with MIKE URBAN CS/MOUSE. The modelling was discontinuous; 10 years were simulated, but dry periods removed. The study also considered groundwater interaction. The calibrated model estimated the frequency and flow volume of CSOs at the main CSO spill location under three scenarios – the catchment with no infiltration, with an 'optimistic' soakaway distribution, and with a 'realistic' soakaway distribution. The study found that the volume and mean of CSOs were reduced by implementing infiltration systems. For example, the mean frequency of CSOs was reduced from 5.2 per annum to 4.4 at the assessment location when a realistic distribution of infiltration measures were applied (the realistic implementation was mainly restricted by the inability to place infiltration systems in areas where groundwater levels prohibit infiltration). The study concluded by remarking on the importance of considering the true potential for infiltration in cases where runoff management is dependent on infiltration. It also stressed the importance of a continuous modelling approach to fully assess the impact of soakaway discharge to the main drainage system. It also recommends that an ideal assessment would include the impact of groundwater levels on soakaways and the drainage network.

Zhang and Hu (2014) studied the effectiveness of rainwater harvesting for flow management in a new industrial park in China. The study determined an optimum storage capacity based on forecast water demand in the park, and indicated the cost benefits of undertaking rainwater harvesting across the park using an optimally derived 900 ML storage. The study applied long term continuous modelling using daily rainfall data and self-developed model to predict the benefits of harvesting with respect to volume. The study presents a good example of the benefits of storage and reuse for volume benefits, but does not consider flow rates.

Ashbolt et al (2013) reported on the possibility of stormwater harvesting to restore pre-development flow in SEQ. The results were based on a calibrated model of an undeveloped catchment in EPA SWMM (3% impervious), over which typical developments with up to 70% impervious area were overlaid with varying levels of runoff interception by harvesting. The harvesting was based on the recommended guideline values of SEQ, which were to intercept the first 10 mm of runoff from areas with an impervious area of up to 40%, and 15 mm on lots with 40% or greater imperviousness. The model was run with a one hour time step, which was considered reasonable by Ashbolt et al. (2013) because the time of concentration of the 361 Ha catchment was in excess of 1 hour. It should be noted however that the change in the time of concentration post-development was not considered. This may be reasonably assumed to be less than one hour based on studies indicating that the time of concentration for a 1500 Ha urbanised catchment in Adelaide was less than one hour (Myers et al., 2013). Runoff flow rates in the study by Ashbolt et al. (2013) were only reported on a daily basis, with results reported based on the mean, frequency and duration of high flows, and a comparison of flow duration curves (or flow exceedance curves). The study indicated that runoff capture was able to reduce the daily mean flow rates toward pre-development levels, but was not able to reproduce pre-development runoff flow rates.

Liao et al. (2013) reported the use of the EPA SWMM model simulate design storms over a 374 Ha catchment in near Shanghai, China. They examined the impact of five WSUD practices (porous pavement, bioretention systems, infiltration trenches, rain barrels and swales) on flood volume, peak flow rate, and the catchment runoff coefficient. The drainage system was reported to have a 1 Year ARI design storm capacity. The study was conducted to assess the impact of WSUD practices on the 1 Year, 2 year and 5 year ARI storm events using representative design storms. The study showed that the WSUD practices were able to reduce the volume of runoff and flooding within catchments, and the peak flow rates. In this study, flood volume was reported as stormwater pit overflow in the SWMM model in a similar manner to Roldin et al. (2012). The study indicated that rain barrels, permeable paving and infiltration trenches generally performed best. Bioretention and grass swales produced lower levels of flood volume and peak flow reduction when applied in the manner simulated in this study. The performance of systems was found to decline with storm events of increasing duration. However this study did not provide any information on the assumed nature of systems at the beginning of the design storm event. For example, the performance of rain barrels, permeable paving and bioretention strongly depend on antecedent conditions prior to the application of a design storm.

Petrucci et al. (2012) reported an investigation into the impact of a program to install rainwater tanks on the existing houses of a 23 Ha residential catchment in France. The study assessed the effectiveness of source control techniques undertaken to reduce the occurrence of flooding in local streets. Source control was adopted by offering rainwater tanks to homeowners in the catchment area. Options for householders included one or two tanks of 0.6 m^3 or 0.8 m^3 each, resulting in the installation of 173 m^3 of tanks on approximately 157 of the 450 homes in the catchment (about one third). The authors examined the influence of the tank installation using an EPA SWMM model calibrated to observed data before and after the tank installation. The simulation was calibrated to runoff flow data pre-installation (4 months) and post installation (5 months). It should be noted that the calibration of the SWMM model did not involve the adoption of rainwater tanks post-installation; rather it used an adjustment of the impervious area interception parameters in EPA SWMM to avoid assuming a value for water usage. The study also assumed that rainwater tanks were connected to entire roof areas. Petrucci et al. (2012) found that the rainwater tanks used were generally ineffective as they were too small to influence runoff from large rainfall events. The tanks could however influence smaller, regular runoff events. They explored whether the impact translated into a reduction in flood volumes, but did so by examining the effect of the tanks on the peak flow of design storm events (using a calibrated model of the post implementation case). They found that the rainwater tanks had little impact on flooding using this procedure. It should be noted that this analysis did not consider the reported occurrence of flooding in the model, nor was influence of tank demand explored. The study generally reported that

- The adopted rainwater tanks were not able to reduce stormwater overflows onto the road of the development
- Rainwater tank size, not connected area, was the limiting factor for performance. More tanks of the same size would have little impact on the reported outcomes (however this study did appear to assume a full roof connection to each tank, which may be considered unrealistic in the Australian context)

Pezzaniti (2003) investigated the flood benefits of rainwater tanks in a small urban catchment located in Greater Adelaide. Using 100 years of historical rainfall data, the antecedent conditions of rainwater tank scenarios were determined for events that were identified as having the same duration and intensities as those typically used in the conventional design storm approach outlined in *Australian Rainfall and Runoff* (Pilgrim, 1987). The antecedent conditions for rainwater tanks when storms of a 5 year ARI occurred in the rainfall time series were variable, depending on tanks size and demand characteristics. For some events the rainwater tanks were found to be full prior to the critical rainfall period. This was largely due to the rainfall burst being embedded in a longer duration rainfall event. It was found that the preceding rainfall partially or entirely filled the tank prior to the critical rainfall period in most cases. Figure 3-1 illustrates this condition, showing the level of a rainwater tank reaching its capacity before rainfall bursts that were determined to be in the range of a 5 year ARI for the 45 to 90 minute storm duration.

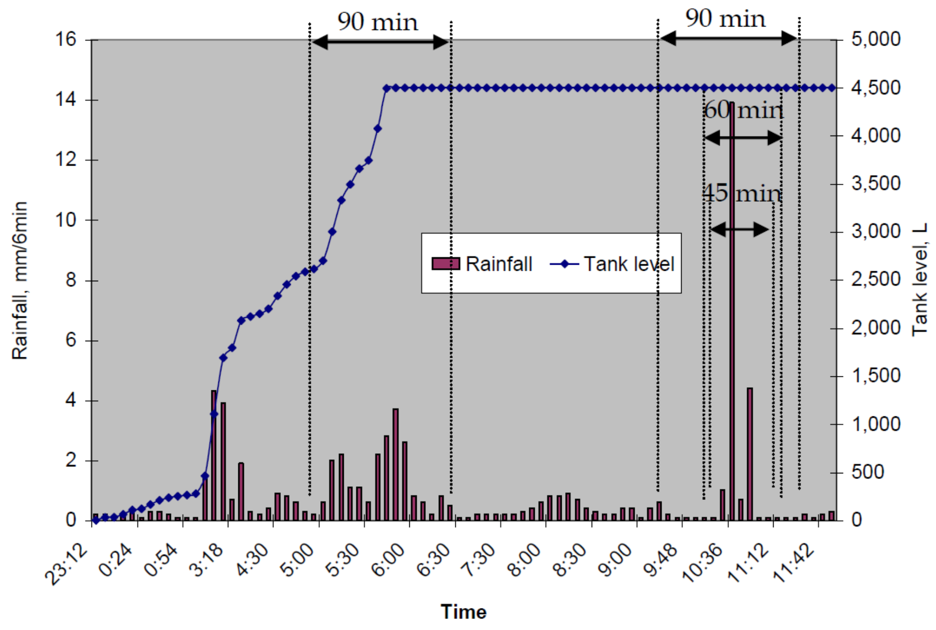


Figure 3-1 – Continuous simulation results highlighting the storage of a rainwater tank as rainfall bursts of a five-year ARI occur on 27/12/1929 (Source: Pezzaniti, 2003)

Pezzaniti et al (2004) studied the effect of detention and retention storages on peak flow rates in urban catchments by simulating hypothetical catchments in the DRAINS model using design storm techniques. The study explored the influence of the position of detention and retention storages in a catchment, and compared the performance of detention and retention regimes. The study ranked the results in order of effectiveness, generally concluding that if retention is applied, it is most effective as a distributed measure throughout a catchment (at the allotment scale, say) while if detention is applied, it is most effective as a lumped equivalent storage at the catchment outlet. The study did not provide details of any assumed antecedent conditions, applying a unit hydrograph technique for the design storm.

Hamel and Fletcher (2014) used the MUSIC model to examine the impact of rain gardens, rainwater tanks and a mixture of the two strategies to preserve the natural flow regime following a typical urban development of a 40 km^2 catchment into McMahons Creek, Melbourne, Australia. The study was based on hourly flow data. The preservation of the flow regime was examined using four flow statistics from calibrated pre-development and postulated post-development flow simulations with and without source control. The statistics compared for each scenario were total outflow, and metrics for the magnitude (95th percentile flow rate, Q95), duration (ratio of Q95 to total flow) and frequency of low flow spells (defined as the period over which flow remains below a threshold, in this case assumed to be the 75th percentile flow rate). Each metric was calculated on an annual basis and the mean and standard deviation were reported for the pre-development, post development and post-development/source control scenarios. The study showed that the assumed rainwater tanks reduced total flow, and restored the frequency of lower flows to pre-development values. However rainwater tanks could not restore the Q95 values or flow duration statistics. Raingardens did not reduce total outflow volume (systems were assumed lined), Q95 values or flow duration. However, frequency of low flow was restored using rain gardens. The best results were found using a combination of tanks and rain gardens. It should be noted however that flows statistics were reported on a daily basis and cannot be compared to instantaneous peak flows which cause drainage flooding due to drainage system overflows in urban catchments.

James et al. (2012) investigated the impact of distributed bioretention in a 154 Ha urban catchment to see whether bioretention could restore pre-development peak flow rates. The study was undertaken using the Bentley Sewergems software and simulation was based on design storms with the Unites States curve number technique for storm simulation. The study found that bioretention could restore peak flows, but not volumes, when they were assumed to infiltrate stored water. However, the study did assume that more than 7% of the treated catchment area was represented by bioretention which is higher than Australian guidelines, which typically cite approximately 2% of the catchment (Water By design, 2009).

Fennessey et al (2001) used a novel approach to investigate the effectiveness of stormwater ponds designed in accordance with design storm approaches on the overall flow regime from a 7.7 Ha undeveloped catchment over which a theoretical urban development was placed in simulations. The study applied 33 years of daily rainfall data in a hydrological model (uniquely developed for this project) to produce a time series of daily flow data from the catchment in pre-development, post development, and post development with stormwater pond. The stormwater pond design was based on then current regulations for detention pond requirements in newly developed catchments. The study took the 33 years of daily flow data and produced a partial series, or ranked series of daily peak flows which were then used to compute a Log Pearson Type III probability distribution of daily peak flows. The data was then used to estimate the 1 , 2 , 5 , 10, 25 , 50 and 100 year ARI resulting from the seven scenarios. The study found that while the design standards produced an outcome that was sufficient to detain flows from the larger flow events (greater than 10 year ARI), none were effective at preserving the 1 or 2 year ARI pre-development flow conditions. The methodology of this study is typical of those applied to produce estimates of the peak flow rates of rivers and streams in standard Australian design practice (Pilgrim, 1999).

3.3.2 Summary

The studies include a wide variety of techniques for simualating the impact of WSUD systems on the flow volume, peak flow rate and flood volume. The EPA SWMM was most widely applied for predicting flow rate, volume and flooding, however few studies assessed the impact of WSUD on peak flows over a truly continuous simulation period, which is required to capture the variability of rainfall and the impact of antecedent catchment conditions (which are influenced by the filling and emptying of storages).

3.4 Proposed Methodology

A diagram illustrating the general approach used to determine the impacts of WSUD systems on peak flow rate, runoff volume and flooding in urbanised catchments is shown in Figure 3-2.

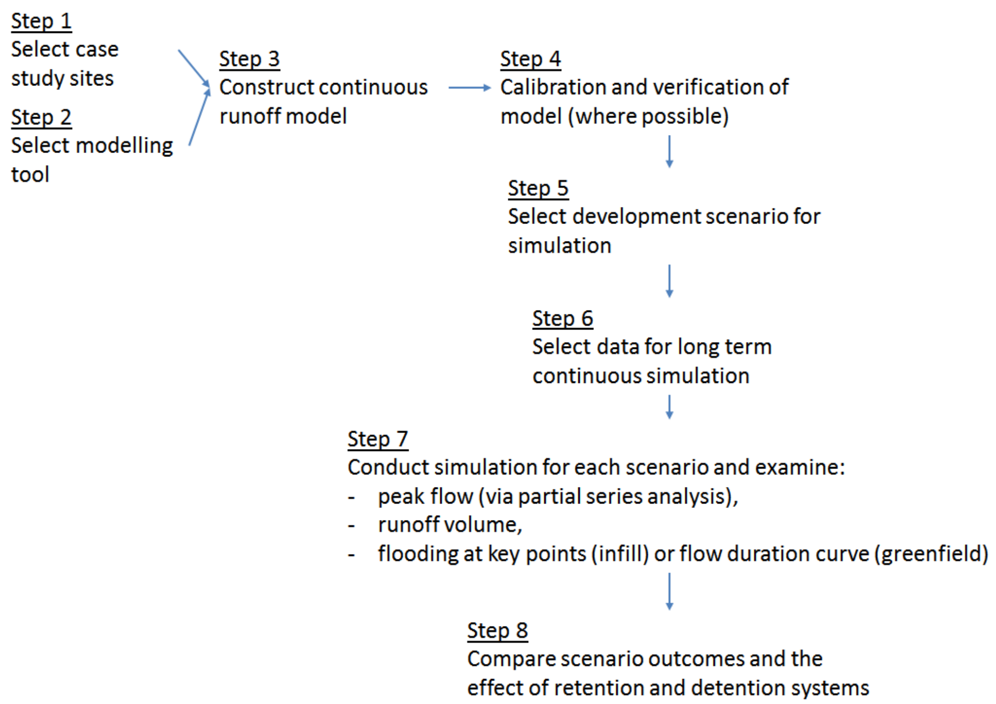


Figure 3-2 – Generalised methodology to examine the impact of WSUD retention and detention systems on peak flow and runoff volumes

The selection of case study sites (Step 1) is described in Section 3.4.1 with additional details specific to each catchment provided as part of each case study site in this report. The selection of modelling tools (Step 2) was identical for each case study site and the rationale is described in Section 3.4.2. The approach to model construction (Step 3) is generally described in Section 3.4.3 with specific details relevant to individual locations provided as part of each case study in this report. Model calibration and verification (Step 4) was undertaken where data was available for case study sites and the methodology for this is provided in Section 3.4.4. The approach for long term continuous simulation (Step 5) is described in Section 3.4.5, with further details on alternate scenarios provided in the case study sections of this report. The selection of a rainfall data series for long term continuous simulation (Step 6) is described in Section 3.4.6. The approaches for determining changes to peak flows (by partial series analysis), the runoff flow volume, flood frequency and the compilation of a flow duration curve (Step 7) are provided in Section 3.4.7.

3.4.1 Step 1 – Selection of Case Study Sites

The selection of case study sites to explore the contribution of WSUD to managing peak flows and runoff volumes from urban catchments in Greater Adelaide was undertaken in consultation with representatives from Local and State Governments. It was considered important that the selected sites include:

- Sites suitable for :
    - Investigating the impacts of infill development on peak flow rates and duration, and the potential for WSUD to address

    - Investigating impacts greenfield development on peak flow rates and duration, and the potential for WSUD to address this impact
    - A performance assessment (PA) of specific WSUD solutions in Task 1
- Sites of varying scale – small to large
- Sites with a flat and sloped topography
- Sites with recorded flow data for calibration and verification

Based on these criteria and consultation with Local and State Government representatives, the sites in Table 3-1 were selected. Two sites were specifically selected to examine the impact of WSUD features in small areas to support the post implementation performance assessment undertaken in Task 1. Further data specific to each catchment is provided in the report section for each case study. The location of each site relative to each other is shown in Figure 3-3.

Table 3-1 – Sites selected for peak flow and runoff volume investigations

Site	Type	Size (Ha)	Mean slope (%)*	Flow data	Report Section
B-Pods – Union Street, Dulwich	Performance assessment	3.4	1.2	No	4
Tarragon Street, Mile End	Performance assessment	2.3	0.2	No	5
Frederick Street, Glengowrie	Infill	45	0.3	Yes	6
Paddocks catchment, Para Hills	Infill	76	5	Yes	7
Flagstaff Pines, Flagstaff Hill	Greenfield	16	5	No	8
* Approximate, based on the slope of the main drainage system line(s)					

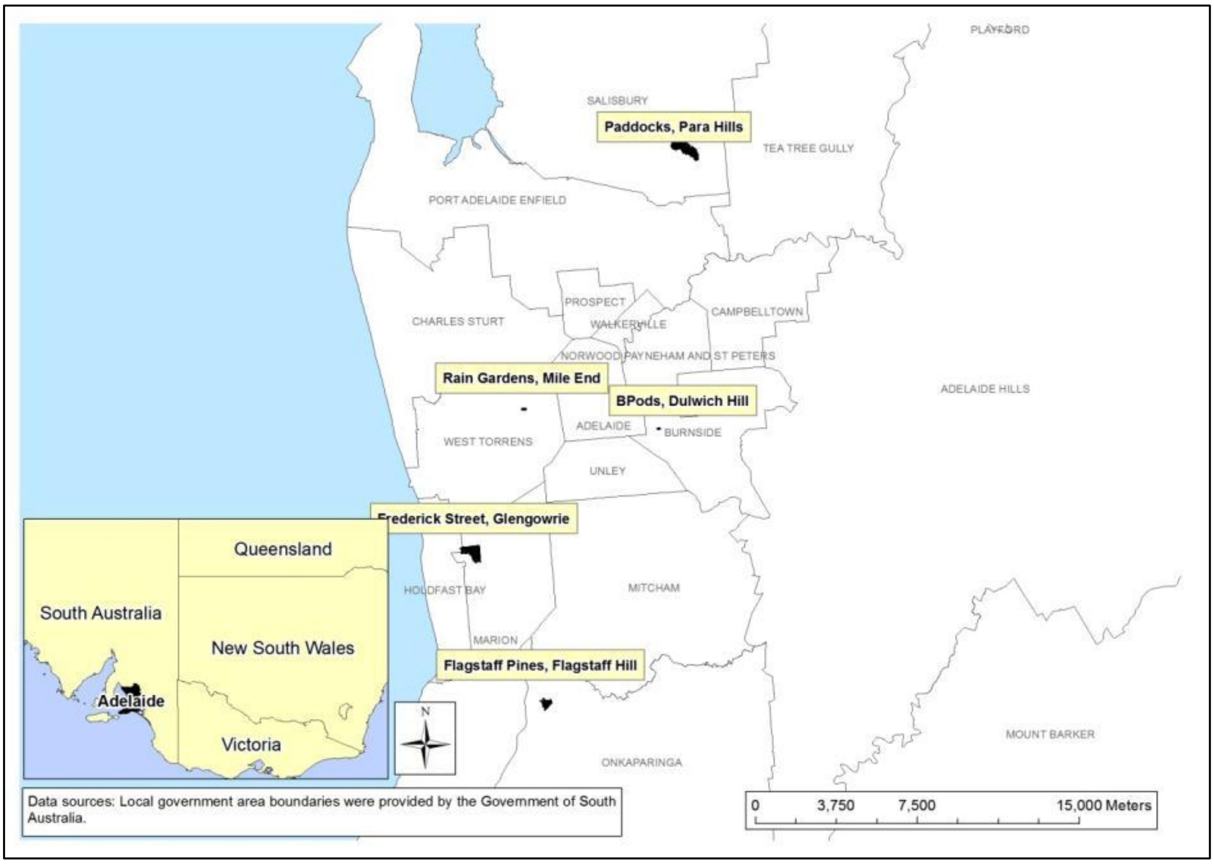


Figure 3-3 – Relative location of the five sites selected for peak flow and runoff volume investigations

3.4.2 Step 2 – Selection of a Continuous Simulation Model

The selection of appropriate models was undertaken with reference to key requirements of the study. These key requirements were compared with known modelling platforms using previous reports which have compared model performance (Elliott and Trowsdale, 2007; CWMR, 2010) and the literature review in Section 3.3.

Amongst the most important capabilities of a model selected for use in this study was the ability to:

(a) Simulate stormwater runoff discharge in the urban environment
(b) Effectively simulate both overland runoff development in the urban environment and flow through stormwater drainage systems (underground and above ground) using accepted methods in Australia
(c) Transport overland flow to catchment outlets, and in the case of an outlet at flow capacity, route flows in excess of this capacity (overflows) overland to the next outlet
(d) Undertake ‘continuous modelling’ (i.e. simulation of rainfall data over a number of years) to produce a time series of rainfall runoff and soil moisture conditions, in addition to event based ‘design’ rainfall analysis
(e) Determine surcharge volume and/or flooding area resulting from drainage system overflow
(f) Undertake modelling at a sub-daily time step (preferably minutes)

(g) Model the hydrologic influence of WSUD options including infiltration, detention, retention, and rainwater harvesting and reuse
(h) Aggregate subcatchments into larger catchments (for the scale-up of small systems)
(i) Be able to undertake dynamic modelling. For example, the model should take into account blockages or at capacity sections of pipe or channel downstream and reflect the impact of this on upstream flow and flooding
(j) Have a capacity for water quality simulation

It was also considered preferable that the model be generally applied by the Australian hydrological modelling community, and be capable of importing existing GIS data to allow for model construction to be assisted by the use of existing stormwater pipe, pit and catchment data provided by the South Australian Department of Environment Water and Natural Resources (DEWNR) and Local Governments.

Models which were considered for use in this study included:

(a) DRAINS
(b) EPA SWMM
(c) PCSWMM
(d) XPSWMM
(e) WaterCress
(f) MUSIC
(g) Urban Developer
(h) SOURCE Urban

Based on the criteria above, the characteristics of these models were compared and are presented in Table 3-2.

Table 3-2 – Summary comparison of model characteristics for selected hydrological models

Criteria	WaterCress	EPA SWMM	PCSWMM	XPSWMM	MUSIC	DRAINS	Urban developer	SOURCE Urban
Model stormwater runoff discharge in the urban environment	Y	Y	Y	Y	Y	Y	Y	Y
Effectively model both overland flow in the urban environment and flow through the existing stormwater pipe network using commonly accepted methods in Australia	N	Y	Y	Y	N	Y	Y*	Y*
Transport overland flow to catchment outlets, and in the case of an outlet at flow capacity, route flows in excess of this capacity (overflows) overland to the next outlet	Y*	Y	Y	Y	Y*	Y	Y*	Y*
Undertake 'continuous modelling' techniques (i.e. modelling based on rainfall data from a period ranging from days up to many years of measured rainfall data) of rainfall runoff and soil moisture conditions in addition to event based 'design' rainfall analysis	Y	Y	Y	Y	Y	Y*	Y	Y
Determine surcharge volume and/or flooding area resulting from system overload	N	Y	Y	Y	Y*	Y	Y*	Y*
Undertake modelling at a sub-daily time step	Y	Y	Y	Y	Y	Y	Y	Y
Model the hydrologic influence of WSUD options including on site infiltration, detention, retention, rainwater tanks, ponds and/or wetlands	Y	Y	Y	Y	Y	Y	Y	Y
Ability to undertake dynamic modelling	N	Y	Y	Y	N	Y	N	N
Aggregate subcatchments into larger catchments (for the scale up of small systems)	Y	Y	Y	Y	Y	Y*	Y	Y
History of application in the Australian context	Y	Y	N	Y	Y	Y	Y	Y
Ease of connectivity with existing GIS data	N	N	Y	Y	N	N	N	Y
Have a capacity for water quality modelling	Y	Y	Y	Y	Y	N	N	Y
Y = Yes, the model is capable of performing this function Y* = The model may be capable of performing this function but is approximate only and/or is measured using a workaround which may not be accurate and was not an intended function of the model during development N = No, the model is not capable of this function								

Based on the finding summarised in Table 3-2, the EPA SWMM, PCSWMM and XP SWMM models provided the most comprehensive coverage of the stated requirements. It should be noted that the EPA SWMM model is the underlying engine for PCSWMM, XP-SWMM and DRAINS (XPSWMM and DRAINS each have the ability to disable the SWMM engine and use simpler techniques if desired). Although DRAINS is widely used for the design of stormwater conveyance systems in Australia, it is not capable of continuous simulation. Following initial trials of EPA SWMM, XP SWMM and PCSWMM, it was found that XP SWMM did not perform well for continuous simulation of medium sized catchments. For example, when a 20 year run of the Frederick Street catchment was attempted using data with a six minute time step, the model was unable to produce results. Subsequent investigation found that XPSWMM produced a large output data file that was not able to be processed. In contrast, PCSWMM and EPA SWMM were both capable of producing the 20 year hydrograph efficiently. Due to the extra tools available in PCSWMM for model construction and data analysis, PCSWMM was selected for use in this study. A further benefit of this model choice was that PCSWMM models are fully compatible with EPA SWMM, an open source model, allowing any subsequent research to build on this project's outputs. It should be noted that use and awareness of PCSWMM in Australia at the current time was not considered important. PCSWMM is widely adopted for design and research overseas. Also, EPA-SWMM, which has been used in the Australian context, is the underlying hydrological and hydraulic model adopted by PCSWMM.

3.4.3 Step 3 – Construct Continuous Simulation Model

The collection and use of data for each case study site is described in detail as part of the reporting of each case study.

3.4.4 Step 4 – Model Calibration and Verification

Model calibration and verification was undertaken at the sites where rainfall and corresponding runoff flow data were available, namely the Frederick Street catchment and the Paddocks catchment. In each case, model calibration was driven by the need to produce a hydrograph with a good fit to the observed flow data. In producing a good fit to the observed data, peak flows and runoff volume were prioritised. Peak flows were prioritised because peak flows formed the basis of the partial series analysis used to estimate the recurrence interval of flow events. Runoff volume was also prioritised to ensure the model was not over- or under-predicting rainfall-runoff to estimate peak flows accurately. It should be noted that data was only available at single points in the drainage system for the Frederick Street and Paddocks models. In each case, however, total runoff was simulated to include the sum of flows in the drainage system and any overland flows) which were greater than the capacity of the drainage system.

To assess the overall fitness of the model to observed data during both model calibration and verification, the following model fitness statistics were employed:

- Nash-Sutcliffe efficiency, r^2
- Percent error in peak, PEP
- Sum of squared residuals, G

The Nash Sutcliffe efficiency is one of the most widely applied criteria to assess simulated and observed flow for hydrological models (Krause *et al.*, 2005; Jain and Sudheer, 2005). The Nash-Sutcliffe calibration statistic is considered sensitive to errors in peak flow, making it ideal for this study where peak flow values are of interest. The r^2 statistic was calculated by assessing the paired values of simulated and observed flow data using Equation 1 (ASCE, 1993):

$$r^2 = 1 - \frac{\sum_{i=1}^{n}(O_i - P_i)^2}{\sum_{i=1}^{n}(O_i - \bar{O})^2} \qquad \text{- Equation 1}$$

Where n represents the number of observed flow data points (effectively the number of time steps in the period of the event), $O_i$represents the observed flow at time i, $\bar{O}$ represents the mean observed flow over the period of the data and P_i represents the predicted flow at time i.

The value of r^2 varies from 1 to -∞. A value of 1 denotes a perfect representation of observed data by the model. A value of zero indicates that the model represents as good an estimate as the mean of the observed data, while anything less than zero indicates that the model performs worse than the mean of the observed data.

In accordance with the recommendations of the ASCE (1993) for presenting adequate data for comparing the adequacy of runoff simulation with other studies, the simple percent error in peak (PEP, Equation 2) and the sum of squared residuals (G, Equation 3) were also calculated using Equations 2 and 3 respectively.

$$PEP = \frac{O_{peak} - P_{peak}}{O_{peak}} \times 100 \qquad \text{- Equation 2}$$

$$G = \sum_{i=1}^{n}[O_i - P_i]^2 \qquad \text{- Equation 3}$$

Where O_{peak} represents the observed peak flow during the event and P_{peak} represents the predicted peak. The model was accepted as calibrated when r^2 values for events were above 0.8 and a majority of PEP values were less than 10%. This was because values of r^2 greater than 0.8 were considered 'high' in a study by Petrucci et al (2012) which investigated rainwater tank performance for peak flow reduction using simulation techniques with a similar data time step to that adopted here, The ± 10% accuracy of peak flow estimation was considered a reasonable estimate of fitness for the purposes of this study where a comparison is required.

The results of the model calibration and verification procedure are presented with the case study catchment data for Frederick Street (Section 6.3.3) and Paddocks (Section 7.3.3). Only a volumetric comparison was available for the Flagstaff Hill catchment, where pre-developed runoff data is compared with other catchments in the Adelaide region (Section 8.3.3).

3.4.5 Step 5 – Simulation Scenario Selection

The selection of a scenario first requires determining the desired level of flood protection which it is intended to provide. Throughout this study, it was assumed that an ARI between 6 months and 5 years is of particular interest to most local governments. This was based on interviews with metropolitan councils undertaken by Kellogg, Brown and Root (2004), in which most local governments indicated that their minor drainage system capacity was within this range. This is also consistent with the recommendations of Australian Rainfall and Runoff (Pilgrim, 1999). It is acknowledged that infill development and WSUD services may have an impact on the major drainage system, but this was not considered within the scope of this research.

For the purposes of this investigation where runoff volume and peak flow rate at the catchment level were a priority, WSUD systems were divided into two major types. These include retention systems and detention systems. The definition of retention and detention was adapted from a study by Scott et al. (1999). Detention systems were those which hold runoff for short periods to reduce peak flow rates, and detained flows are released into drainage systems and watercourses. While detention systems may alter peak flow rates, the volume of water discharged from a catchment fitted with detention systems remains relatively unchanged because water is only temporarily detained, not withdrawn from the system. Examples include:

- on site detention tanks,
- detention ponds, and
- lined bioretention systems/rain gardens which collect water for disposal.

Retention systems are those where stormwater is held for longer periods resulting in water being reused, infiltrated on site or evaporated. Retention systems may reduce peak flows as well as runoff discharge to the drainage system because water is effectively withdrawn from the formal drainage system and used on site and disposed of via infiltration and evapotranspiration, or as wastewater (in cases where it is used in toilets and clothes washing). Examples of retention systems include:

- rainwater tanks and other on-site reuse systems,
- infiltration systems,
- unlined bioretention/rain gardens which allow stormwater to infiltrate, and
- permeable paving.

The simulation scenarios for each of the case study catchments are presented in Table 3-3. Further details on each scenario, including the range of sub scenarios for each site, are detailed in the respective case study methodology sections. For Frederick Street, scenarios were selected in 1993, 2013 and 2040. These times were selected to coincide with existing flow data from the catchment, the current scenario (with infill development) and the infill scenario in 2040, which coincides with the end of the current 30 year plan for Greater Adelaide (SA DPLG, 2010). The Paddocks catchment was also investigated in 1993, when flows were available for calibration. Subsequent models were theoretical infill scenarios, as there has been no infill in the catchment since this time. The selection of WSUD scenarios to apply to infill and greenfield development in Frederick Street, The Paddocks and Flagstaff hill were selected based on the findings of the literature review in Section 2.

Table 3-3 – General overview of goals and scenarios for each case study catchment

Site	Assessment	Scenarios
Union Street, Dulwich	Performance assessment of B-Pod devices	1. Current catchment, no B-Pods 2. Current catchment, with B-Pods
Tarragon Street, Mile End	Performance assessment of bioretention systems	1. Current catchment, no bioretention 2. Current catchment, with bioretention
Frederick Street, Glengowrie	Examine the impact of infill development on runoff; examine the impact of WSUD systems on this runoff	1. 1993 catchment 2. 2013 catchment a. No WSUD devices b. Retention tanks c. Detention tanks d. Street scale bioretention 3. 2040 catchment a. No WSUD b. Retention tanks c. Detention tanks d. Street scale bioretention
Paddocks catchment, Para Hills	Examine the impact of infill development; examine the impact of WSUD systems on the change in runoff	1. 1993 catchment 2. 1993 catchment, 25% infill a. No WSUD b. Retention tanks c. Detention tanks d. Street scale rain gardens 3. 2040 catchment, 50% infill a. No WSUD b. Domestic rainwater tanks c. Domestic detention tanks d. Street scale rain gardens
Flagstaff Pines, Flagstaff Hill	Impact of greenfield development on flow frequency in a natural waterways; contribution of WSUD in protection of natural flow regime	1. Pre-developed catchment flows 2. Post development catchment flows a. Without WSUD b. With allotment scale retention c. With allotment scale detention d. Street scale rain gardens

3.4.6 Step 6 – Selection of Continuous Rainfall Data

For simulation of runoff from each case study site, a long term time series of continuous rainfall data was required to produce a sufficiently long runoff time series for partial series analysis (see Section 3.4.7). This data should ideally be from a location close to the site location and of adequate quality to capture variation in flows (i.e. with minimal periods of missing or accumulated data). The selection of a time period for analysis was based on a time period considered effective for a frequency analysis like that proposed in Section 3.4.7. *Australian Rainfall and Runoff* is a national guideline for rainfall and runoff estimation in the Australian context, and according to Book IV, Section 2 (Pilgrim, 1999):

"Suggestions have sometimes been made regarding minimum length of record that should be used in a flood frequency analysis. While it is always desirable to have at least 10 to 15 years of data, situations occur where short records may have to be used as there is no better alternative."

To identify suitable rain gauges, pluviograph records from the greater Adelaide region were retrieved from an online database of Bureau of Meteorology gauges provided by eWater[2]. From this, those with a minimum of 15 years of continuous record were examined further. The quality of data was then assessed using the tools included with the MUSIC software package (eWater, 2012), which illustrated graphically where periods of missing and accumulated data were present in data records. It was found that most gauges, including the Adelaide (Kent Town) (023090) gauge immediately adjacent to the Bureau of Meteorology office in Adelaide, had periods of missing or accumulated data. The longest period of good quality rainfall data for simulation was found to be the Parafield Airport gauge (BOM ref 023013), where a continuous 19 year period of good quality data was available between January 1973 and January 1992. Some events were still considered suspicious, including unusually high rainfall intensity measured on 18 October 1973. However similarly high rainfall was noted to occur at a nearby rain gauge (Edinburgh RAAF, BOM ref 023083) at this time. As such, this 19 year continuous period of good quality data was adopted for producing a runoff time series for peak flow and runoff volume analysis in all case studies. In other circumstances where a sufficiently long time series is not available, or where a project aims to compare the effect of rainfall gauge in different areas, the investigation may consider adopting a lower quality rainfall data record with gaps and accumulated data adjusted using standard techniques. However, as an apparently complete 19 year rainfall record was available in the Adelaide region was available for this study, this process was not considered.

Personal communication with other researchers indicated that a successful partial series analysis should be based on a period of flow data 5 to 10 times longer than the ARI flow rate being determined. Based on this feedback, it is acknowledged that 19 years of data may not produce a highly accurate estimate of runoff for flood frequency analysis at the 5 year and 10 year ARI flow rates determined in accordance with Section 3.4.7, however in the absence of better rainfall or flow records, the 19 years rainfall data and subsequently predicted runoff flow rates were considered adequate for producing a reasonably accurate and, more importantly, precise estimate of peak flows for the purposes of comparing scenarios at each case study site.

In some case study catchments, multiple rain gauges were located in the catchment. This data was used for as rainfall input for calibration and verification of the catchment flow model. However, these rainfall records were not applied to produce a long time series hydrograph as there was

[2] See http://www.ewater.com.au/products/ewater-toolkit/urban-tools/music/pluviograph-rainfall-data-tool/

insufficient data available from these gauges. In addition, the time series included periods of missing data or periods of accumulated data.

3.4.7 Step 7 – Analysis of Runoff Volume, Peak Flow (Partial Series Analysis) and Flooding

Runoff Volume

The total runoff volume for each scenario was assumed to consist of the sum of total runoff from each simulated sub-catchment. The mean annual runoff volume was assumed to be the total runoff volume divided by the simulation period.

Peak Flow Analysis

To analyse the characteristics of peak flow in a given scenario for each case study site, flow frequency analysis techniques were applied to runoff flow rates. The runoff value was represented by the sum total of drainage flows (such as pipe flow or channel flow) at the outlet of the catchment, *plus* any overflow. The analysis of flow was based on the techniques described by *Australian Rainfall and Runoff*, Book IV (Pilgrim, 1999) and Ladson (2008). There are two main alternatives presented by these authors for estimating the frequency of peak flows:

- Partial series analysis
- Annual series analysis

According to Pilgrim (1999) and Ladson (2008), partial series analysis is appropriate for determination of flow frequency less than the 10 year ARI. For less frequent flows (greater than 10 years), the annual series analysis technique is preferred. The main focus of this research was to examine the impact of WSUD on more frequent flow rates (up to the five year ARI), and as such the partial series analysis technique was applied to examine the frequency of flows from the case study catchments with and without the inclusion of WSUD systems (retention and detention).

There was little information on the use of partial series analysis to estimate the return period of flows from urbanised catchments. The partial series analysis for this research was therefore conducted in accordance with the framework set out by Pilgrim (1999) and Ladson (2008), which generally refers to rural catchments and streams. Some techniques for peak flow analysis in urbanised areas were also adopted from Ghafouri (2006) who examined the peak flows from four urbanised catchments in New South Wales.

The partial series analysis began with the selection of a peak flow threshold value. With little guidance on the selection of this value for urbanised catchments, the method of Ghafouri (2006) was applied, where the annual peak flows were extracted and the minimum value of this annual series was applied as the flow threshold. The partial series was then determined based on the selection of all peak flow rates above this threshold value and separated by more than twelve hours. This separation was considered sufficient to ensure that flow peaks were selected which represented ‘unique’ events. While this may be selected as a number of days or even weeks for analysing stream flow from large rural catchments, the 12 hour period was considered sufficient for the relatively small urban catchments where formal drainage produces a much shorter runoff response time.

The number of events included in the partial series was assumed to be reasonable where the average number of events per year (*k*) was less than or equal to three times the number of years of record (*N*, 19 years). This approach was taken based on the methods adopted by Ghafouri (2006). While more recent data is available to suggest that *k* values of 4 or 5 may be appropriate (Pham, 2013) for partial series analysis, this study was undertaken on much larger catchments in New Zealand and no such study was available for Australian conditions, nor small urban catchments. As such, a maximum of *3N* (or 57) events was assumed to be appropriate for the partial series. In cases where more than 57 events were above the threshold, excess events were excluded from the partial series.

Based on the procedures of Pilgrim (1999) and Ladson (2008), it was assumed that the partial series of peak flows were represented by a negative exponential distribution, and the ARI of peak flows (or plotting position) was determined using Equation 4. This equation was recommended by Pilgrim (1999) and is generally referred to as the 'Cunnane formula' in other literature (Maidment, 1993; Cunnane, 1978).

$$PP(m) = \frac{N + 0.2}{m - 0.4} \qquad \text{- Equation 4}$$

Where *PP(m)* refers to the plotting position, and is equivalent to the estimated ARI of the individual flow rate, *N* refers to the number of years of record and *m* refers to the rank of the flow value in the partial series.

While other plotting position formulas are available, such as the Weibull, Gringorten and Hazen formulae, it should be noted that the selection of a plotting position formula is of greatest interest when the hydrological concern is characterisation of extreme flow events (such as greater than 20 Year ARI). In this study, the main interest is to identify flow events up to the 5 year ARI. The plotting position formula employed does not have a great influence over events of this magnitude in a time series with a sufficiently long enough to ensure that the 5 Year ARI is not ranked in the top three events of the partial series (Maidment, 1993).

According to Pilgrim (1999), the resulting peak flow rates for required average recurrence intervals can then be estimated by plotting the results log normally, with Log_{10} *[PP(m)]* on the x-axis and the corresponding peak flow on the y-axis. There is little information available on the best method to determine specific flow values from this chart, with recommendations including the fitting of a line of best fit (Pilgrim, 1999; Ladson 2008) or linear interpolation between individual points (Pilgrim, 1999). In this study, the value of the 1, 2, 5 and 10 year ARI flow values are determined using a linear fit to the full partial series as recommended by Ladson (2008). While this method may not give an accurate estimate of peak flow for design purposes, it was considered appropriate to estimate and compare the impacts of infill development and WSUD implementation in a catchment.

A complication to this approach became evident during trial runs of the existing urbanised catchments in Frederick Street, Glengowrie (Section 6) and the Paddocks, Para Hills (Section 7). An example of the partial duration series plot for the selected end point of the 1993 scenario of the Frederick Street catchment (Section 6) is shown in Figure 3-4. It shows how there is a linear trend to increasing peak flow rates as the average recurrence interval approaches approximately 2.5 years. After this, there appears to be a change in the linear trend. Further investigation indicated that peak flow rates with an average recurrence interval less than 2.5 years were almost completely conveyed by the underground pipe system for which the model was calibrated. When the ARI of peak flow was greater than 2.5 years, the flow rate at this point started to include more significant overland flow in addition to pipe flow because the drainage system in the catchment reached capacity (hence flows which cannot enter the drainage system were conveyed overland). This change is effectively a change in the flow conditions, where the minor drainage system is reaching capacity. These conditions must be considered in the analysis of peak flows. It was therefore necessary to ensure that peak flow estimates were split into the two populations illustrated in Figure 3-4:

1. Events where peak flow is conveyed in the pipe system, and therefore where peak flows may be considered below system capacity (e.g. for Frederick Street, flows less than the 2.5 year ARI) and
2. Events where the peak flows were conveyed by the underground pipe system and the overland flow channel, and therefore where peak flows may be considered above system capacity (e.g. for Frederick Street, flows greater than the 2.5 year ARI).

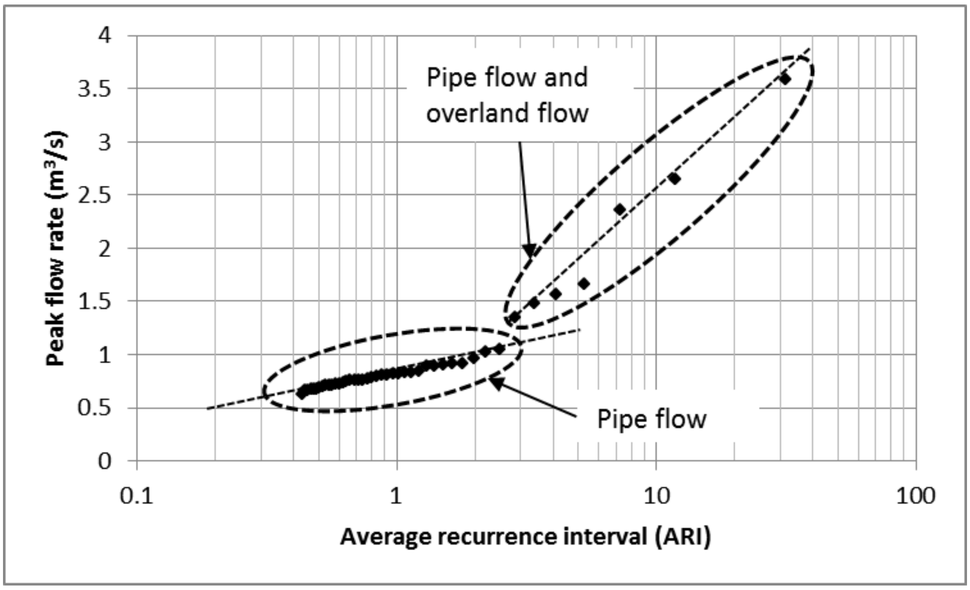


Figure 3-4 – Sample flow frequency curve based on partial series analysis of flows in Frederick Street, 1993 scenario

To address this, estimates of peak flows at standard durations (6 month, 1 year, 2 year and 5 year) were produced from the two separate populations - one population representing flow rates before overland flow becomes significant and one for flow rates where overland flow becomes significant. The capacity of each system is reported with in the peak flows section for each case study site.

Flooding

The occurrence of flooding in the catchment was quantified for the infill development case studies, namely Frederick Street (Section 6) the Paddocks (Section 7). The manner in which flooding was defined for these catchments is detailed in the methodology for these case studies – for Frederick Street in Section 6.3.5 and for the Paddocks in Section 7.3.5. In summary, the amount of flooding was examined by identifying a point in the catchment where flooding was known or suspected to occur. The occurrence of flooding at this point was reported as a time series of volumes at each time step in the SWMM model results. A partial series of the peak flood volumes was produced from this time series, and the recurrence interval of flooding in excess of an acceptable volume was estimated.

3.4.8 Step 8 – Determining the Effect of WSUD Retention and Detention Systems

The impact of WSUD retention and detention options was determined by comparing the peak flow rate, runoff volume and flood frequency for each of the scenarios at each case study site.

For case study sites where a performance assessment was undertaken (Sections 4 and 5), the comparison of the site runoff volume and peak flow characteristics were compared to illustrate the effectiveness (or otherwise) of the WSUD solution implemented with a 'no WSUD' case.

For the infill development case studies (Sections 6 and 7), the change in runoff volume and peak flow characteristics were compared with differing levels of development. The ability of WSUD to preserve runoff volume, peak flow rates and flood frequency to the pre-infill development threshold was determined.

For the greenfield site (Section 8), runoff volume and peak flow rates were determined for the 0.5, 1, 2 and 5 year ARI for a pre- and post-development scenario. The ability of WSUD retention and detention techniques to preserve the pre-development flow regime was then examined. In addition, the flow duration curve was constructed based on ranked flow rates in the time series generated from the simulation.

3.5 Summary

A need to identify a methodology to assess the impact of infill development (or greenfield development) on flow characteristics from an existing catchment was identified, in addition to a means of assessing the impact of WSUD on the post infill development (or greenfield development) flows. Characteristics of interest included:

- Mean annual runoff volume
- Peak flow rates
- Flood frequency (for post-infill development cases)
- Flow duration (where streams exist)

A review of literature was undertaken examining current approaches to this work. The review indicated there were several methods employed in existing studies, most of which were based on design storm events, limiting study findings with respect to WSUD because of assumptions required over the amount of water stored by systems when a design event occurs. With reference to established methods in Australian Rainfall and Runoff, a methodology was developed based on longer term continuous simulation. It was proposed that mean annual runoff volume pre-and post-infill development be assessed as the mean of annual flow volumes. A partial series analysis of peak flow rates at the end of the catchment was used to examine catchment peak flow characteristics. A case study location for flooding was identified and a partial series analysis of flooding volumes was conducted using a similar approach to the end of catchment peak flow. The flow duration curve was considered appropriate to examine the impact of greenfield development on urban streams.

4 Case Study 1: Performance Assessment of B-Pods in Union Street, Dulwich

4.1 Introduction

'B-pods' are a small scale, underground stormwater interception tank installed at the kerbside of local roads. Currently unique to the City of Burnside local government area, Adelaide, B-Pods are placed along road verges to intercept flows from individual house roofs. The B-Pod unit consists of a plastic 'milk crate' style unit surrounded by a geotextile which is placed in the ground. The unit is placed in line with the drainage pipe from property on the council owned verge adjacent to the road. The location of B-Pods is generally selected to provide passive irrigation of street trees. The water collected and stored in the tank system is allowed to infiltrate into surrounding soil. As such, they are defined as a retention system for the purposes of this study. B-Pods are designed for passive irrigation and to reduce flow volumes entering the street system. Figure 4-1 illustrates a typical B-Pod arrangement, including dimensions. Currently located in several streets across the City of Burnside, their impact on peak flow rates and runoff volumes has not been examined. There is however anecdotal evidence that they are improving tree health though passive irrigation.

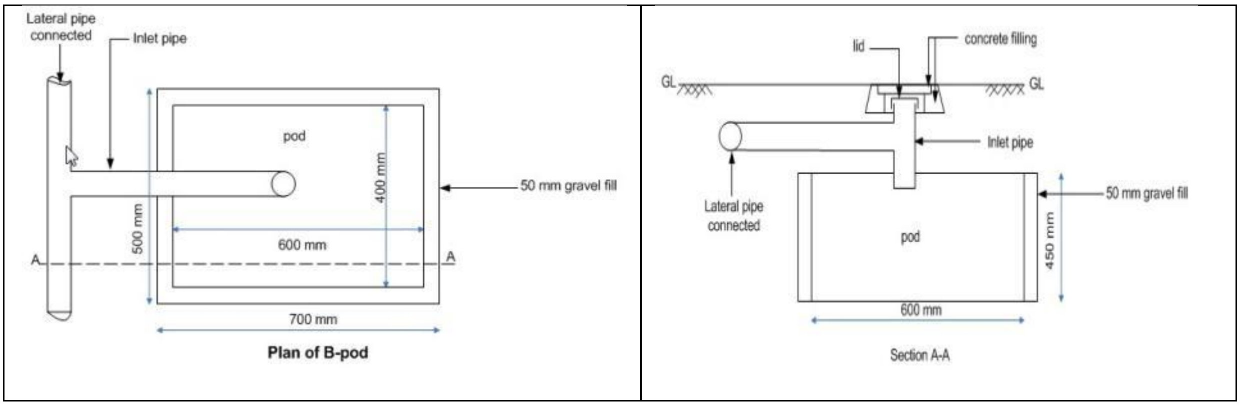


Figure 4-1 – Plan and section view of a typical B-Pod installation

B-Pods are typically installed in conjunction with road works to reduce the cost of B-Pod construction. The strategy for B-Pod placement during road design is to place B-Pods in locations where they may be situated in line with the stormwater drainage outlet of homes, on council owned land between the road and footpath, and where these sites are within a few metres of an existing or planned street tree.

4.2 Aims

To examine the impact of City of Burnside B-Pods on peak flow and runoff volume on a typical street in Adelaide.

4.3 Methodology

To examine the impact of B-Pods on peak flows and runoff volumes, the eight steps in Section 3.4 were followed. Site selection (Step 1) involved consultation with representatives from City of Burnside, and the recent installation of B-Pods in Union Street, Dulwich was selected as a case study. Background information on this location is provided in Section 4.3.1. The computer software for the analysis (Step 2) was PCSWMM, selected in accordance with Section 3.4.2. The model used to estimate runoff and peak flow rates was assembled based on the catchment data and assumptions outlined in Section 4.3.2. There were no data available for model calibration and verification (Step 4). The scenarios used to assess the performance of B-pods (Step 5) are described in Section 4.3.3. The long term continuous rainfall data applied in hydrological modelling (Step 6) was the 19 years of data from Parafield Airport, as described in Section 3.4.6. The estimation of peak flow and runoff volume (Step 7) was conducted in accordance with the procedures in Section 3.4.7. The results of the analysis (Step 8) are presented and compared in Section 4.4.

4.3.1 Site selection

To examine the impact of B-Pods on peak flows and runoff volumes, the Union Street catchment in Dulwich was selected in consultation with representatives from the City of Burnside, as typical of current B-pod installations. The Union Street catchment is located approximately two kilometres east of the Adelaide CBD. The road and kerb in Union Street was upgraded in 2010, with the inclusion of 38 B-Pods at certain locations. The location of the Union Street catchment is shown in Figure 4-2, indicating existing drainage and B-pod locations. Despite the location of a drain along the Eastern end of the street, there were no stormwater inlets noted along the entirety of Union Street, except at the intersection with Stuart Road.

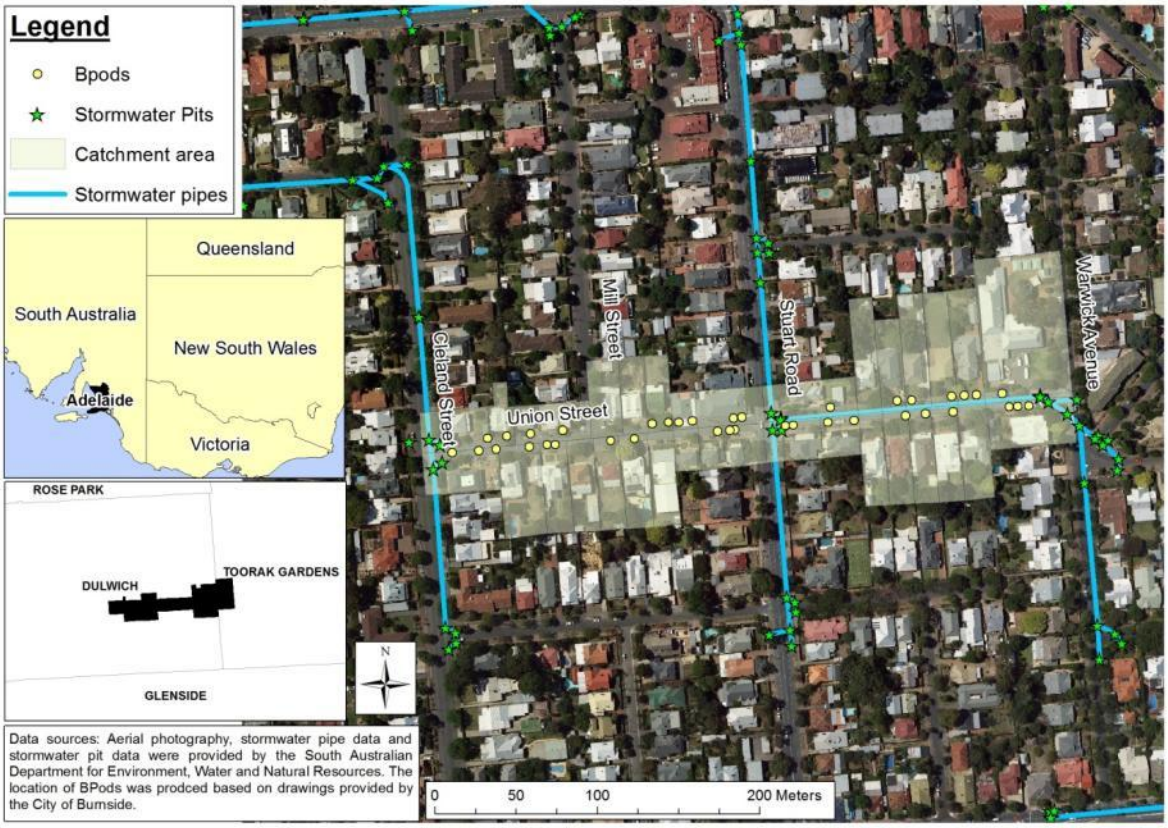


Figure 4-2 – Location of the Union Street, Dulwich catchment used in this analysis, indicating nearby roads and B-Pod locations

The catchment selected was wholly urbanised with residential development, including some redeveloped sites. Drainage of Union Street was separated by stormwater pits at the intersection of Stuart Road. Based on this, the 3.4 Ha catchment was assumed to consist of two separate sections:

- Section 1, Union Street East – Catchment area from Warwick Avenue at the east of the catchment to the intersection of Stuart Road and Union Street
- Section 2, Union Street West – Catchment area from Stuart Road at the centre of the catchment to the intersection of Cleland Avenue and Union Street

The site has a slope of approximately 1.2% from east to west. The drainage system shown in the diagram generally continues in a north westerly direction, eventually entering the River Torrens at a point north east of the CBD.

4.3.2 Model Assembly

Previous Modelling

Previous modelling of the B-Pods catchment along Union Street was conducted using MUSIC as part of the Goyder Institute WSUD research project (Tjandraatmadja et al., 2014). MUSIC was used to explore the potential of the B-Pods to reduce runoff volume and, in doing so, reduce the load of stormwater pollution heading downstream. As part of this process, the catchment area of the Union

Street B-pods was assumed to consist only of the road and surrounding residential allotments. This was considered appropriate as the aim of the analysis is to examine the impact of B-Pods on runoff from a single residential street. This same catchment data was adopted for the peak flow modelling.

Catchment Characteristics and Modelling Data

The 3.4 Ha catchment was simulated by treating each allotment as a subcatchment based on cadastre data from the City of Burnside. Some dual occupancy allotments were lumped into a single subcatchment. In addition, the north and south side of the road carriage way was also treated separately. The catchment area and the proportions of directly connected impervious area, indirectly connected impervious are and pervious areas within each catchment were estimated by GIS analysis of an aerial photo, and site inspection to alleviate uncertainties. The results of this process are shown as an aggregate for the entire catchment in Table 4-1. Catchment surface storage properties were assumed based on the calibrated Frederick Street model. According to the City of Burnside, the native soil type in the area was clay. Pervious area parameters in the model were therefore were adopted based on Horton infiltration parameters from O'Loughlin and Stack (2012) for a Type 4 soil.

Table 4-1 – Summary of catchment properties for the Union Street catchment

Catchment property	**Value**
Total area (Ha)	3.4
Directly connected impervious area (%)	44.4
Indirectly connected impervious area (%)	9.9
Pervious area (%)	45.7
Number of subcatchments	33
Impervious area storage (mm)*	1.0
Pervious area storage (mm)*	5.0
Soil type	Clay
Maximum infiltration (mm/hr)	75.0
Minimum infiltration (mm/hr)	2.0
Decay constant (h^{-1})	2.0
Drying time (days)	5.0
** Data assumed based on Frederick Street model*	

B-pod Characteristics

The properties and storage capacity of B-Pods were determined based on the procedures outlined in the post implementation assessment of B-Pods presented by Tjandraatmadja et al. (2014). The dimension of each pod was assumed based on construction drawings from the City of Burnside. A site inspection revealed that some B-Pods may have a much larger excavation than that indicated on construction drawings, and this was investigated by conducting a sensitivity analysis of storage volume (see Section 4.3.3). The properties of each B-Pod were applied singularly or in pairs (as appropriate) to the treated subcatchments in PCSWMM. The properties of the B-Pods applied in PCSWMM are provided in Table 4-2. It should be noted that the B-pods were simulated as infiltration systems in PCSWMM. As such, some properties, such as surface storage, were assumed to be present to ensure that the system operated effectively without significant overflow occurring due to insufficient surface storage. The underlying storage was reduced accordingly such that the net impact was equivalent to a B-Pod.

Table 4-2 – Properties common to all B-Pods in the Union Street catchment

Surface storage properties	**Value**
Surface storage depth (mm)	250
Vegetation volume	0
Soil storage properties	
Thickness (mm)	250
Porosity (volume fraction) [1]	0.9
Conductivity (mm/hr)[2]	Heavy Clay - 0.36 Sandy clay - 3.6 Sand - 360
Underdrain properties[3]	
Drain coefficient (mm/hr)	0
Drain exponent	0
Drain offset (mm)	0
LID Properties	
No. Units	No. B-pods in catchment
Area of each unit (m^2)	0.35 (standard size, volume = 158 L) 2 (larger size, volume = 700 L)
Percentage of impervious area treated	Percentage of impervious area of each subcatchment represented by roofs

[1] Value adopted as representative. There is no media inside a B-Pod device, but a porosity < 1 must be assumed in the model.
[2] Based on infiltration properties provided by eWater (2012)
[3] There is no underdrain present
[4] LID properties refers to 'low impact development properties', a US term that essentially refers to a WSUD treatment system

Other Modelling Assumptions

- In the absence of any flow data for the Union Street catchment, there was no reliable means of calibrating the Union Street hydraulic model.
- It was assumed that inflows to the Union Street site from upstream do not influence the effectiveness of the B-Pods. This is considered reasonable because there are drainage pits at the Eastern end of Union Street intercepting this flow. Furthermore, the purposes of the analysis was to provide a comparison of the peak flow characteristics within the catchment boundary both with and without B-Pods installed – in this case, any flow entering the catchment is consistent in both cases.
- Soil properties of the catchment surface have been adopted based on the calibrated Frederick Street model. This was not considered a critical assumption because (a) the soil properties will be the same for the sake of comparing scenarios and (b) key infiltration parameters from B-Pods, which determine the emptying time of the storage, was a separate

parameter and was subjected to sensitivity analysis (see Section 4.3.3). As a result of this, B-Pods with high and low infiltration are still directly comparable because the contributing runoff volume from the catchment remains the same – only the ability to dispose of water retained by BPods was different.

- Overflows from B-Pods, which occur when the storage volume reaches capacity, are assumed to proceed immediately to the gutter outlet. Any runoff that is not from roof surfaces also drains in this manner.

4.3.3 Modelling Scenarios

There were two main scenarios compared to investigate the impacts of B-Pods on peak flow in Union Street. These were:

1. Scenario A: No B-Pods – represents the Union Street catchment without WSUD devices. All flow continues down the street to the nearest intersection. Flow is divided into two components – Union Street East and Union Street West
2. Scenario B1: With B-Pods – represents the Union Street catchment as built, with 38 B-Pods situated as shown in Figure 4-2.

Additional scenarios based on Scenario B1 were also assessed, and are summarised in Table 4-3. These additional scenarios (scenarios B2 to B6) were conducted as a sensitivity analysis of the known properties of the catchment which affect B-Pod performance, namely underlying soil conditions and B-Pod excavation volume. Soil conditions were subject to sensitivity analysis because soil types vary widely across greater Adelaide and it was considered important to assess the impact that B-pods might have in different soil environments. The B-Pod size was studied because a field study of the B-Pod storage volume indicated that the storage volume may be higher than the design volume. The field study was undertaken by pouring known volumes of water continuously into a single B-Pod. Despite the design volume of approximately 159 L for each B-pod unit, the installed volume was found to be in excess of 700 L.

The size difference was attributed to construction methodology. According to the City of Burnside, construction contractors tend to simplify B-Pod construction by over-excavating native clay soil for the B-pod storage volume and backfill with aggregate material, which effectively increases the storage volume of the B-pod. The effect of this larger storage volume was also assessed, by assuming a 700 L B-Pod storage volume.

Table 4-3 – Scenarios for Case Study 1 - B-Pods in Union Street

Scenario	Description
CS1-A	No B-Pods
CS1-B1	With B-pods, standard volume, clay sub-soil
CS1-B2	With B-pods, standard volume, sand-clay sub-soil
CS1-B3	With B-pods, standard volume, sandy sub-soil
CS1-B4	With B-pods, larger volume (700 L), clay sub-soil
CS1-B5	With B-pods, larger volume (700 L), sand-clay sub-soil
CS1-B6	With B-pods, larger volume (700 L), sand sub-soil

4.4 Results

Results indicating the effectiveness of the B-pods for reducing peak flow and runoff volume in the Union Street catchment, and a sensitivity analysis of the assumed B-pod properties, are presented in the following sections.

4.4.1 The Impact of B-Pods in the Union Street Catchment

The reduction in peak flow rates attributed to the presence of B-Pods on the peak flow rate and volume of runoff in the Eastern and Western sections of Union Street, Dulwich, are presented in Table 4-4. These peak flow rate data are shown as percentage differences in Figure 4-3.

The results indicate that the B-Pods reduce the total runoff volume from the two catchments by approximately 1%. The differences in the estimated peak flow rates with and without B-Pods are small, but there is a reduction in peak flows achieved by applying B-Pods. The impact of the B-Pods on peak flows appears to be higher for more frequent events (1 year ARI flow rates) and decreases as the event becomes less frequent.

Table 4-4 – Peak flow and mean annual runoff volume from Union Street East and West with and without B-Pods

		Average Recurrence Interval (ARI) Flow (L/s)				Mean Annual runoff (kL)
Scenario		**0.5**	**1**	**2**	**5**	
No B-Pod	Union St West	31	59	87	125	3746
	Union St East	33	61	89	127	4120
	Sum	-	-	-	-	**7866**
B-Pod	Union St West	30	58	85	122	3703
	Union St East	28	57	86	124	4086
	Sum	**-**	**-**	**-**	**-**	**7788**

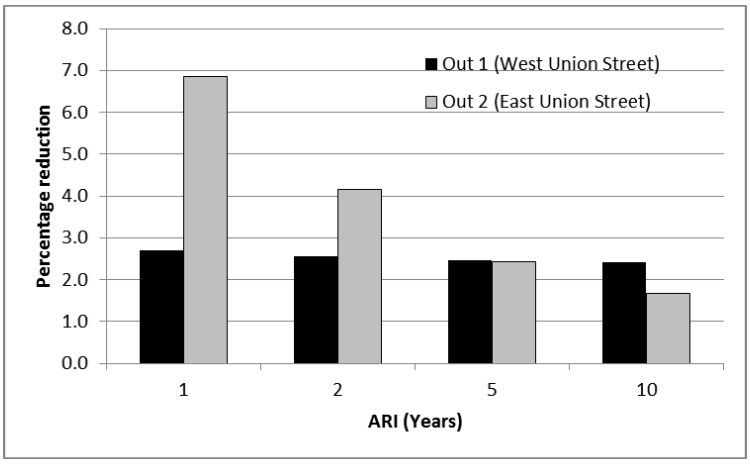


Figure 4-3 – Percentage reduction in untreated catchment peak flow rates attributable to the 158 L B-Pod design

4.4.2 Sensitivity Analysis of B-Pod Performance

Sensitivity analysis was undertaken to investigate the effect of subsoil type and B-Pod volume on the B-Pod performance.

Effect of Soil Type

The impact of underlying soil type on runoff volume is shown in Table 4-5. The data illustrates that well-draining soils can reduce mean annual runoff volume by up to 18% compared to the catchment without B-Pods, but that these values are more modest for clay based soils.

Table 4-5 – Impact of B-Pod subsoil on the runoff volume reduction

Scenario	Annual runoff (kL)	Reduction (%)
No B-Pod	7866	-
B-Pod – Clay	7788	1.0
B-Pod – Sandy Clay	7230	8.1
B-Pod – Sand	6397	**18.7**

The results in Figure 4-4 compare the reduction achieved by installing standard size (159 L) B-pods in environments in a clay, sandy clay or sand soil environment. Results are presented as a percentage reduction with respect to catchment without B-Pods. The results in Figure 4-4 are for Union Street West. A similar relationship exists for Union Street East (data not shown). The results indicate a higher reduction in peak flow rates, compared to the no B-Pod scenario, when more freely draining soils are present. As shown in Table 4-2, the increase in soil drainage for the clay, sandy clay and sand soil is by a factor of 100 for each scenario, however this factor did not linearly translate to the outcomes for peak flow improvement.

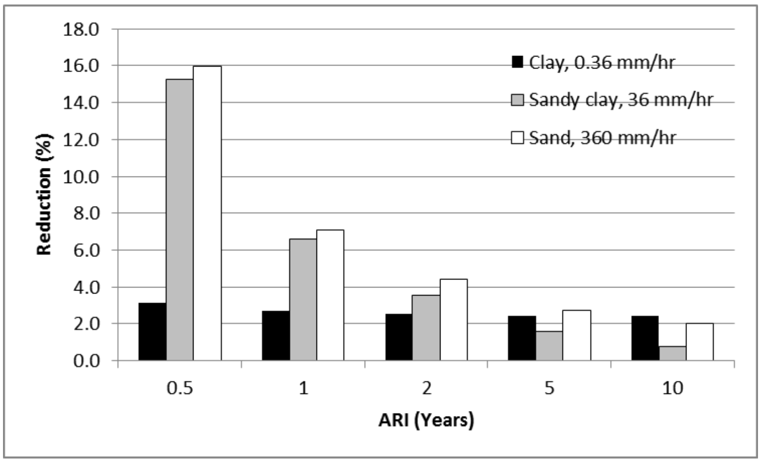


Figure 4-4 – Impact of underlying soil type on the effectiveness of the B-pods for reducing peak flow rates of the no B-Pod case, Union Street (West)

It should be noted that uncertainty tends to increase with the prediction of lower frequency flow rates, and it is suspected that this is the cause of deviations in the effectiveness at the 5-year and 10-year ARI flow rate where it appears the clay substrate is marginally better than sandy clay. This is attributable to the linear plot between ARI and flow assumed for the partial series, which was adopted as a precise but not necessarily accurate estimation of the ARI.

Effect of B-Pod volume

The impact of B-Pod volume on total catchment runoff volume is shown in Table 4-6. The results suggest that increasing the B-Pod volume has little impact on runoff volume reduction, despite the increased interception volume and area over which infiltration may occur. However, when the larger system is applied over a sand soil, up to 25% of annual runoff volume is retained by the B-Pods, due to the influence of higher infiltration rates over a larger area combined.

Table 4-6 – Impact of B-Pod storage volume on the runoff volume reduction

Scenario	Annual runoff (kL)	Reduction (%)
No B-Pod	7866	-
B-Pod – Standard (158 L), clay	7788	1.0
B-Pod – Larger (700 L), clay	7778	1.1
B-Pod – Larger (700 L), sand-clay	6667	15.2
B-Pod – Larger (700 L), sand	5919	24.7

The effect of the B-Pod volume on the peak flow reduction is shown as a percentage reduction compared to the no-B-Pod scenario in Figure 4-5 for Union Street West. A similar relationship exists for Union Street East. There is a higher reduction in peak flow rates, compared to the no B-Pod scenario, when larger B-Pods are constructed. However, the impact is quite small on percentage flow reductions.

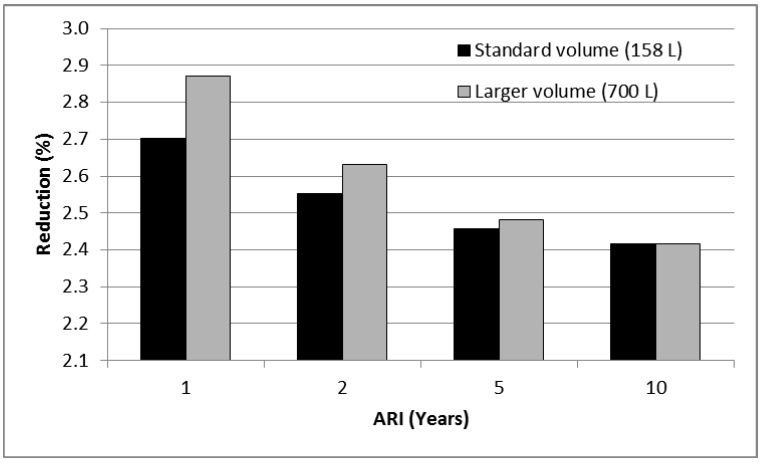


Figure 4-5 – Percentage reduction in peak flow rates without B-Pods by applying standard and larger size B-Pod volumes, Union Street West

4.5 Discussion

The results indicated that B-Pods had a minor impact on peak flow rates and runoff volumes in the design conditions. For example, 1 year ARI peak flows were reduced by between 2.7% and 6.9%. The reduction was lower for less frequent (higher ARI) events. The total runoff volume of each catchment was reduced by approximately 1%, but this was because of the dense clay subgrade which restricted infiltration in the area they were applied.

The performance of the B-Pods was better in soils where the infiltration rate was less restrictive than clay. For example in a sand soil the mean annual runoff was reduced by 18.7%, and the 1 year ARI peak flow rate was reduced by between 7.1% and 9.2%. However, providing a larger B-Pod storage volume had little impact on the effective reduction in flow rates and runoff volume, except where a very high soil infiltration rate was assumed and the larger surface area for infiltration took notable effect. For example, B-Pods achieved a mean annual runoff volume reduction of 24.7%, and a reduction in the 1 year ARI peak flow of 13.3% and 11.1% when larger systems were applied in a sand environment. It is recommended that where larger systems are considered, the possibility of connecting greater impervious area also be considered, as this may improve overall impact of peak flow rates.

It should be noted that the modest reductions in peak flow rates and volume reductions in the clay soils in Union Street should not be considered to illustrate a poor performance overall. The primary intention of the B-Pods was to provide a means of irrigating street trees which were previously irrigated by truck or not irrigated at all. Peak flow and runoff volume reductions were not a major consideration in design or implementation. It is recommended that B-Pods are explored further in terms of their contribution to the overall realm of WSUD by considering these findings for peak flow in addition to their mains water demand reduction. A cost benefit analysis should also consider the cost of implementation and the savings associated with avoiding or reducing street tree watering with a water truck. The implementation of these kerb side irrigation measures also presents an opportunity to explore the impact of this infiltration measure on the integrity of road and kerb infrastructure, which remains a concern for infrastructure design engineers (Tjandraatmadja et at., 2014).

4.6 Summary

B-Pods are 158 L kerb side retention systems implemented by City of Burnside to intercept roof runoff. The effectiveness of applying B-Pods to a 3.4 Ha urban catchment in Union Street, Dulwich Hill was examined based on comparing the simulated mean annual runoff volume and peak flow rate of the current catchment with and without B-Pods. The results indicated that B-Pods had a minor impact on peak flow rates and runoff volumes in the design conditions, mainly because of the clay subgrade which restricted infiltration. The impact of applying B-Pods in different soils, or with larger storages (700 L) was also examined. The performance of the B-Pods was better in soils where the infiltration rate was less restrictive than clay, but increasing the storage size had little improvement without increasing the potential rate of infiltration into soil.

5 Case Study 2 – Rain Gardens in Tarragon Street, Mile End

5.1 Introduction

In recent years, rain gardens have been installed by the City of West Torrens throughout the local government area in conjunction with road upgrade works. There were more than 90 rain gardens installed throughout the local government area at time of publication, with a significant proportion of these in the suburb of Mile End, an inner western suburb of Adelaide. The design of the rain gardens was based on the guidelines from Melbourne Water (2005) and FAWB (2009). A typical plan and cross section of these rain gardens is shown in Figure 5-1. This case study examines the effectiveness of the rain gardens for flow management in a typical street setting in Mile End. Tarragon Street was selected as a representative street which included rain gardens in recent road upgrade works. A photograph of a typical raingarden arrangement in Tarragon Street, Mile End, is shown in Figure 5-2.

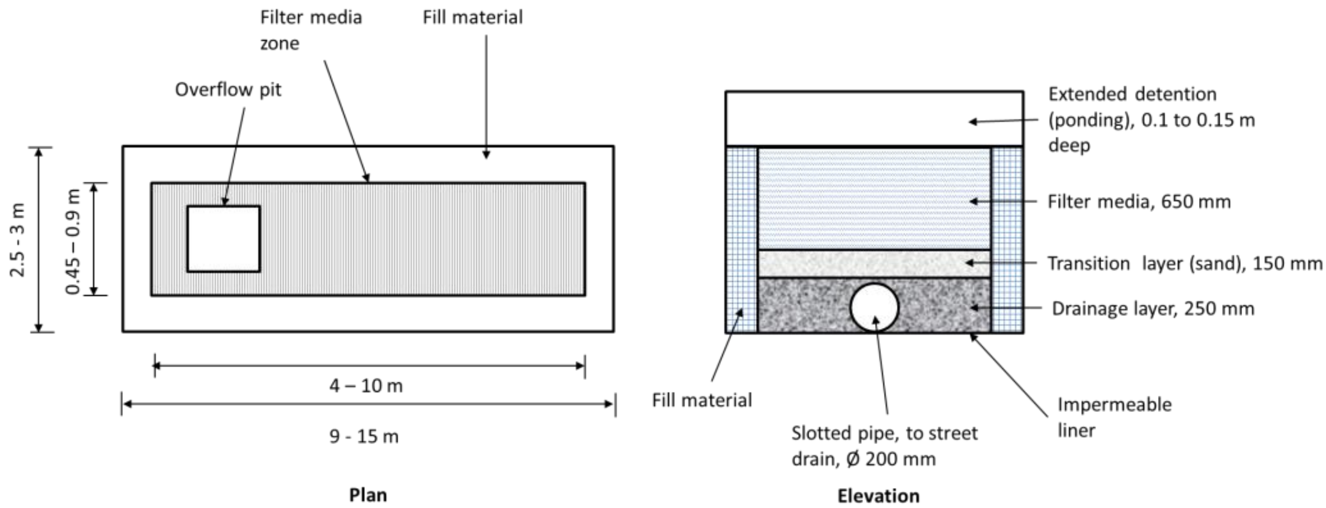


Figure 5-1 – Plan and elevation view of rain gardens in Mile end (adapted from construction drawings from City of West Torrens)

Figure 5-2 – Photograph of typical bioretention system arrangement in Tarragon Street, Mile End

5.2 Aims

To examine the impact of rain gardens on peak flows and runoff volumes on a typical street scale catchment in Adelaide.

5.3 Methodology

To examine the impact of the rain gardens on peak flows and runoff volumes, the eight steps in Section 3.4 were followed. Site selection (Step 1) involved site inspections and consultation with council and the subsequent selection of Tarragon Street, Mile End, with background information about the catchment provided in Section 5.3.1. The computer software for the analysis (Step 2) was PCSWMM, which was selected for reasons described in Section 3.4.2. A model of the catchment was assembled based on the data and assumptions outlined in Section 5.3.2. There were no data available for model calibration and verification (Step 4). The scenarios used to assess the performance of rain gardens (Step 5) are described in Section 5.3.3. The long term continuous rainfall data applied in hydrological modelling (Step 6) was the 19 years of data from Parafield airport described in Section 3.4.6. Peak flow and runoff volume analysis of simulated flows (Step 7) were conducted in accordance with the procedures in Section 3.4.7. The results of the analysis (Step 8) are presented and compared in Section 5.4.

5.3.1 Site Selection

To examine the impact of rain gardens on peak flows in a typical suburban street, six gardens were selected from a section of Tarragon Street, Mile End between Bagot Avenue and Ebor Avenue. The location of the rain gardens selected for this case study is shown in Figure 4-2.

Figure 5-3 – Location of the Mile End rain garden catchment used in this analysis, indicating selected roads

The 2.3 Ha catchment selected is wholly urbanised with residential development which originally proceeded with the construction of “workman’s homes” in the late 19th and early 20th century. While many of these original homes are still present in the area, there has been redevelopment of several allotments since this time with newer housing or infill development. The drainage system in the catchment is also shown in Figure 5-3. The drainage system continues to the West, eventually proceeding to the Adelaide airport drain, the Patawalonga Lake and, finally, Gulf St Vincent.

5.3.2 Model Assembly

Previous modelling

Previous modelling of the Mile End rain garden catchment was conducted using MUSIC as part of the post implementation assessment reported by Task 1 of this project (Tjandraatmadja et al., 2014). MUSIC was used to explore the potential of the rain gardens to provide water quality treatment using the default algorithms for bioretention systems and the characteristics of the systems installed. During this study, the catchment was assumed to consist of the roadway and surrounding allotments.

Catchment Characteristics and Modelling Data

The 2.3 Ha catchment selected for this study was assumed to consist of Tarragon Street, Mile End between Bagot Avenue and Ebor Avenue including the immediately surrounding residential allotments. The site has a reasonably flat topography, with a total slope of approximately 0.2%. There were six rain gardens installed along the road as indicated in Figure 5-3. The proportion of directly connected impervious area, indirectly connected impervious area and pervious areas in the catchment were estimated using site inspection and GIS analysis of an aerial photo with the results shown in Table 4-1. The loss parameters of the catchment were assumed based on typical values for urban surfaces recommended by Rossman (2010) and the calibrated values found for the Frederick Street model.

Table 5-1 – Summary of catchment properties for the Tarragon Street catchment

Catchment property	Value
Total area (Ha)	2.3
Directly connected impervious area (%)	44.0
Indirectly connected impervious area (%)	9.1
Pervious area (%)	46.9
Impervious area storage (mm)*	1.0
Pervious area storage (mm)*	5.0
Number of subcatchments	6
Soil type	Clay
Maximum infiltration (mm/hr)	75.0
Minimum infiltration (mm/hr)	2.0
Decay constant (h^{-1})	2.0
Drying time (days)	5.0
* Assumed based on the Frederick Street model	

The number of subcatchments was based on the number of gardens. As such, allotments were grouped for this analysis to reduce computation time. Unlike the B-Pod units in Case Study 1 (Section 4), it was not necessary to simulate individual allotments because the gardens each treat runoff from multiple allotments and adjacent roads.

Rain Garden Characteristics

Each rain garden was given a reference name, shown in Figure 5-3. The surface properties of each rain garden were measured during a site inspection in April 2013. The subsurface properties, including soil filter dimensions, were collected from as-built construction drawings provided by the City of West Torrens. The properties of each rain garden were applied accordingly to each of the six rain garden catchments in PCSWMM suing the bioretention simulation tool. The area properties of each garden are presented in Table 5-2. PCSWMM data entry was based on these properties. Additional properties shared by all rain gardens were applied based on the data in Table 5-3.

Table 5-2 – Surface area properties of the six rain gardens in the Tarragon Street model

	RG1	RG2	RG3	RG4	RG5	RG6
Surface area (m^2)	33.9	29.5	27.9	33.3	45	37.5
Filter Area (m^2)	5.12	3.83	3.52	4.50	6.72	4.41
Storage depth (mm)	175	125	175	125	175	125

Table 5-3 – Properties common to all rain gardens in the Tarragon Street model

Surface storage properties	
Vegetation volume	0
Soil storage properties	
Thickness (mm)	850
Porosity (volume fraction) *	0.44
Field capacity (volume fraction) *	0.062
Wilting point (volume fraction) *	0.024
Conductivity (mm/hr) *	150
Conductivity slope*	5
Suction head*	1.93
Underground storage properties	
Height (mm)	250
Void ratio#	0.1
Conductivity (of soil at base, mm/hr)	0
Underdrain properties^	
Drain coefficient (mm/hr)	18.5
Drain exponent	0.51
Drain offset (mm)	50

* Based on the typical values for a sandy soil as provided by Rossman (2010)
\# Void ratio is based on the void ratio of combined gravel and drainage pipe volume
^ Underdrain properties based on dimensions of raingarden RG5, considered a highly efficient drainage rate and therefore conservative for this analysis

Other Assumptions

- In the absence of any flow data for the Mile End catchment, there was no reliable means of calibrating the model. As such, until flow data is made available for the system, it was assumed that the parameters for the model are a true and accurate representation of the site conditions.
- It was assumed that inflows to the Tarragon Street catchment from upstream do not influence the effectiveness of the rain gardens in Tarragon Street. This was considered a reasonable assumption because the purpose of the analysis were to provide a comparison of the peak flow characteristics of the catchment without rain gardens and with rain gardens installed, independent of any upstream or downstream impacts on flow.
- When rain gardens were simulated by PCSWMM, all garden overflows were conservatively assumed to proceed immediately to the overflow pit and into the subsurface drainage system without detention.

5.3.3 Model Scenarios

There were two main alternative scenarios compared to investigate the impacts of rain gardens on peak flow in the Tarragon Street catchment. These were:

- Scenario A: No rain gardens – represents the catchment without WSUD devices, but with side entry pits installed where rain gardens have been situated

- Scenario B1: Rain gardens – represents the catchment as-built, with six rain gardens situated as shown in Figure 5-3 and with the properties described in Section 5.3.2.

Additional scenarios based on each of these two main scenarios were assessed, and all scenarios are summarised in Table 5-4. These additional scenarios (scenarios B2 to B4) were conducted as a sensitivity analysis of the presence of an impermeable liner at the base of the filter media. High levels of retention were not anticipated from the as-built rain gardens in Scenario B1 because the impermeable liner material restricts on site infiltration, and Scenarios B2 to B4 were used to explore the impact of infiltration through the base of the system (assuming an impermeable liner was still placed on the filter walls). The infiltration rate into underlying soil was also subject to sensitivity analysis. Soil conditions were subject to sensitivity analysis because soil types vary widely across greater Adelaide and it was considered important to assess the impact that rain gardens might have in different soil environments where infiltration may be possible.

Table 5-4 – Scenarios for Case Study 2 – Rain Gardens in Tarragon Street

Scenario	Description	Soil conductivity (mm/hr)
CS2-A	No rain gardens	0
CS2-B1	With 6 gardens, impermeable base (fully lined)	0
CS2-B2	With 6 gardens, permeable base, clay	0.36
CS2-B3	With 6 gardens, permeable base, clay-sand	3.6
CS2-B4	With 6 gardens, permeable base, sand	360

5.4 Results

The study findings indicating the effectiveness of the rain gardens for reducing peak flow and runoff volume in the Tarragon Street catchment, and a sensitivity analysis of the rain garden properties, are presented in the following sections.

5.4.1 The Impact of Rain Gardens in the Tarragon Street Catchment

The impact of the rain gardens on peak flow rates are illustrated in Figure 5-4.

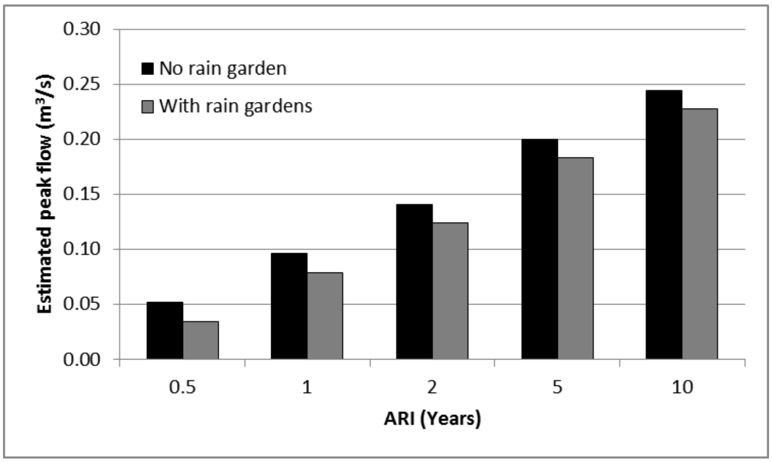


Figure 5-4 – Comparison of peak flow rates with and without rain gardens in the Tarragon Street catchment

The results in Figure 5-4 indicate that the rain gardens in Tarragon Street effectively reduce the peak flows in the catchment, particularly for more frequent events. For example, the estimated 6 month ARI peak flows were reduced by 39% by the rain gardens. This reduction was lower for the less frequent flows however, with reductions of 18%, 11% and 6% for the 1 year, 2 year and 5 year ARI peak flow estimates, respectively. In the case of these lined systems where infiltration does not occur at the base, this reduction in peak flows is attributable to the detention period of stormwater during ponding and filtration, at the surface of the gardens, and in the soil filter zone.

It should be noted that there was little reduction in the total volume of runoff. Over the entire 19 year period of simulation, there was only a 0.2% reduction in the total 63 ML runoff volume estimated for the catchment. This is because each of the rain gardens is impermeable, distributing all water to the drainage system beneath, and any retention attributed only to soil moisture evaporation between rainfall events.

5.4.1 The Impact of Soil infiltration on Filter Performance

The total volume of runoff from the catchment improved with greater levels of infiltration through the base of the rain gardens. Table 5-5 illustrates the mean annual runoff volume for the no infiltration case and the three assumed soil scenarios.

Table 5-5 – Impact of soil underlying rain gardens on the mean annual runoff from the catchment

Case	Description	Mean annual runoff (ML)	Mean annual runoff reduction (%)*
A	No Gardens	3.3	-
B1	Gardens, no infiltration	3.2	0.2
B2	Gardens, clay base	3.2	0.6
B3	Gardens, clay-sand base	3.1	3.4
B4	Gardens, Sand base	2.9	9.8
* Compared to Case A			

By allowing for infiltration to occur it was found that improvements in the peak flow of stormwater runoff were marginal. A comparison of the three soil types are shown in Figure 5-5. The peak flow reductions were similar to the values without infiltration.

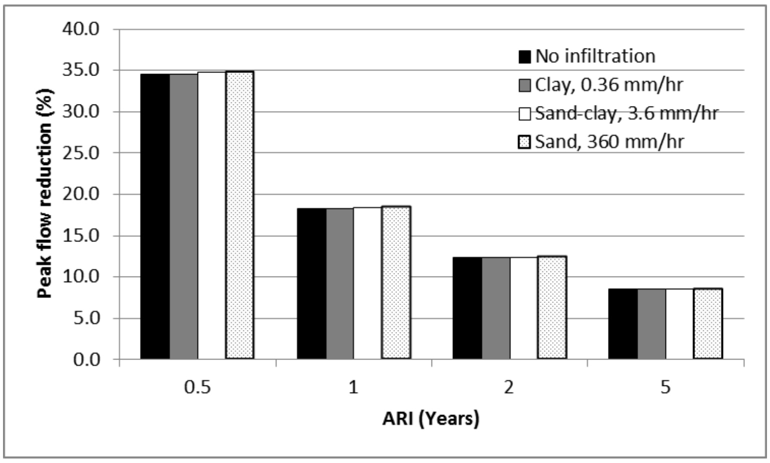


Figure 5-5 –Comparison of the reduction in peak flow achieved by having no infiltration, or infiltration via clay, sand-clay or sand at the base of the rain gardens

5.5 Discussion

The results indicated that the rain gardens currently implemented in Tarragon Street, Mile End, had little impact on the volume of runoff generated from the catchment because of the impermeable liner which was installed. However the rain gardens provided benefits in the form of detention which reduced the peak flow rate of runoff from the catchment area. The peak flow reductions were higher for more frequent events. For example the 0.5, 1, 2 and 5 year peak flows were reduced by 39%, 18%, 11% and 6% respectively.

The simulated removal of a liner at the base of the system to allow infiltration to occur produced improvements in the stormwater runoff volume of from the catchment, with between 0.6% and 10% reduction in runoff volume for a clay and sand soil environment, respectively (assuming an underdrain was still present). However, allowing for infiltration at the base of the system had negligible impact on the peak flow rate even when well-draining soils were assumed to be present.

Overall, the results indicated that rain gardens present a promising technology for reducing peak flow rates and to some extent runoff volume from urban catchments. Further research is suggested to explore the optimal surface storage and underground storage arrangement to maximise the potential of each system to reduce peak flows. It should also be noted that the performance of the rain gardens as a WSUD device should also consider their potential to improve water quality, which was explored by Tjandraatmadja et al (2014). To further explore the benefits of rain gardens for flow and water quality improvement, it is recommended that field research is undertaken on the rain garden design to verify that the simulated flow and water quality outcomes are occurring with the selected design, soil media and vegetation in a South Australian climate. Research should also be undertaken to fully quantify the costs and benefits of rain gardens in the streetscape, both in terms of tangible and intangible costs and benefits.

5.6 Summary

The effectiveness of applying street scale rain gardens to a 2.3 Ha urban street catchment in Mile End was examined. The examination was based on comparing the simulated mean annual runoff volume and peak flow rates from the catchment with and without rain gardens. The results indicated that the rain gardens had little impact on the volume of runoff generated from the catchment. However, the rain gardens provided benefits in the form of detention which reduced the peak flow rate of runoff from the catchment area. The peak flow reductions were higher for more frequent events. The removal of an impermeable liner at the base of the rain garden to allow infiltration to occur from the system produced improvements in the stormwater runoff volume. However, allowing for infiltration at the base of the system had negligible impact on the peak flow rate even when well-draining soils were assumed present.

6 Case Study 3 – Frederick Street Catchment

6.1 Introduction

There are several areas in the Adelaide metropolitan area where standard of the underlying drainage system has been reduced by infill development for the reasons described in Section 3.1. The 44.7 Ha Frederick Street catchment, illustrated in Figure 6-1 is one such area subjected to redevelopment. Intensive flow and land use monitoring in the catchment in the early 1990s provide an opportunity to explore the impact of infill development in this catchment over a 20 year period.

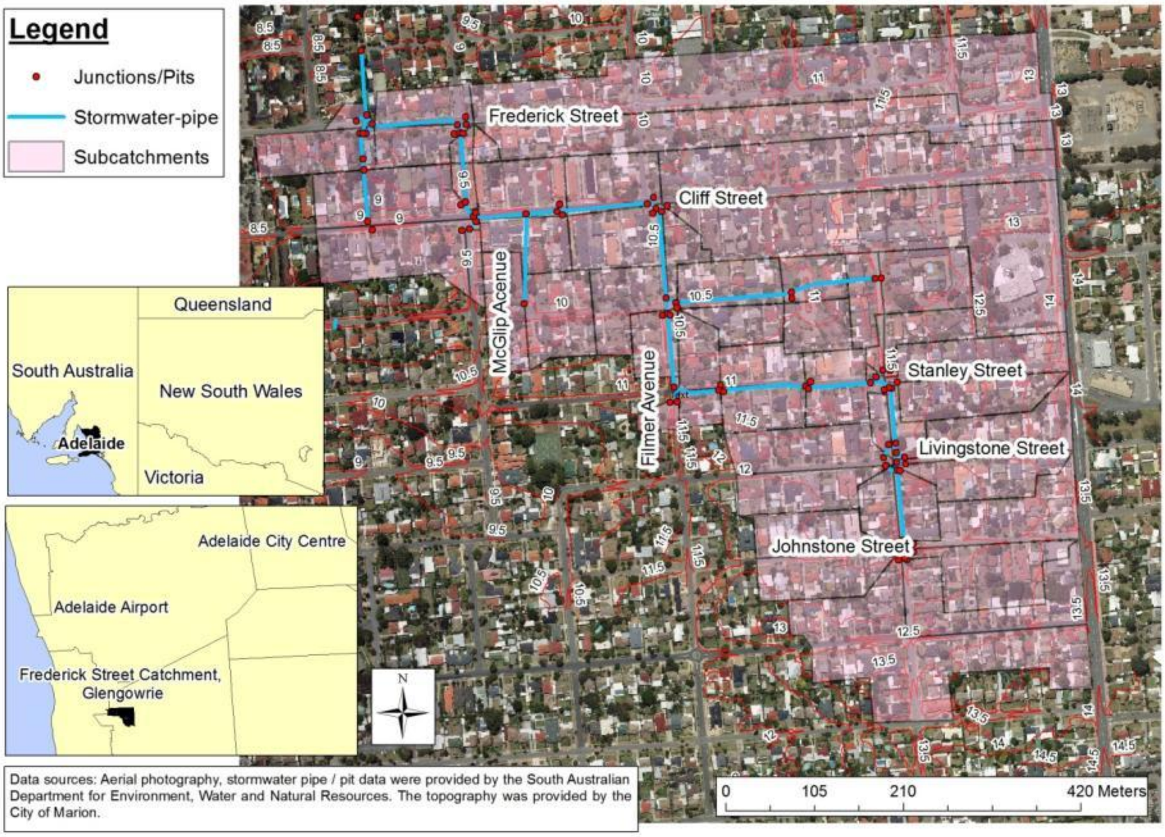


Figure 6-1 – Location of the Drain-18 catchment, indicating selected roads

Development is known to have occurred in the Frederick Street catchment area, mainly by the redevelopment of individual allotments from a single dwelling to multiple units, or an increase in dwelling size. Redevelopment was evident based on a comparison of aerial photographs from 1993 and 2013. For example, redevelopment of several blocks has occurred on the corner of Filmer Avenue and Cliff Street, as shown in Figure 6-2. For this reason, the Frederick Street catchment represents an ideal opportunity to explore the effects of infill development on runoff flow rate and volume, by comparing peak flow and runoff volumes before and after the development. The same simulation techniques can also be applied to explore the potential of on-site and distributed WSUD systems to manage the change in runoff flow rate and volumes.

Figure 6-2 – Examples of redevelopment on the Corner of Cliff Street and Filmer Avenue in the Frederick Street catchment

6.2 Aims

There were two aims to the infill development case study of the Frederick Street catchment:

- To estimate the impact of infill development on a medium sized catchment
- To explore the opportunities to overcome these impacts with WSUD tools

6.3 Methodology

To examine the impact of the increased development density on peak flow, flooding and runoff volume from the Frederick Street catchment, and potential WSUD solutions to ameliorate this impact, the eight steps in Section 3.4 were followed. Site selection (Step 1) involved a review of available flow data in Greater Adelaide, with the selection of Frederick Street as a medium size, low slope catchment. Further information on the site selection and character is provided in Section 6.3.1. The computer software for the analysis (Step 2) was PCSWMM, selected in accordance with Section 3.4.2. The model was assembled based on the data and assumptions outlined in Section 6.3.2. The model was calibrated and verified in accordance with the procedures in Section 3.4.4, with the results of calibration and verification provided in Sections 6.3.3 and 6.3.4. The scenarios used to assess the impact of infill development on the catchment flow, and the potential for WSUD to overcome these changes (Step 5) are described in Section 6.3.6. This includes details of the measured and assumed development character up to 2040 in Section 6.3.7, and details on the assumed nature of potential WSUD solutions in Sections 6.3.8 (retention based systems) 6.3.9 (detention based systems) and 6.3.10 (street scale rain gardens). The long term continuous rainfall data applied in hydrological modelling (Step 6) was the 19 years of data from Parafield Airport described in Section 3.4.6. Peak flow and runoff volume analysis of simulated flows (Step 7) was conducted in accordance with the procedures in Section 3.4.7. The method used to assess the current levels of flooding in the catchment and ways this may be ameliorated using WSUD is described in Section 6.3.5. The results of the analysis (Step 8) are presented and compared in Section 6.4. A summary of results is provided in Section 6.5.

6.3.1 Site Selection

Flow and development character in the Frederick Street catchment were well characterised in 1992 to 1993 as part of the 'Q/Q project', a joint venture between the University of South Australia (UniSA) Urban Water Resources Centre, the UniSA School of Pharmacy and Medical Sciences, the Bureau of Meteorology and the SA Department of Transport. The Frederick Street catchment was selected as this information provided a good baseline from which current and future development could be extrapolated. The Frederick Street catchment also represents:

- a catchment size of 44.7 Ha, that results in a manageable model size,
- a reasonably flat topography, and
- one of very few sites in the greater Adelaide region where flow data was available for the production of a calibrated model for continuous simulation of an urbanised catchment

6.3.2 Model Assembly

Previous Modelling

Previous modelling of the Frederick Street catchment was undertaken by Kemp (2002) who used runoff flow data from the catchment to verify that the ILSAX model was suitable for simulating runoff in urban catchments. Pezzaniti (2003) undertook similar modelling using DRAINS and SWMM. The input data to these simulations were based on data collected in the Frederick Street catchment in and around 1992 as part of the original monitoring of the catchment by the 'Q/Q Group' (Bruce et al., 1994; Argue et al., 1994). The catchment contributing areas (pervious, directly connected impervious and indirectly connected impervious) for the ILSAX model were determined by analysis of aerial photography and on-site inspection by students from the University of South Australia.

Kemp (2002) found that the ILSAX model performed well for simulating the storms modelled, providing that the directly connected impervious area from the input data was reduced by 10% and added to indirectly connected impervious area. Other key parameters of the model discussed by Kemp (2002) are provided in Table 6-1.

Table 6-1 – Key properties of the Frederick Street catchment model developed by Kemp (2002)

Model parameter	Value
Impervious area depression storage (loss)	1 mm
Pervious area depression storage (loss)	5 mm
Impervious area roughness, N_{imp}	0.01 (no units)
Pervious area roughness, N_{perv}	0.03 (no units)
Catchment slope	0.1% to 0.5%
GUT Factor (a measure of gutter efficiency)	7.66
Soil infiltration rate – Initial	125 mm/h
Soil infiltration rate – Final	6 mm/h
Shape factor	2 h^{-1}

Catchment characteristics and Modelling Data

The location and drainage system in the Frederick Street catchment was previously shown in Figure 6-1. Other general characteristics of the Frederick Street catchment have been described previously by Kemp (2002). In brief, the 44.9 Ha catchment has been fully urbanised, with most development occurring in the 1940s and 1950s. The nature of development is almost completely residential with small areas of commercial land use. The underlying soils of the catchment are sandy to silty clays containing some lime. The catchment is relatively flat, with average gutter slopes from 0.2% to 0.5% (see contours on Figure 6-1). Table 6-2 summarises the characteristics of the catchment based on the 1993 data collection.

Table 6-2 – Summary of catchment properties for the Frederick Street catchment

Catchment property	Value
Total area (Ha)	44.9
Directly connected impervious area (%)	30.4
Indirectly connected impervious area (%)	17.1
Pervious area (%)	52.5

The layout of stormwater pipes, junctions and side entry pits was derived from current mapping data from the City of Marion and is also illustrated in Figure 6-1. It should be noted that there was some discrepancy in the data provided by the City of Marion and the modelling data available in the previous catchment model developed by Kemp (2002). The pit, pipe, surface level and slope data in the model from Kemp (2002) tended to differ from the data provided by the City of Marion, and may be a result of using different sources of information in model compilation. The City of Marion data was used as a primary reference for the purposes of this study. This is because it was considered to be most appropriate to combine with surface elevation data across the catchment also provided by City of Marion. The surface elevation data was not strictly required for an ILSAX model, but important in a SWMM model for the determination of surface slope and major flow paths (road surfaces).

Before beginning calibration of the SWMM model of the Frederick Street catchment, the parameters in Table 6-1 were used as a starting point. The catchment imperviousness and percentage of connected and indirectly connected impervious area was adopted based on the calibrated model of Kemp (2002), with 10% less directly connected area than that measured by photography and field inspection. There was no GUT factor in the SWMM model, however the GUT factor may be considered to be somewhat represented by the catchment width in SWMM. However the width parameter in SWMM does not allow the user to separate time of entry and gutter flow time. A full description of catchment width is provided by Rossman (2010). Briefly however, for the Frederick Street catchment, width was estimated based on the catchment area and the length of overland flow, which was assumed to be 25 m (based on the approximate distance of travel of rain from housing lots to gutters).

Climate and Flow Data

There were six rainfall and two flow monitoring stations initiated in 1992 as part of the 'Q/Q project', which were spread across the Drain-18 catchment. The properties of the gauges are shown in Table 6-3, and their locations with respect to the Frederick Street catchment are depicted in Figure 6-3 along with other nearby flow and rainfall monitoring stations. As shown, there are two rainfall gauges within the Fredrick Street catchment and one flow gauge at the catchment outlet: Frederick Street Drain at Glenelg (A5040561) and Morphett Arms Hotel Pluviometer at Glengowrie (A5040556). Based on their proximity to all parts of the catchment, these gauges were selected to represent rainfall in the Frederick Street catchment.

Table 6-3 – Description of flow and rainfall gauges in the vicinity of the Frederick Street catchment

Station	Number*	Location	Data available	Dates
Frederick Street drain at Glenelg	*AW504561* **A5040561**	138°31'47.3" E 34°59'05.6" S	Water level Velocity Rainfall	30/06/1992 – 06/11/1996 23/06/1993 – 06/11/1996 05/12/1991 – 24/05/2004
Maxwell Terrace at Glenelg Tramway	*AW504554* **A5040554**	138°31'35.8" E 34°58'47.6" S	Water level Velocity Rainfall	23/08/1993 – 05/03/1996 23/08/1993 – 10/07/1995 08/08/1990 – 16/01/2001
Glenelg Coles car park	*AW504565,* **A5040565**	138:30:54.7 E 34:58:43.7 S	Rainfall	01/02/1992 – 17/09/2001
Willoughby Park pluviometer at Sturt River	*AW504555* **A5040555**	138°32'11.4" E 34°58'48.4" S	Rainfall	09/08/1990 – 17/09/2001
Morphett Arms Hotel pluviometer at Glengowrie	*AW504556* **A5040556**	138°32'14.6" E 34°59'14.3" S	Rainfall	09/08/1990 – 31/05/2000
Women's Bowling Club pluviometer at Glenelg	*AW504557* **A5050557**	138°31'19.9" E 34°58'57.0" S	Rainfall	09/08/1990 – 17/09/2001
AWXXXXXX* indicates a now defunct site code which may be used in previous literature; **AXXXXXXX represents a site code current at time of writing				

Figure 6-3 – Location of gauging stations within and near the Frederick Street catchment

The nearest long-term climate station to the Frederick Street catchment is the Bureau of Meteorology station at Adelaide Airport (023034). At this station, the mean annual rainfall has been reported to be 446 mm. The mean monthly rainfall is shown in Figure 6-4. For comparison, the mean monthly rainfall of the Parafield Airport gauge is shown, indicating that the data from Parafield Airport provides a reasonable estimation of monthly rainfall averages.

The mean annual evaporation is approximately 1900 mm/annum. Mean monthly evaporation is shown in Figure 6-5. The mean monthly evaporation data from this gauge was used to simulate the effect of evaporation on catchment surface storages in the SWMM model for Frederick Street.

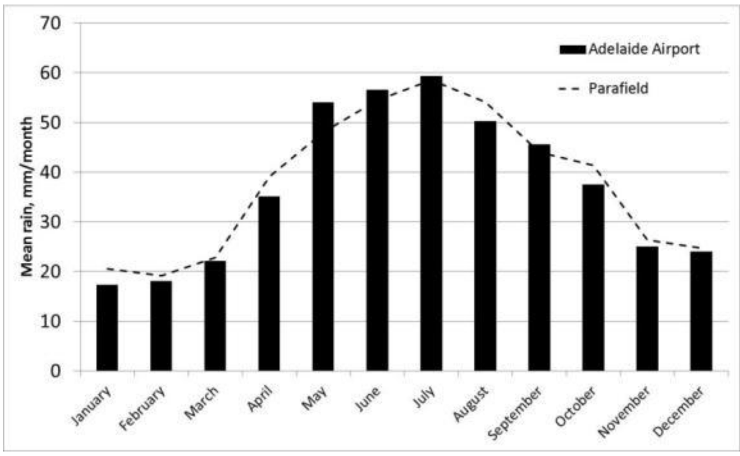


Figure 6-4 – Mean monthly rainfall at the Adelaide Airport BOM gauge (023034)

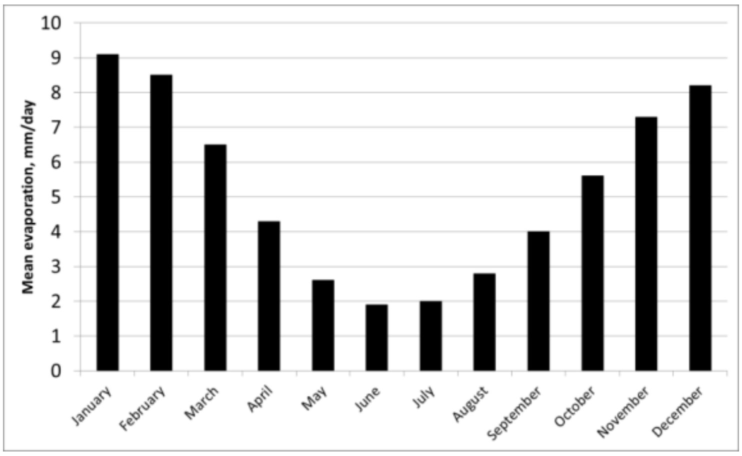


Figure 6-5 – Mean daily evaporation at the Adelaide Airport BOM gauge (023034)

Observed flow for the Frederick Street catchment was available from the Frederick Street Drain gauge described in Table 6-3. The quality of data from this flow gauge was assessed with respect to rainfall and found to be generally good.

Other Assumptions

- There was an additional pipe in data files from the SA Department of Environment, Water and Natural Resources which was not in the original catchment data from 1992/1993. The pipe heads south of the catchment area at the intersection of Filmer Avenue and Stanley Street which may indicate a larger catchment area has been created since the original data collection in 1992/1993 by the Q/Q group. This catchment area was ignored in this analysis because this pipe does not appear to have been present during the original monitoring beginning in 1992. Furthermore, a site inspection indicated that the inlets to this pipe were fully blocked with sediment. It is recommended that the condition of this pipe is inspected if further flow investigations are initiated at the Frederick Street drain.
- As noted previously, it was assumed that the data available from records at the City of Marion were a true and accurate representation of surface, pit and pipe levels (and thus, surface and overland flow slopes) in the catchment.

6.3.3 Model Calibration

Calibration of the Frederick Street catchment model was undertaken based on observed flow with emphasis on replicating the hydrograph (as opposed to long term volume), particularly peak flows. The fitness of the model was assessed using the methods outlined in Section 3.4.4. The initial catchment and pipe parameters were assumed based on the previous model by Kemp (2002) and by reference to recommended data from the SWMM manual (Rossman, 2010).

A detailed explanation of catchment parameters is provided by Rossman (2010). In summary, during calibration, adjustments were made to the assumed values of the following parameters, ensuring that values stayed within reasonable limits based on the known catchment characteristics and the recommendations of Rossman (2010):

- Manning's N values of impervious area (N Imperv)
- Manning's N value of pervious areas (N Perv)
- Manning's N value of pipes
- Catchment width (not included in ILSAX model)
- Horton Infiltration parameters:
    - Maximum infiltration rate
    - Minimum infiltration rate
    - Decay constant
    - Drying time (not included in ILSAX model)

For comparison with previous work, calibration events were selected from the events used by Kemp (2002) for calibration of the ILSAX model. To ensure that enough events were available for verification, only events from 1992 were used for calibration, which still provided seven calibration events across the summer and winter months. The characteristics of these events are shown in Table 6-4. Figure 6-6 shows a plot of the total observed flow and rainfall volume for each of the calibration and verification events used in the study.

Table 6-4 – Calibration events for the Frederick Street catchment model

#	Date	Time	Observed Peak Flow (m^3/s)	Rainfall (mm)		Observed runoff volume (m^3)
				A5040561	A5040556	
Cal1	03/07/1992	2340 to 0400	0.38	10.8	11.6	1629
Cal 2	11/07/1992	0324 to 0900	0.15	9.4	8.2	1191
Cal 3	19/07/1992	0418 to 0700	0.37	4.4	5.4	732.6
Cal 4	07/08/1992	1542 to 2000	0.35	9	8.8	1153
Cal 5	30/08/1992	0106 to 0630	1.24	22.2	24.4	3816
Cal 6	31/08/1992	1248 to 1530	0.40	5	6	744.5
Cal 7	18/12/1992	1642 to 0030	1.34	39.6	39.2	5983

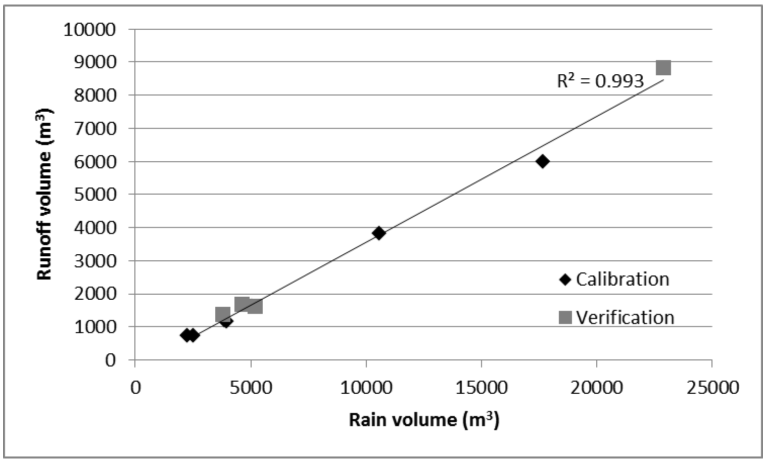


Figure 6-6 – Comparison of rainfall volume and runoff volume for the calibration and verification events of the Frederick Street model

Figure 6-6 indicates that there was little deviation from a linear plot of rainfall volume and runoff volume for each event. This suggests that most of the runoff from these events was sourced from impervious areas, with little contribution from pervious areas in these events, with the possible exception of one verification event.

The initial and final calibration values are presented in Table 6-5. The results of the model calibration for the events in Table 6-4 are shown in Table 6-6.

Table 6-5 – Initial and final estimate values for the Frederick Street SWMM model

Model parameter	Initial Value	Final Value
Impervious area depression storage (loss)	1 mm	0.5
Pervious area depression storage (loss)	5 mm	5
Impervious area Roughness, N_{imp}	0.01	0.013
Pervious area roughness, N_{perv}	0.03 (no units)	0.03
Catchment slope	0.1% to 0.5%	As is
Catchment length (to determine 'Width')	25 m	18 m
Soil infiltration rate – Initial	125 mm/h	100 mm/h
Soil infiltration rate – Final	6 mm/h	8 mm/hr
Decay ('shape factor' in ILSAX)	2 h^{-1}	3 h^{-1}
Drying time	-	5 days

Table 6-6 – Fit of the simulated to observed flow data for calibration events of Frederick Street model

Event	Observed Peak Flow (m³/s)	Simulated Peak Flow (m³/s)	PEP*	r^2	G
Cal1	0.38	0.38	1.34	0.95	0.02
Cal2	0.15	0.17	7.97	0.93	0.01
Cal3	0.37	0.34	-9.87	0.96	0.01
Cal4	0.35	0.36	1.21	0.93	0.02
Cal5	1.24	1.30	5.34	0.96	0.20
Cal6	0.40	0.32	-18.31	0.95	0.02
Cal7	1.34	1.45	8.89	0.95	0.44
* Percentage error in peak					

Figure 6-7 and Figure 6-8 illustrate the fit of the simulated to the observed hydrograph. Examples for each event are shown in Appendix A.

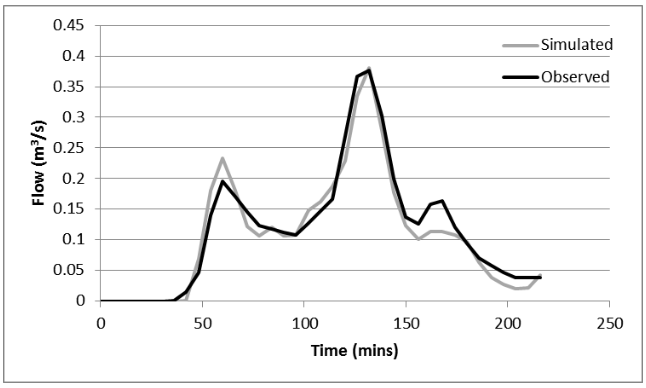


Figure 6-7 – Comparison of the observed and simulated flows for event Cal1 (Frederick Street)

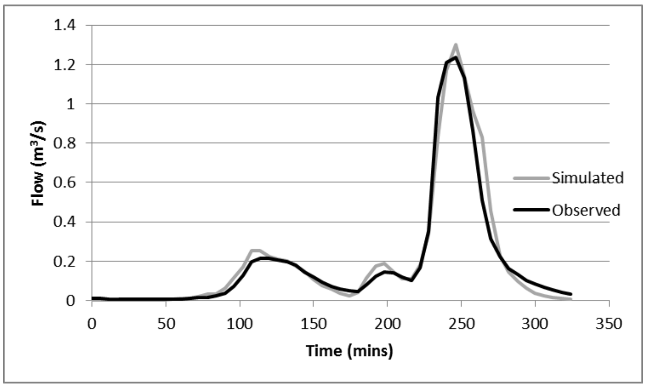


Figure 6-8 – Comparison of the observed and simulated flows for event Cal5 (Frederick Street)

6.3.4 Model Validation

Model validation was conducted to check that the model calibration had not provided a fit only to the calibration period. In the process of model validation, events were selected from the observed flow time series (from 1993 to February 1995) to examine whether the calibrated model was able to adequately predict peak flows outside the initial calibration period. Peak flow was again prioritised by using the r^2 and PEP values as primary indicators. For this reason, observed flow events for validation were selected where observed flow was in excess of 0.8 m^3/s. Events were selected such that flows 3 hours prior to and following the peak flow event were examined. There were four events selected in the observed time series, with the characteristics shown in Table 6-7. A plot of observed runoff and rainfall volume for these events was previously presented in Figure 6-6.

Table 6-7 – Validation events for the Frederick Street catchment model

#	Date	Time	Observed Peak Flow (m^3/s)	Rainfall (mm)		Observed runoff volume (m^3)
				A5040561	A5040556	
V1	30/08/1993	1400 to 2000	0.80	11.2	11.8	1618
V2	19/09/1993	1042 to 1400	0.93	8.2	8.6	1363
V3	13/12/1993	2218 to 0500	1.49	51	51	8818
V4	17/06/1994	0230 to 0900	1.01	9.8	10.6	1683

The results of the validation check are shown in Table 6-8, with selected hydrographs shown beneath. The results for all events are shown in Appendix A. As all events show a reasonable fit to the data, the model was accepted as suitable for the purposes of this study.

Table 6-8 – Fit of the simulated to observed flow data for verification events of Frederick Street model

Event	Observed Peak Flow (m^3/s)	Simulated Peak Flow (m^3/s)	PEP	R^2	G
V1	0.80	0.89	10.48	0.99	0.02
V2	0.93	0.92	-0.93	0.96	0.09
V3	1.49	1.63	8.84	0.88	1.28
V4	1.01	0.93	-8.06	0.94	0.14

Selected hydrographs comparing the observed and simulated results are shown in Figure 6-9 and Figure 6-10. The results for all verification events are shown in Appendix A.

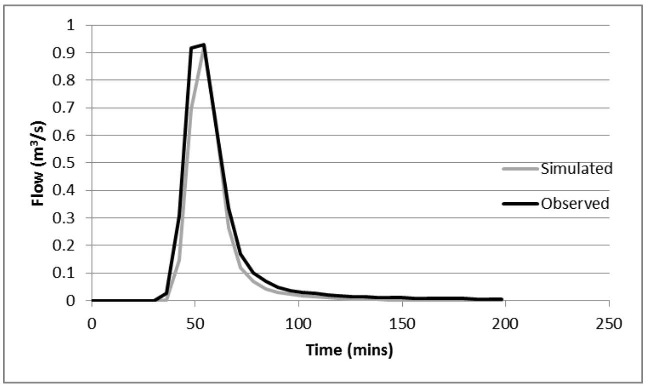


Figure 6-9 – Comparison of the observed and simulated flows for event V2 (Frederick Street)

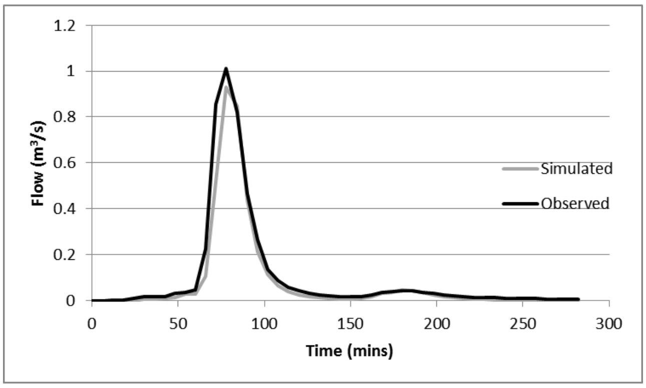


Figure 6-10 – Comparison of the observed and simulated flows for event V4 (Frederick Street)

6.3.5 Assessing Flooding in the Frederick Street Catchment

Flooding in the Frederick Street catchment was examined at a representative location in the catchment, namely a single grated drainage pit on the southern side of Cliff Street, West of the intersection with Gillespie Street. The occurrence of flooding in the PCSWMM model was indicated by temporary ponding of water above stormwater pits unable send water into subsurface pipes (due to an at-capacity inlet, an at capacity drainage pipe or a pipe surcharge) nor send water downstream via the road carriageways (due to the location of the pit at a sag point). The extent and frequency of flooding was recorded throughout the catchment and key areas of flooding were identified using this data. Of these locations, the 'indicator' location was selected based on the frequency of events which occur above a volume threshold considered likely to disturb local traffic, and the exposure of this location to catchment wide conditions.

In the Frederick Street catchment, the flood volume threshold was assumed to be the volume of water stored on the side of the road in excess of that volume which allows for a minimum 2 m wide lane of dry trafficable road surface during storm conditions. This threshold condition is illustrated in Figure 6-11. According to O'Loughlin (1993), the volume of flooding which causes this flood depth to occur can be estimated using Equation 5.

$$V = \frac{d^3}{6S_c}\left(\frac{1}{S_{L_1}} + \frac{1}{S_{L_2}}\right) \qquad \text{- Equation 5}$$

Where d refers to depth, S_c refers to crossfall of the road, S_{L1} refers to the slope of the road approaching one side of the pit and S_{L1} refers to the slope of the road approaching the other side of the pit. These parameters are illustrated in Figure 6-11.

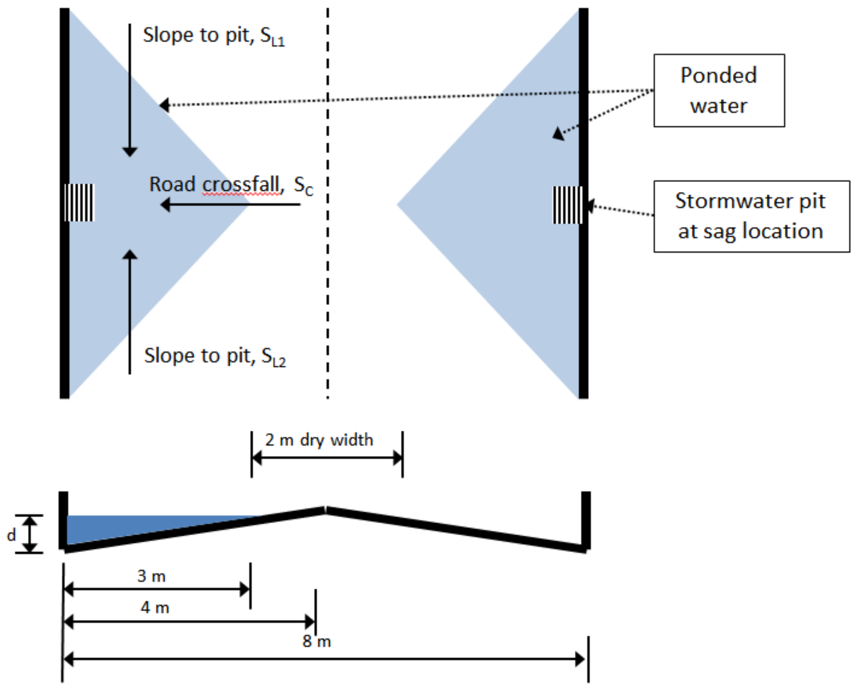


Figure 6-11 – Illustration of assumed design threshold where one lane of trafficable pavement was preserved

City of Marion reported that flooding commonly occurs at the Western end of Mitchell Street, however this flooding results from a single catchment in the Frederick Street model and flooding at this point has been a long term issue attributable to the design and/or construction of the drainage system, not recent infill development. The single grated drainage pit on the southern side of Cliff Street, West of the intersection with Gillespie Street, was selected as a sag point exposed to runoff from most of the catchment via overland flows. This pit was designated C.3 in the catchment model. At this location, the depth of water that causes flooding to occur in excess of that in Figure 6-11 was 75 mm, and the corresponding volume was 4.2 m^3.

To report the impact of WSUD measures on flooding, the recurrence interval of a 4.2 m^3 flood volume at this location was reported for each scenario. The recurrence interval was produced based on analysis of the partial series of peak flood volumes at the flood location. Peak flood volumes were reported by SWMM, in a similar manner to flow rates (Section 3.4.7). Briefly, an annual time series of peak flood volumes was reported, and the minimum annual volume was used as a threshold. This threshold was used to produce a list of all peak flood volumes above it, and a maximum of 3N or 57 events was selected for the partial series (where N represents the number of years of record, or 19 in the current case). The events were plotted using the same methods outlined in Section 3.4.7 and the recurrence interval of the critical flood volume (4.2 m^3) at point C.3 was determined by linear interpolation. It should be noted that there is little evidence of such a procedure being undertaken for flood volumes in the past. While the data should not be considered accurate, it is considered a reasonable approach for comparison in the current study.

6.3.6 Model Scenarios

There were a large number of scenarios used to explore runoff characteristics of the Frederick Street catchment. These can generally be divided into two main groups. All scenarios were undertaken using one of three *development scenarios*. These development scenarios were used to identify the impact of infill development on the peak flow and runoff volume from the catchment. The details of each development scenarios are provided in Section 6.3.7. The development scenarios were then used as the basis for *WSUD scenarios* to investigate the impact that WSUD systems might have on mitigating the changes in peak flow and runoff volume due to the infill development. The assumptions behind WSUD scenarios are detailed in Section 6.3.8 for onsite retention systems (e.g. rainwater tanks), Section 6.3.9 for on-site detention systems (e.g. detention tanks) and Section 6.3.10 for street scale bioretention systems.

6.3.7 Development Scenarios

There were three development scenarios simulated using the Frederick Street model. These were:

1. 1993 Development – the calibration case of the model based on the known catchment properties measured in the field study described by Argue et al (1994) and Lee and Bruce (1995).
2. 2013 Development – the calibration case model with additional development included based on a survey of aerial photos of the catchment in February 2013.
3. 2040 development – the 2013 development scenario, adjusted with additional development projected to occur by 2040, based on the rate of development that occurred between 1993 and 2013. Previous studies across the City of Marion and City of Holdfast Bay catchments have indicated a projected growth of 0.85% per annum in the urbanised area of the City of Marion and City of Holdfast Bay catchment (Tonkin, 2013). However a lot analysis within the Frederick Street catchment indicated that the growth in housing allotments was 0.65%. The latter figure was used in this analysis.

A summary of the final properties for each scenario is shown in Table 6-9. Further detail about each scenario is provided below.

Table 6-9 – Development scenarios for the Frederick Street catchment

Case	Description	Mean impervious area (%)	Mean connected impervious area (%)	Mean indirectly connected impervious area (%)
1993	1993 development (calibration case)	47.5	30.4	17.1
2013	1993 + observed new development	51.7	35.0	16.7
2040	2013 + projected development	56.2	40.0	16.3

The 1993 development scenario was identical to that outlined by Argue et al (1994) and Lee and Bruce (1995). This data was subject to some alterations during calibration as outlined in Section 6.3.3. Using the data in 1993 as a basis, the changes in the catchment were then determined using the following two steps:

1. Determine changes in the number of allotments and houses in from 1993 to 2013. Using this as a basis, project the changes in the number of allotments and houses between 2013 and 2040.
2. Examine the nature of infill development that has occurred from 1993 to 2013. Use this as a basis to derive new catchment properties for 2013 and 2040.

The changes in allotment numbers from 1993 to 2013 were compared using aerial photographs of the Frederick Street catchment in 1993 and 2013. These photographs were provided by the City of Marion and the SA Department of Environment, Water and Natural Resources, respectively. By counting the number of allotments and houses in each sub-catchment area in 1993 and comparing this data to the number of allotments and houses in each sub-catchment in 2013, the increase in the number of houses was determined for each catchment. The results of these findings across the entire catchment are presented in Table 6-10. This indicated that subdivision occurred at an annual rate of 0.91% in the Frederick Street catchment between 1993 and 2013. Furthermore, each subdivision involved the removal of one home and replacement with an average of 2.3 homes (or alternately, the construction of 1.3 homes in addition to an existing home on each allotment). This equates to an annual growth rate of housing allotments of 0.65%. Based on this information the nature of development in the Frederick Street catchment in 2040 was projected, with the results also shown in Table 6-9.

Table 6-10 – Characteristics of allotment subdivision and house construction in the Frederick Street catchment

	Allotments	**Subdivisible allotments**	**New houses**	**No. subdivisions**
1993	555	358	-	-
2013	632	298	77	60
2040*	781	233	149	65
* projected based on growth between 1993 to 2013				

The second step of the process was conducted using the data for the catchment in 1993, and current aerial photography, to examine allotments and determine the mean levels of:

- directly connected impervious area,
- indirectly connected impervious area, and
- pervious area.

This data was then used to produce a representative allotment from 1993 and a representative infill allotment (following subdivision). The characteristics of these allotments are presented in Table 6-11.

Table 6-11 – Assumed characteristics of allotments in Frederick Street catchment area - 1993 and 2013

	1993 Allotment		Redeveloped allotment	
	Area (m^2)	**%**	**Area (m^2)**	**%**
Directly connected impervious	168	23	512	70
Indirectly connected impervious	138	19	110	15
Pervious	425	58	110	15
Impervious (total)	306	42	622	85
Total Area	731	-	731	-

It was therefore assumed that every subdivision involved the removal of a representative 1993 allotment, to be replaced with a representative 2013 allotment with the characteristics shown in Table 6-11.

Following the simulation of base scenarios without any WSUD features incorporated into new development, retention and detention based WSUD scenarios were explored to determine their impact on catchment peak flow rates. The retention based WSUD treatment scenarios explored for the Frederick Street catchment are described below.

6.3.8 On-site Retention Scenarios

The retention systems explored for the WSUD scenarios of the Frederick Street catchment were rainwater tanks and infiltration systems. Each system was simulated in an identical manner, as they represent systems that intercept and permanently hold water on site (effectively diverting water from the drainage system permanently). There were four key variables used to describe rainwater tank and infiltration system properties, including:

- storage volume,
- water demand or infiltration rate,
- connected roof area and
- number of tanks/systems per property.

Storage Volume

To examine the impact of storage volume, storages of 1 kL, 5 kL and 10 kL were simulated. This captured the range of rainwater tank volumes recommended by existing or recent legislation in South Australia and NSW (1 kL minimum) and Queensland (5 kL minimum, in a policy no longer implemented). The 10 kL system was studied as an additional case.

Tank Demand or Infiltration

The impact of water demand between 100 L/day and 1666 L/day was examined. The demand or infiltration scenarios are shown in Table 6-12. Demand larger than 200 L/day was based on achieving the emptying times in Table 6-12 for the 1 kL, 5 kL and 10 kL tanks, where such emptying times were considered reasonable with respect to potential plan area and hydraulic conductivity of soils in Adelaide.

Table 6-12 – Demand/infiltration rate and corresponding emptying time of assumed retention storages

Demand (L/day)	Emptying time (days)			Scenario
	1 kL	5 kL	10 kL	
100	10	50	100	Approximate demand for cold water to laundries[1]
200	5	25	50	Approximate demand for hot and cold water for both toilet and laundry[1]
333	3	15	30	Based on reasonable emptying time for infiltration into soil
1000	1	5	10	Based on reasonable emptying time for infiltration into soil
1667	0.6	3	6	Based on reasonable emptying time for infiltration into soil
[1] Goyder Institute for Water Research, 2011				

For comparison, the average demand for indoor hot and cold water use is reproduced below based on the

Connected Roof Area

The connected roof area to each tank was assumed to be 100 m^2 in all scenarios. The average floor plan area of new housing in South Australia in 2008 – 2009, according to data from the Australian Bureau of Statistics[3] was approximately 200 m^2. However, it is uncommon to connect the entire roof area of houses to rainwater tanks. For example, Chong et al. (2012) conducted a survey of Queensland homes which had installed rainwater tanks in response to changes in the Queensland Development Code MP 4.2. The code required 5 kL rainwater tanks with a minimum connected roof area equal to half of the total roof area, or 100 m^2 (whichever is lesser). This detailed survey revealed that although all 20 homes surveyed had a roof area greater than 200 m^2, and thus required a minimum roof area connection of 100 m^2, very few homes were achieving the minimum connected roof area to the tank.

Total Number of Retention Tanks in the Catchment

The number of retention tanks assumed present in the catchment was explored with two scenarios. The first was on the assumption of one retention tank per new home, and where each redevelopment site was assumed to be broken into two allotments - thus 2 tanks per redevelopment site. This resulted in 154 tanks in 2013, and 288 in 2040. The second was on the assumption a complete retrofit of all houses in the catchment with retention tanks. This resulted in 632 tanks in 2013 and 776 in 2040.

6.3.9 On-site Detention Systems

The detention systems explored using the Frederick Street model were on site detention tanks. The four variables used in the simulation of detention tanks were:

- tank size (volume),

[3] http://www.abs.gov.au/ausstats/abs@.nsf/featurearticlesbytitle/8BB3F6B866BC35CECA2578A000153026?OpenDocument

- tank shape,
- orifice size and
- number of detention tanks.

Each of these variables determines the volume intercepted and/or the emptying time of the detention tank. The interception volume and rate of emptying are important because they influence the downstream hydrograph and the availability of storage at the beginning of rain events.

All detention tank scenarios assume a connected roof area per dwelling of 100 m^2. It was also assumed that detention tanks empty by gravity onto the impervious area of the catchment with a direct connection to the drainage system. In any given catchment, there may be circumstances where a detention system cannot empty by gravity to the drainage system. Examples include sites where a dwelling is situated on a site which slopes away from the road and drainage, or on sites where site constraints necessitate an underground tank where the outlet is below the street drainage system. In a future cost assessment, these circumstances should be adequately costed, as they may require additional costs for pumping, or plumbing to achieve drainage by gravity.

Detention Tank Volume and shape

The detention tank volumes simulated were 1 kL, 5 kL and 10 kL, selected to allow a comparison of detention tank and retention tank performance. In adjusting the volume for a tank with a fixed orifice, the way in which water drains from a tank is influenced by tank shape. The tank shape was assumed to be a cube in the case of a 1 kL tank and a rectangular prism in the case of the 5 kL and 10 kL tank, with the dimensions of each shown in Figure 6-12. The tank shape was selected based on installation of a tank which provided a compromise between footprint and tank height.

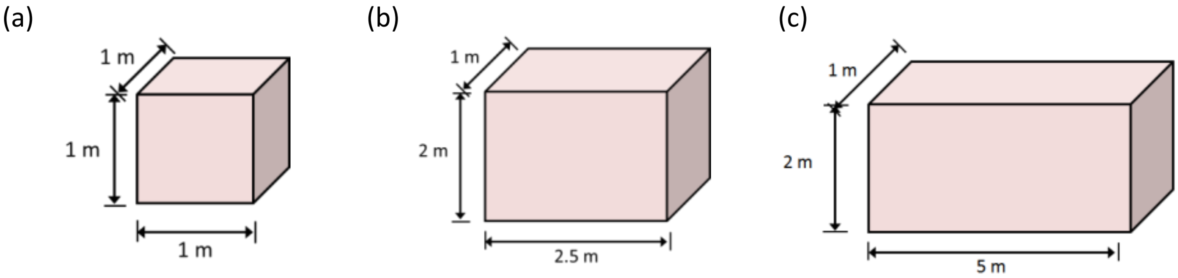


Figure 6-12 – Assumed dimension of the (a) 1 kL (b) 5 kL and (c) 10 kL detention tank (not to scale)

Outflow orifice size

The outflow orifice size examined was between 10 mm and 50 mm, in 10 mm increments. This was assumed reasonable based on the orifice size of detention tanks already recommended in Greater Adelaide, such as by the City of Tea Tree Gully (30 mm) and the City of Mitcham (19 mm).

Number of Detention Tanks

The number of detention tanks assumed was identical to rainwater tanks, using two scenarios. The first was on the assumption of one detention tank per new home, and where each redevelopment site was assumed to be broken into two allotments - thus 2 tanks per redevelopment site. This resulted in 154 tanks in 2013, and 288 in 2040. The second was on the assumption a complete retrofit of all houses in the catchment with detention tanks. This resulted in 632 tanks in 2013 and 776 in 2040.

6.3.10 Street Scale Bioretention

The implementation of street scale bioretention was explored based on the arrangement of bioretention systems in Mile End (Section 5). Based on an examination of the Mile End rain gardens in Mile End, it was found that rain gardens were situated on each side of the road and at intervals of approximately 100 m. Therefore, to examine the impact of bioretention gardens in the Frederick Street catchment, rain gardens were placed on road sides at 100 m intervals throughout the catchment. Each rain garden was assumed to be identical with rain garden RG5 Section 5.3.2, with surface properties previously presented in Table 5-2 and drainage properties as presented in Table 5-3.

6.4 Results

The results of the Frederick Street simulations are provided in the following sections. Section 6.4.1 describes the impact of increased development density on runoff from the Frederick Street catchment. Section 6.4.2 describes the impact of retention tanks on runoff when fitted to new homes constructed since 1993. Section 6.4.3 describes the impact of retention tanks on runoff when fitted to all homes in the catchment. Section 6.4.4 describes the impact of detention tanks on runoff when fitted to new homes in the catchment constructed since 1993. Section 6.4.5 describes the impact of detention tanks on runoff when fitted to all homes in the catchment. Section 6.4.6 describes the impact of street scale bioretention systems on runoff from the catchment.

6.4.1 The Impact of Infill Development in the Frederick Street Catchment

Impact of Infill Development on Runoff Volume

The simulation of the same 19 years rainfall data over the catchment scenarios in 1993, 2013 and 2040 indicated that the volume of runoff from the catchment was 50.1 ML/annum in 1993, increasing by 14% to 57.2 ML/annum in 2013. The 1993 flow volume increased by 30% to 65 ML/annum in the 2040 scenario. These results are illustrated in Figure 6-13.

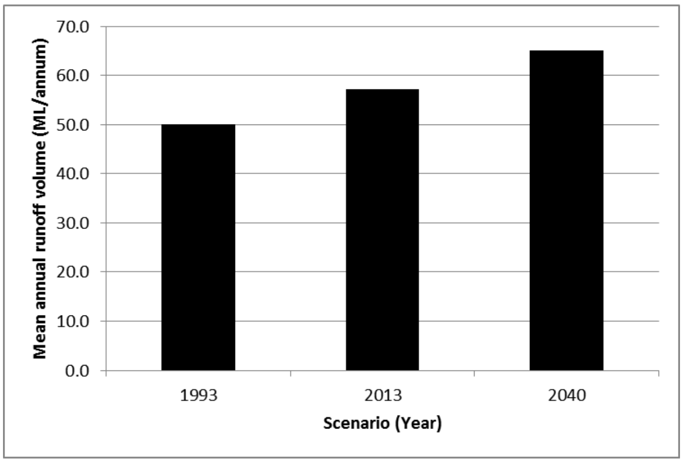


Figure 6-13 – Mean annual runoff volume from Frederick Street in the 1993, 2013 and 2040 scenarios

Impact of Infill Development on Peak Flow Rates

Table 6-13 shows the changes in the estimated 6 month, 1, 2 and 5 Year ARI peak flows at the outlet of the Frederick Street catchment for the calibration case (pre-infill development scenario, 1993), the simulated current case (2013) and the future infill scenario (2040). Note that these flow rates represent the sum of drain (pipe) flows and overflows conveyed at the surface. Based on an analysis of when overland flow begins to occur, the existing system is estimated to be capable of conveying events up to the 2.5 year ARI, or approximately 1 m^3/s. The capacity of the system was previously estimated to be 1.32 m^3/s (personal communication with David Kemp, based on ILSAX simulation of on-grade capacity). The results in Table 6-13 indicate that the 6 month, 1, 2, 5 and 10 year ARI peak flow rates increase as infill development takes place across the catchment.

Table 6-13 – Predicted peak flows (m^3/s) from the Frederick Street catchment in 1993, 2013 and 2040 based on partial series analysis

Case	0.5 Year ARI	1 Year ARI	2 Year ARI	5 Year ARI
1993	0.68	0.82	0.97	1.81
2013	0.78	0.93	1.09	1.91
2040	0.89	1.04	1.19	2.00

Impact of Infill Development on Flood Frequency

Based on the methodology in Section 6.3.5, flooding was assessed based on the frequency of a flood volume of 4.2 m^3 at a case study location on the southern side of Cliff Street, West of the intersection with Gillespie Street (model reference node C.3). It was also assumed that the drainage system was able to preserve road widths by ensuring flooding remains less than this value up to and including the 5 year ARI flows.

The events which plotted most closely to the 5 Year ARI in the partial series were those with a return period estimate of 4.2 and 5.3 years. Both of these events caused flooding in excess of the 4.2 m^3 threshold (21 m^3 and 13 m^3, respectively) in the 1993 scenario. All events in the partial series above the 2.5 year ARI caused flooding in excess of this threshold. In the 2013 scenario, this frequency increased to include all events above the 2.2 year ARI, and in the 2040 scenario, this frequency had increased to include all events above the 1.7 year ARI. It should be stressed that these flood frequency values are only estimates linearly interpolated from a partial series of peak flood volumes at the flood assessment point. The figures are estimated for comparative purposes in this report.

6.4.2 Effect of Retention Tanks on Stormwater Runoff when fitted to New Homes

Management of Stormwater Runoff Volume Using Retention Tanks

Runoff volume increases of 30% were projected to occur due to infill development from 1993 to 2040 (Figure 6-13). The mean annual runoff volume from the catchment outlet when 1 kL, 5 kL and 10 kL onsite retention tanks were introduced in conjunction with new housing is presented in Figure 6-14 for both the 2013 and 2040 development scenario. These scenarios represent the case where tanks were installed on new homes (e.g. two tanks per redeveloped allotment) with a connected roof area of 100 m^2.

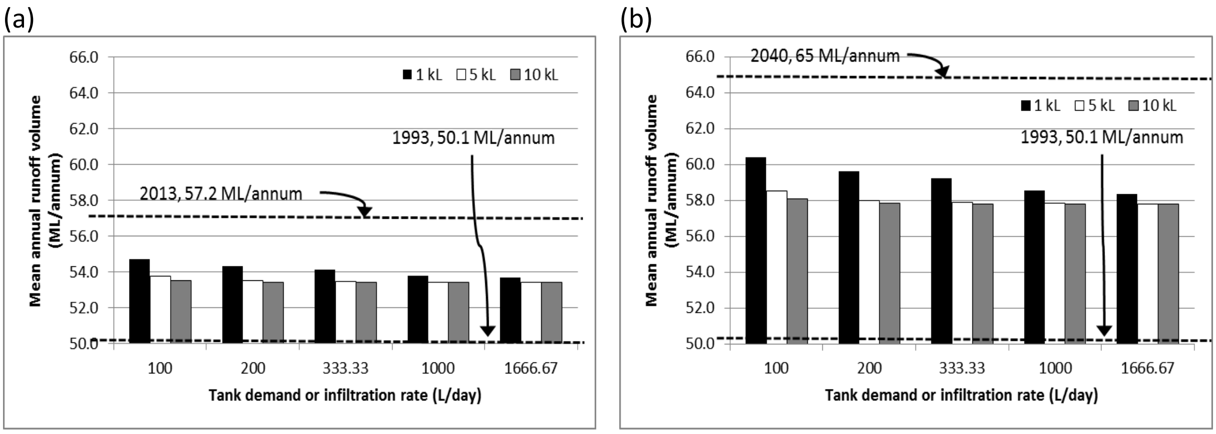


Figure 6-14 – The total runoff volume from the Frederick Street catchment when applying 1 kL, 5 kL and 10 kL rainwater tanks to new homes up to (a) 2013 and (b) 2040

These results indicate that the volume of runoff in 2013 could have been reduced by approximately 4.3% to 6.0% using rainwater tanks of 1 kL to 5 kL volume, respectively, with typical usage (100 L/day). The volume of runoff in 2040 could be reduced by 7.1% to 9.9% using rainwater tanks of 1 kL to 5 kL volume, respectively. Using this data, it is also possible to quantify the amount of water retained for reuse or infiltration, as shown in Figure 6-15.

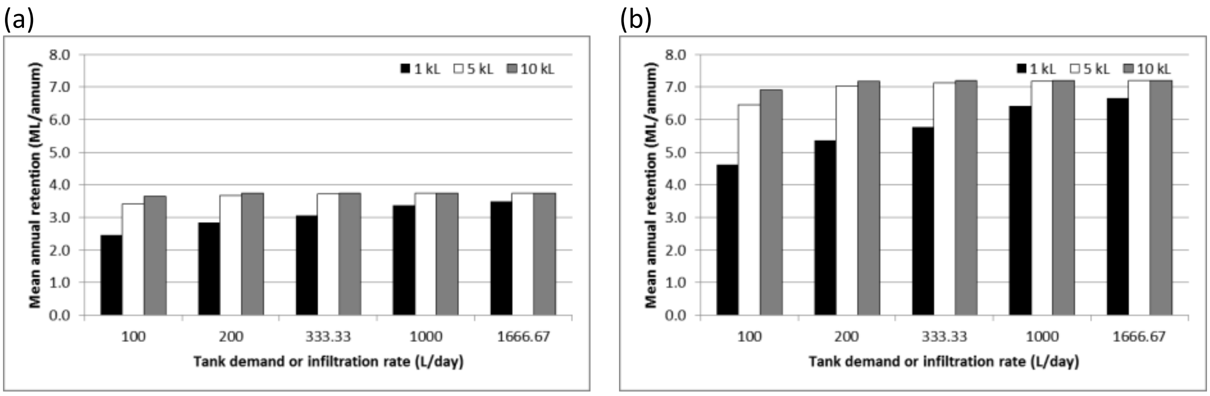


Figure 6-15 – The total runoff volume retained from the Frederick Street catchment when applying 1 kL, 5 kL and 10 kL rainwater tanks to new homes up to (a) 2013 and (b) 2040

These results indicate that there was very little volume retention benefit by increasing the tank size from 5 kL to 10 kL, with the greatest reductions occurring between 1 kL and 5 kL. In addition, there was little benefit to simulating high levels of demand/infiltration for the 5 kL and 10 kL rainwater tanks. The annual runoff volume was reasonably similar for demand/infiltration rates between 333 L/day and 1666 L/day, indicating that the optimal level of demand (or infiltration) is between 200 L/day and 333 L/day with the assumed connected roof area. This has implications for designing infiltration systems. For a known infiltration rate, an infiltration surface area can be designed to dispose of 330 L/day into soil to achieve best results, as larger disposal rates will not produce extra benefits in terms of volume.

Management of Peak Flow Rates Using Retention Tanks

The peak flow rates from the catchment in the 1993, 2013 and 2040 development scenarios were presented in Table 6-13. The effect of retention systems on these peak flow rates are presented in Figure 6-16 for 2013 and 2040. Each diagram also compares the effect of tank size (1 kL, 5 kL or 10 kL) and tank demand (from 100 L/day to 1666 L/day). These results assume two tanks at each redeveloped site, where each has a connected roof area of 100 m^2.

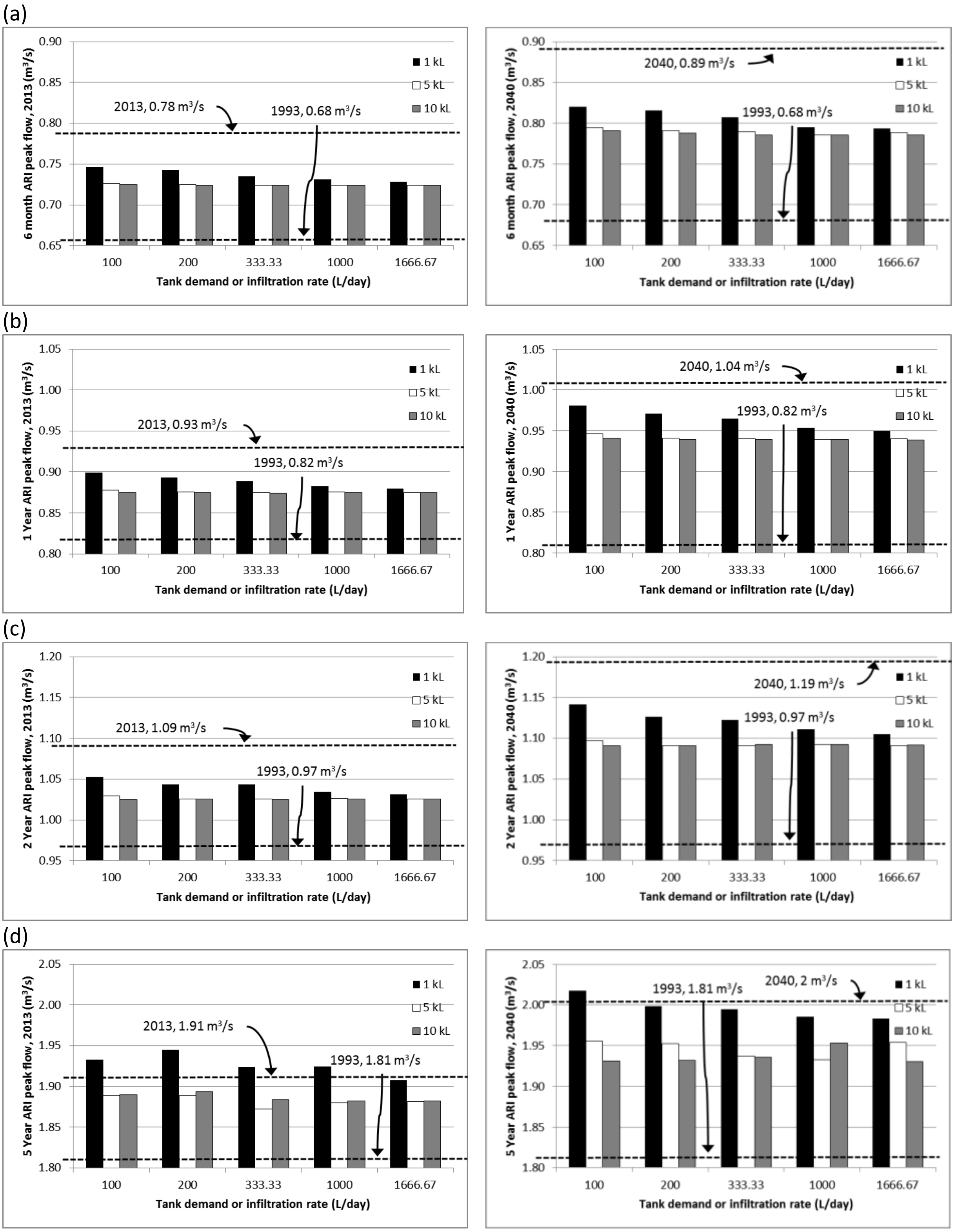


Figure 6-16 – The effect of retention systems on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI of peak flows from the Frederick Street catchment in 2013 and 2040

The impacts of retention on the 6 month, 1, 2 and 5 year ARI when applied to new housing between 1993 and 2013 (Figure 6-16) was found to be small. Despite reducing peak flows to levels lower than those in the absence of rainwater tanks, the simulated scenarios were unable to restore peak flow rates to the 1993 levels. Even when 10 kL retention tanks were applied to new housing with very high water use (1666 L/day, in the range of generous infiltration) in 2013, the 6 month, 1, 2 and 5 year ARI peak flows were reduced by 7%, 6%, 5% and 2%, respectively, compared to the required reduction of 13%, 12%, 11% and 6% to restore peak flow rates to 1993 levels. Similar results were found for the 2040 case. Assuming 10 kL tanks with very high levels of use/disposal (1666 L/day) the 0.5, 1, 2 and 5 year ARI outflows were reduced by 12%, 10%, 9% and 3%, respectively, compared to the required reduction of 23%, 21%, 19% and 10% to restore peak flow rates to 1993 levels. Generally, the reduction in the peak flows was lower when the frequency of peak flows reduced. For example, for any given scenario, the percentage reduction of 0.5 year ARI peak flows was higher than the percentage reduction of 5 year ARI peak flows.

Management of Flood Volume Using Retention Tanks

Flood management was assessed at the single case study location using the procedures in Section 6.3.5. Figure 6-17 shows the ARI of flooding in excess of the critical depth at the flood assessment point achievable using 1 kL, 5 kL and 10 kL retention tanks with varying levels of demand or infiltration. The results shown indicate that improvement on the frequency of the critical flood was achieved using retention systems. For example, in 2013, 5 kL tanks could effectively restore the ARI of the critical flood to levels in 1993. This was not achieved in 2040. Like the flow rate and volume results, there was little benefit progressing from a 5 kL retention tank to a 10 kL retention tank regardless of the level of reuse, nor was there benefit in adopting demand/infiltration levels higher than 333 L/day. The optimum tank size for this catchment would appear to be between 1 kL and 5 kL to reduce flooding.

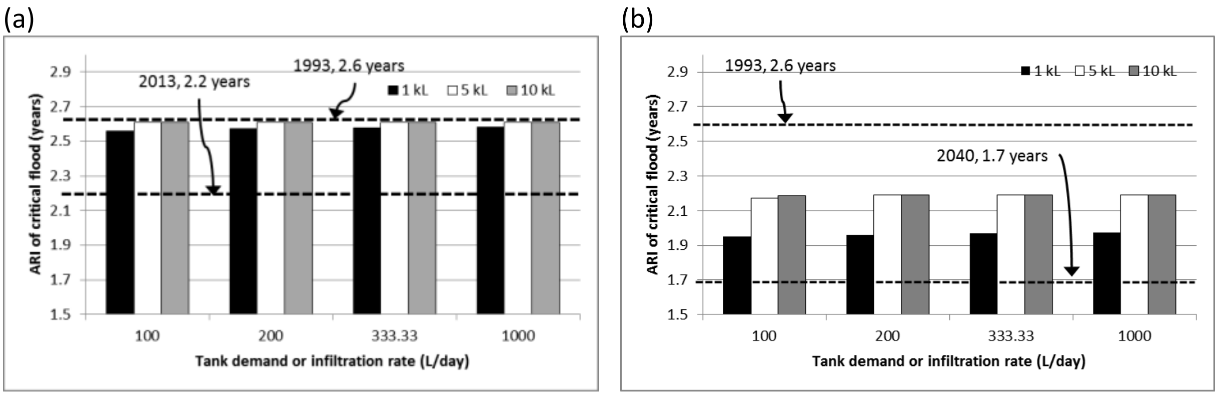


Figure 6-17 – The ARI of critical flood depth at the flood assessment point when retention tanks were applied to the Frederick Street catchment (a) 2013 and (b) 2040 scenarios

6.4.3 Effect of Number of Assumed Retention Tanks in 2040

Management of Stormwater Runoff Volume Using Retention Tanks on All Homes

The stormwater runoff volume generated by increasing the number of retention tanks in the 2040 scenario of the Frederick Street catchment are shown for the 1 kL, 5 kL and 10 kL tanks in Figure 6-18. The figure compares the result of having retention tanks fitted to new homes (2 per redeveloped allotment, or 288 homes) with the result of tanks fitted to all homes in the catchment (776 homes). In each case, the connected roof area was 100 m^2. The impact of adopting rainwater tanks on all homes in 2040 on the volume retained for reuse or infiltration is shown in Figure 6-19.

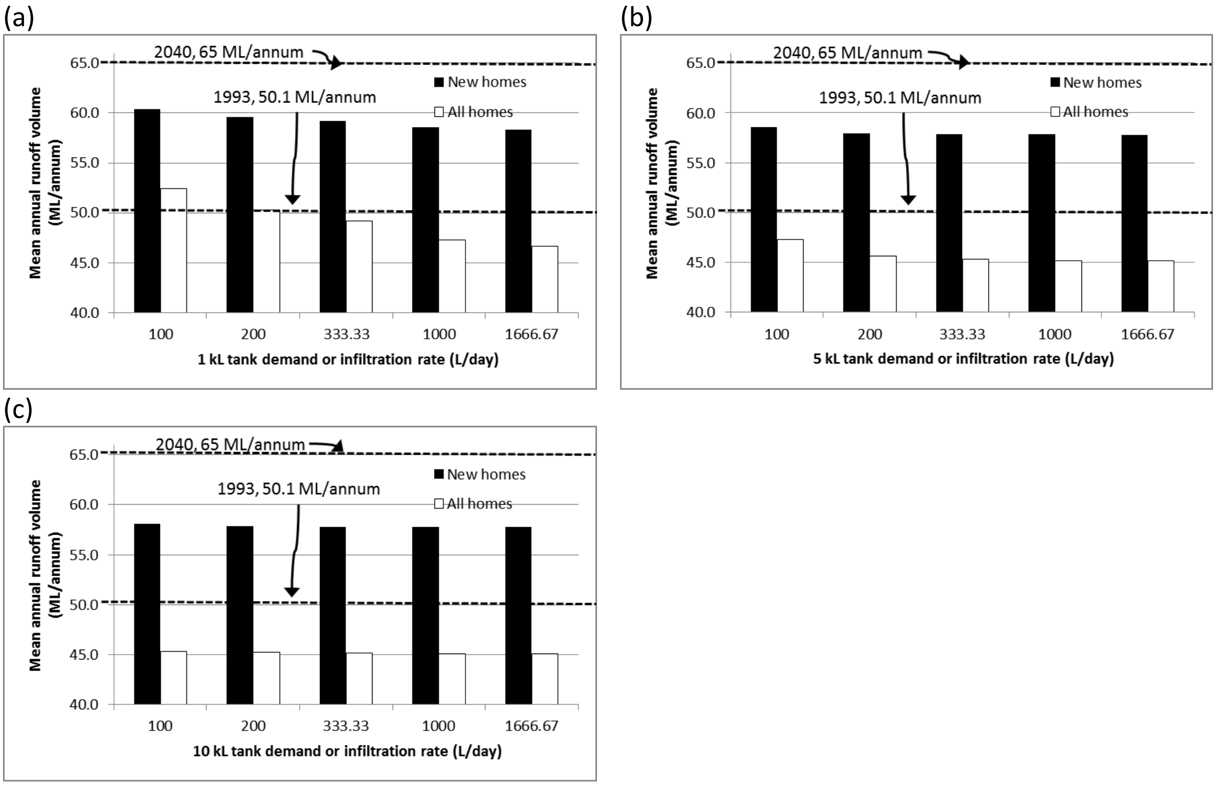


Figure 6-18 – Comparing the impact of assumed rainwater tank numbers on the mean annual runoff volume in 2040 using (a) 1 kL tanks (b) 5 kL tanks and (c) 10 kL tanks

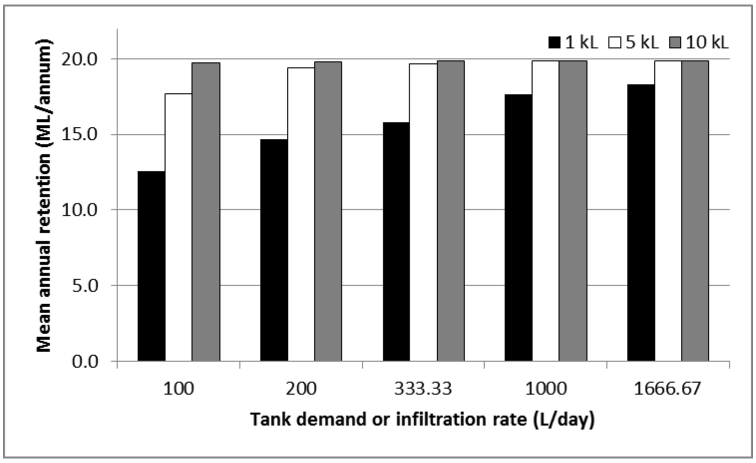


Figure 6-19 – The total runoff volume retained from the Frederick Street catchment when applying 1 kL, 5 kL and 10 kL rainwater tanks to all homes in 2040

The results show that increasing the number of tanks has produced a much greater runoff reduction in 2040. The results also indicate that the effect of increasing the tank size had greatest effect when the tanks were increased from 1 kL to 5 kL, with little additional volume reduction when 10 kL tanks were adopted. The adoption of 5 kL tanks with average demand (100 L/day) was able to reduce the runoff volume to levels lower than those observed prior to infill development.

Management of Peak Flow Rates Using Retention Tanks on All Homes

The effect of assumed tank numbers on the ARI of peak flows in the 2040 scenario of the Frederick Street catchment is presented for the 1 kL and 5 kL tank volume in Figure 6-20 (1 Year ARI) and Figure 6-21 (5 Year ARI). These results compare the assumption of providing 1 kL and 5 kL rainwater tanks to new homes or all homes in the catchment, where each tank had a 100 m^2 connected roof area.

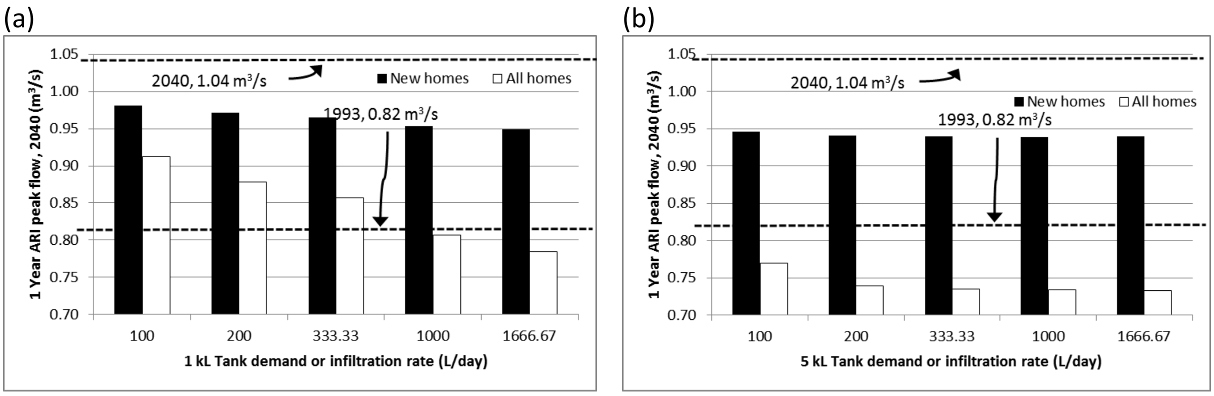


Figure 6-20 – The impact of assumed rainwater tank numbers on the 1 Year ARI using (a) 1 kL and (b) 5 kL tanks

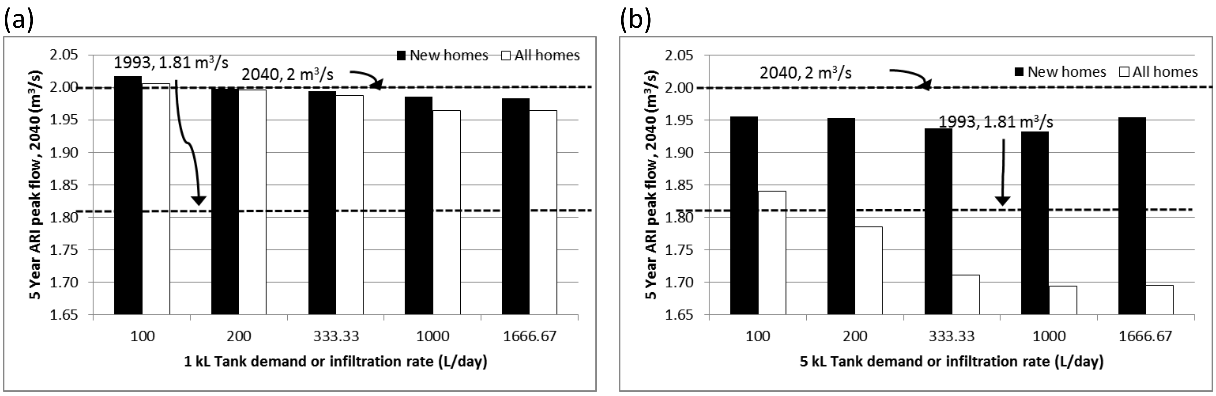


Figure 6-21 – The impact of assumed rainwater tank numbers on the 5 Year ARI using (a) 1 kL and (b) 5 kL tanks

The results indicate that when tanks were fitted to only new homes in 2040, peak flows could not be restored. However, when the entire catchment was fitted with retention tanks, the 1 year ARI and 5 Year ARI peak flows observed in 1993 were restored, or reduced below the pre-infill development levels in 1993. For 1 kL tanks, very high levels of use or infiltration were required, but they were most effective for more frequent events (lower ARI). However modest levels of usage (100 L/day) could reduce peak flows to near target levels of up to the 5 year ARI when 5 kL retention tanks were fitted to all homes in the catchment.

Management of Flood Volume Using Retention Tanks on All Homes

The ARI of critical flood depths achieved when tanks were fitted to every home in the catchment, or just new homes (assuming two tanks per subdivision), are shown in Figure 6-22. The results indicate that to reduce flooding it was far more effective to install tanks across the entire catchment rather than new homes only. When all homes were fitted with retention tanks, the ARI of the critical flood depth at the case study location was almost restored to pre-infill development levels in 1993 using

1 kL tanks. The adoption of 5 kL tanks improved this even further. The results for 10 kL tanks resembled those of the 5 kL tanks (data not shown) indicating there was little improvement in adopting tank sizes greater than 5 kL and that the optimum tank size was between 1 kL and 5 kL. This may also be a function of connected roof area, which in this case was 100 m^2.

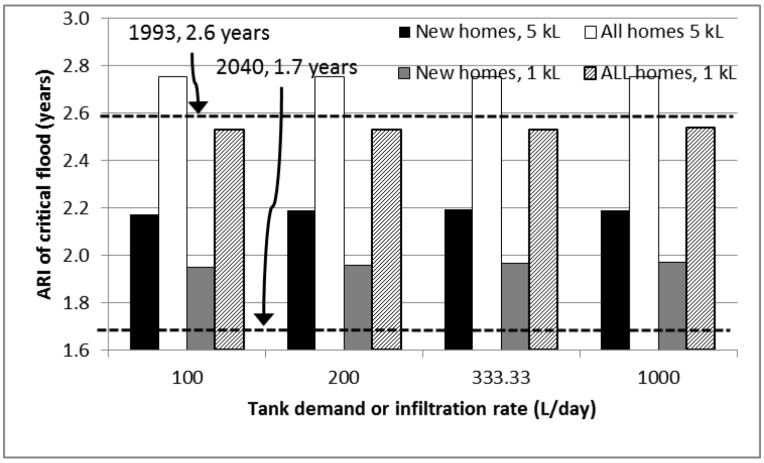


Figure 6-22 –The ARI of the critical flood depth when retention tanks were applied to new homes or all homes across the catchment in 2040

6.4.4 Effect of Detention Tanks on Stormwater Runoff when fitted to New Homes

Management of Stormwater Runoff Volume Using Detention Tanks

Stormwater detention systems are not designed to have an impact on stormwater runoff volumes. In the simulations for this report, the volume of water intercepted by detention systems was released in its entirety back to the stormwater drainage system and there was no impact on stormwater runoff volume.

Management of Peak Flow Rates Using Detention Tanks

The peak flow rates from the catchment in 1993, 2013 and 2040 were presented in Table 6-13. The effect of detention systems on these peak flow rates from the catchment are presented in Figure 6-23 for 2013 and 2040. These diagrams also compare the effect of detention tank size (1 kL, 5 kL or 10 kL) and demand (from 100 L/day to 1666 L/day). These results assume two detention tanks at each redeveloped site, where each tank has a connected roof area of 100 m^2.

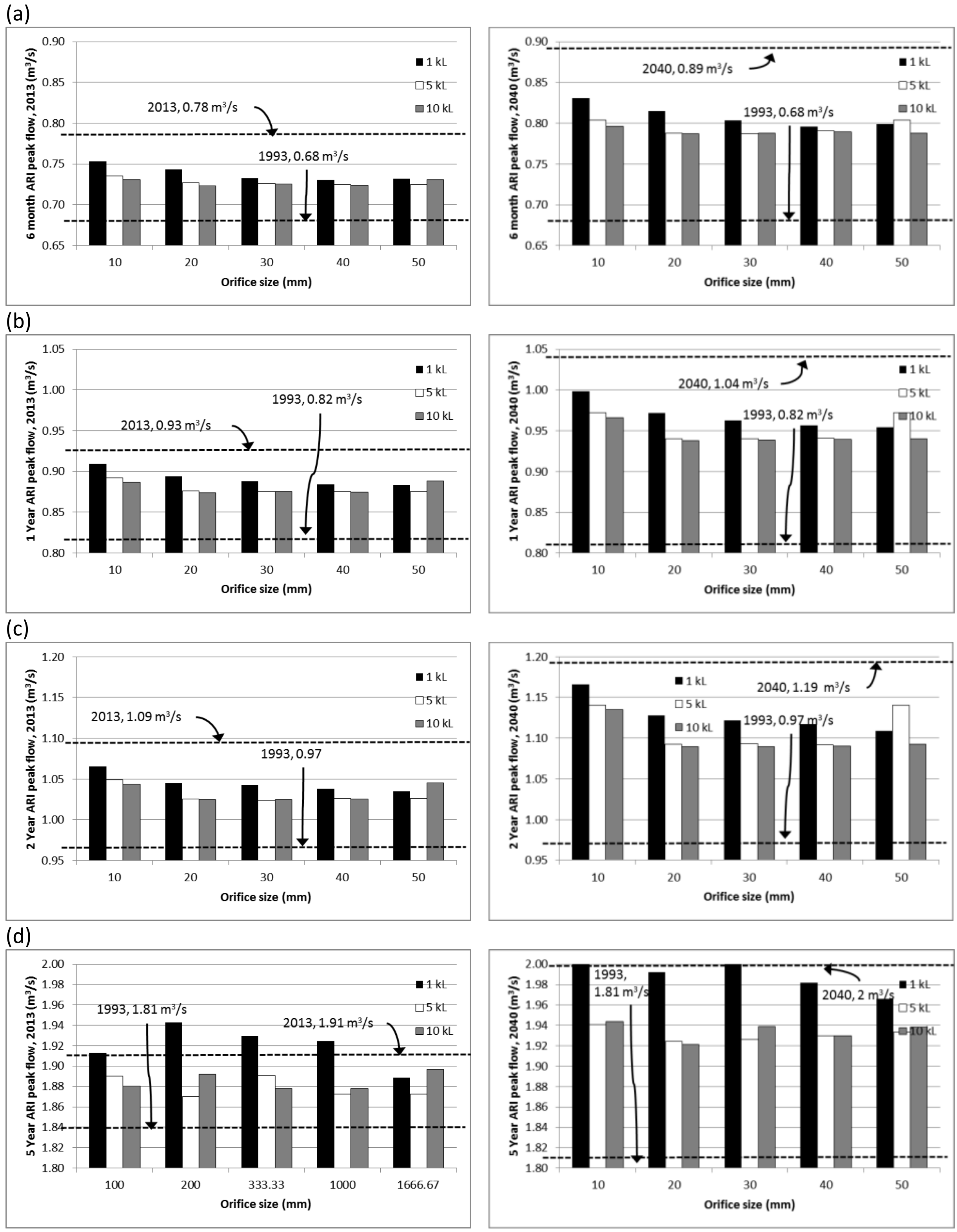


Figure 6-23 – The effect of 1 kL, 5 kL and 10 kL detention tanks on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI peak flows from the Frederick Street catchment in 2013 and 2040

The results for detention tanks applied to new homes in 2013 and 2040 indicate that none of the proposed detention tank sizes were able to restore the 6 month, 1, 2 and 5 year ARI peak flow rates, however larger capacity tanks were most effective. In general, orifice sizes in the mid-range of those simulated (20 mm to 40 mm) performed best. Similar to retention tank results, the reduction in the peak flows was lower when the frequency of peak flows reduced. For any given scenario, the percentage reduction of 6 month ARI peak flows was higher than the percentage reduction of 5 year ARI peak flows. It should also be noted that the 10 mm orifice size assumption produced higher levels of routing error in the simulation. This may be because the flows from detention tanks with a 10 mm orifice were quite small, and experience with SWMM modelling in the past (Myers et al., 2012) has shown that the routing of small flows through a stormwater network produces a relatively high routing error (errors in mathematical calculations of volume as the movement of water occurs through the drainage system). These errors resulted in losses in the order of 5% in runoff volume estimation, which may be responsible for the results for the 10 mm orifice which tend not to conform to a trend. The routing error value reduced when larger diameter orifices are assumed.

The 5 year ARI values were not following a trend, indicating that prediction of an ARI above the capacity of the catchment may have been poor due to the limited number of events above the drainage system capacity which could form a partial series from which to estimate a 5 year ARI event.

Management of Flood Volume Using Detention Tanks

The assessment of flood volume was undertaken using the procedures in Section 6.3.5. Using the calibrated catchment model in 1993 with the 19 years of rainfall data, it was found that flooding in excess of the critical flood depth occurred approximately every 2.6 years. Figure 6-24 shows impact of detention tanks on the ARI of this critical flood depth, except for the 10 kL tanks which responded in a similar way to 5 kL tanks. The results indicated that the ARI of the critical flood depth was improved when detention systems were applied to new allotments. In the 2013 scenario, detention tanks were able to restore the pre infill development flood frequency, but this was not the case in the 2040 scenario. Larger tanks tended to reduce flood volumes at the case study location, but there was little improvement by adopting a tank greater than 5 kL. Like the end of catchment peak flow management results, orifice sizes tended to be most effective in the range of 20 mm to 40 mm.

(a) (b)

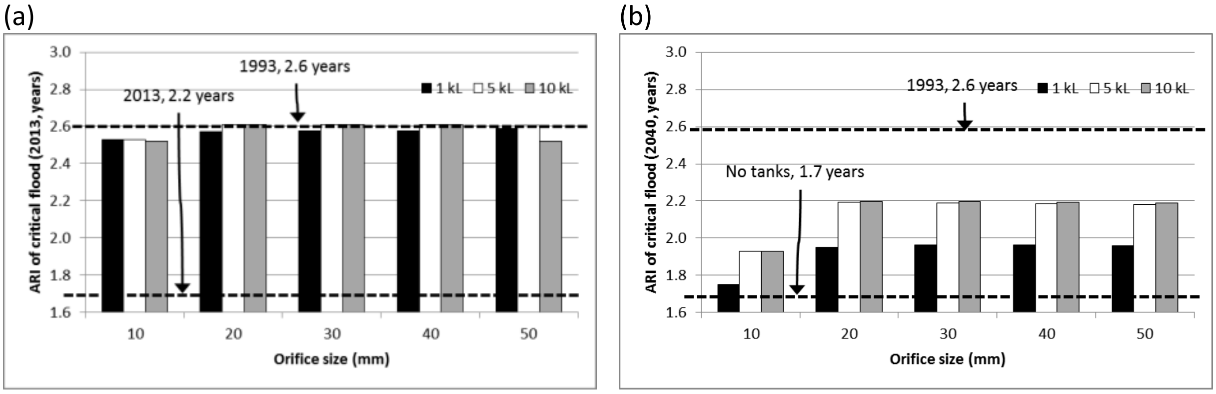


Figure 6-24 – The ARI of critical flood depth at the Frederick Street catchment flood assessment point for the (a) 2013 and (b) 2040 scenario

6.4.5 Effect of Number of Assumed Detention Tanks in 2040

Management of Peak Flow Rates Using Detention Tanks on All Homes

The effect of the number of detention tanks on the ARI of peak flows in the catchment is presented for 1 kL and 5 kL tanks in Figure 6-25 (1 Year ARI) and Figure 6-26 (5 Year ARI). The results compare the impact of having a detention tank on new homes (288 allotments) or all homes (788 allotments) in the catchment in 2040 with a 100 m^2 roof connection.

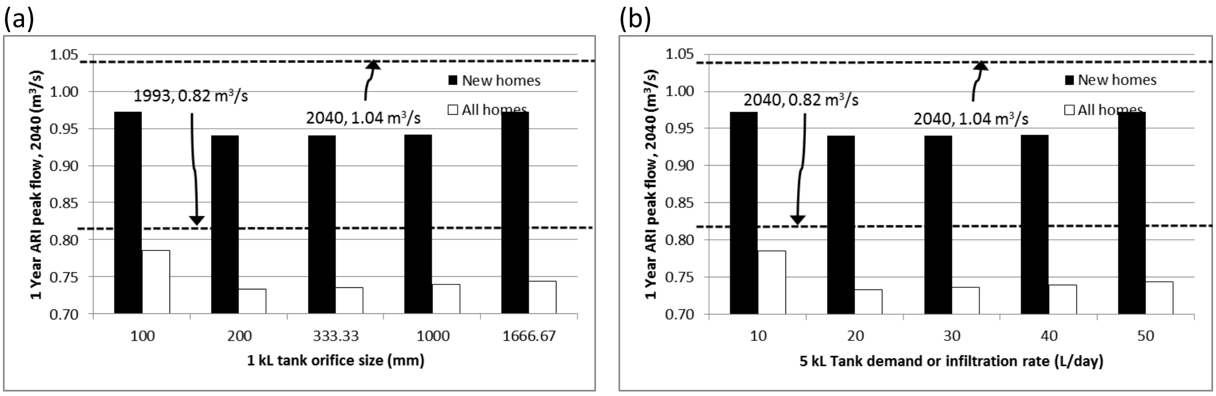


Figure 6-25 – The impact of assumed detention tank numbers on the 1 Year ARI in Frederick Street using (a) 1 kL and (b) 5 kL tanks

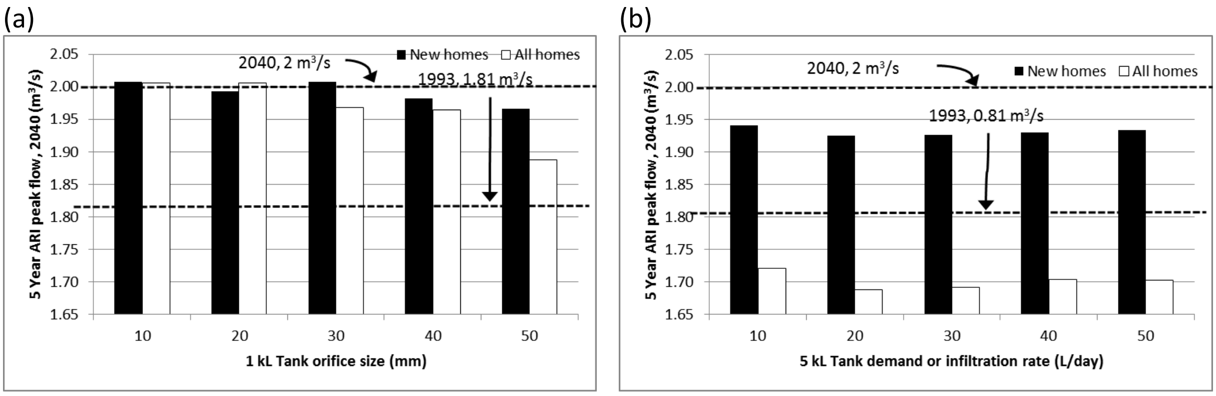


Figure 6-26 – The impact of assumed detention tank numbers on the 5 Year ARI in Frederick Street using (a) 1 kL and (b) 5 kL tanks

Management of Flood Volume Using Detention Tanks on All Homes

The ARI of critical flood depth at the Frederick Street flood assessment point when tanks were fitted to every home in the catchment, or new homes only (assuming two tanks per subdivision), are compared in Figure 6-27 for 1 kL, 5 kL and 10 kL tanks. The results indicate that it was far more effective to install tanks across the entire catchment rather than new homes only. The results also indicated that there was no improvement in adopting tank sizes greater than 5 kL and that the optimum tank size was between 1 kL and 5 kL. This may also be a function of connected roof area, which was 100 m^2 in this case.

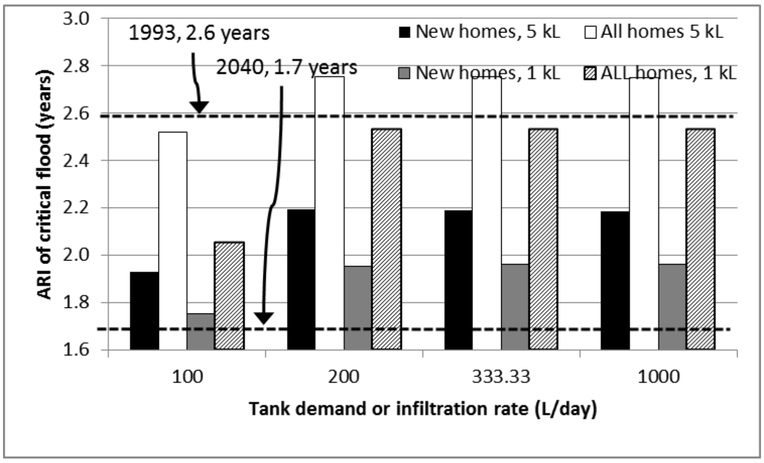
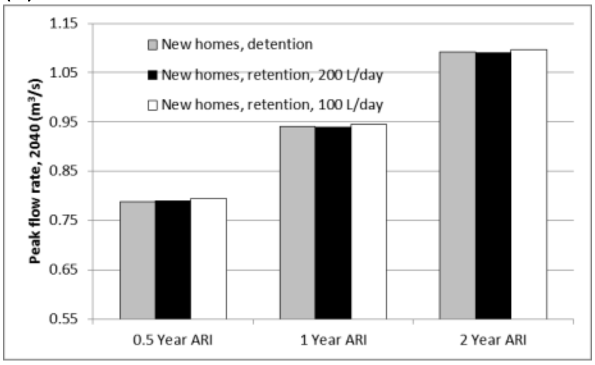


Figure 6-27 – The ARI of critical flood depth at the Frederick Street catchment flood assessment point for the for 1 kL and 5 kL detention tanks

6.4.6 Comparing Detention and Retention – Peak Flow Rates and Flooding

Using the data above, it was possible to examine the effectiveness of detention tanks and retention tanks for peak flow reductions in Frederick Street catchment when applied in equal numbers across the catchment. A comparison for the 6 month, 1 year and 2 year peak flow rates are presented in Figure 6-28. The figures compare the impact of 5 kL retention tanks with 200 L/day demand and 5 kL detention tanks with a 20 mm orifice. The results were similar for the 1 kL case. Results were also similar for the 5 year ARI, but the data was not shown because the y-axis scale detracted from interpreting the results at the lower ARI values.

(a)



(b)

Figure 6-28 – Comparison of the peak flow resulting from 5 kL retention (100, or 200 L/day demand) and 5 kL detention (20 mm orifice) on (a) new homes in 2040 and (b) all homes in 2040

The results indicate that there was little difference between the peak flow rates resulting from identical scenarios with detention and retention tanks of equal volume applied to new homes. When these tanks were applied to new and existing homes, the results again indicated there was little difference between retention and detention. When the resulting flood volumes of these scenarios were plotted, there was little difference between these volumes. This indicates that any judgement over the application of retention and detention should consider more than just peak flow and flooding impacts. For example, retention can provide benefits in terms of providing an alternative water source and/or groundwater recharge. However site conditions may not be suitable, and reuse infrastructure may require long term maintenance. Detention tanks may be relatively simpler and cheaper to install, and they ensure that flow is still available for any downstream municipal harvesting scheme. In the absence of this, however, detention does not provide benefits in terms of flow volume control. It is recommended that these results are explored further on a larger catchment where the effects of selecting detention or retention for flow management may become more pronounced. It is also recommended that future investigations consider the application of a combined retention and detention scenario. This would reflect the performance of centrally controlled on-site harvesting systems which ensure that storages are emptied prior to predicted storm events. These may improve the overall peak flow and demand reduction benefits of applying on-site storage systems.

6.4.7 Effect of Street Scale Bioretention on Stormwater Runoff

Management of Stormwater Runoff Volume using Street Scale Bioretention

The street scale bioretention systems simulated in the Frederick Street catchment were assumed to be connected to the drainage system via an underdrain, and lined with an impermeable material. There was a negligible reduction of runoff of approximately 0.2% in 2013 and 2040 achieved by the lined street scale bioretention systems.

Management of Peak Flow Rate using Street Scale Bioretention

The effectiveness of street scale bioretention for the management of peak flow rates across the Frederick Street catchment are shown in Figure 6-29. The results indicate that street scale bioretention was able to restore peak flow rates to near those levels found for the 1993 scenario in 2013, however they were not able to do so for the 2040 scenario.

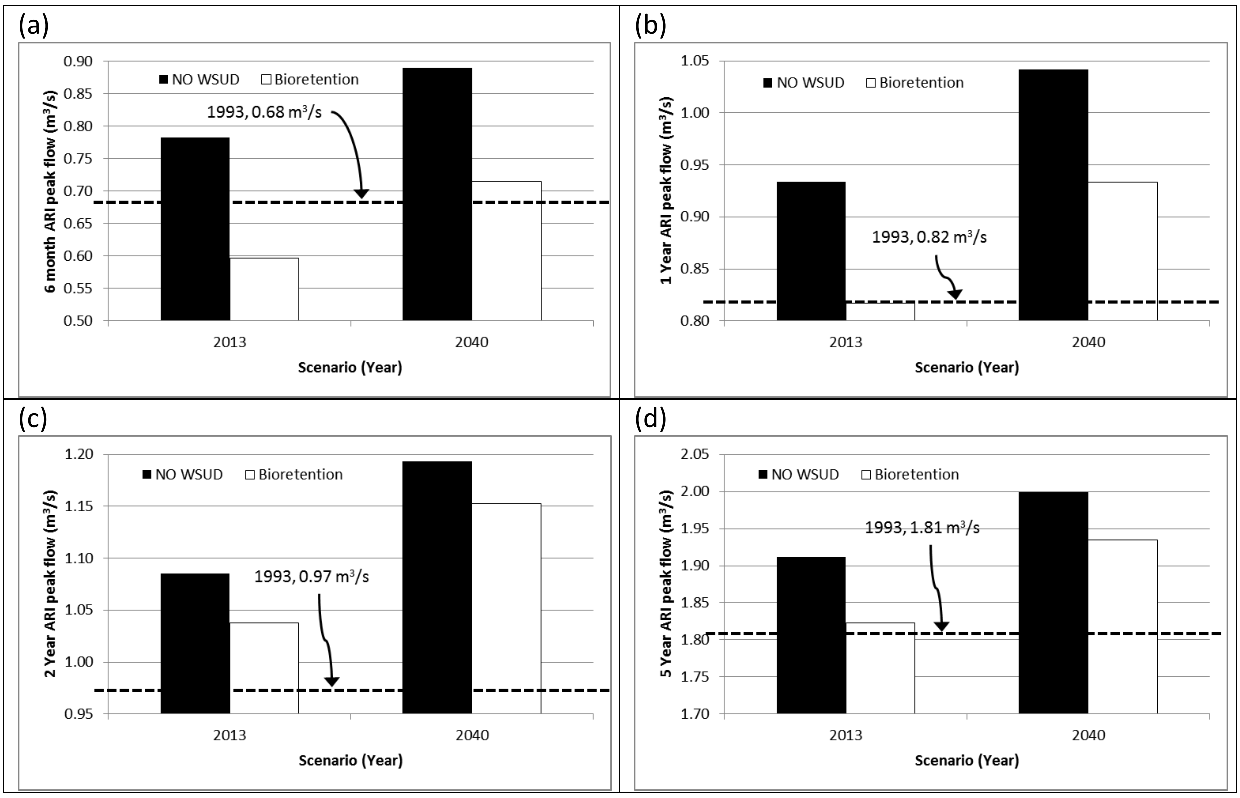


Figure 6-29 – The impact of street scale bioretention on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI peak flows from the Frederick Street catchment in 2013 and 2040

The effectiveness of street scale bioretention may be attributed to the detention of flows during the ponding and filtration process, which is particularly effective because of the large connected impervious area that is possible at the street scale. Retention and detention tank scenarios have been restricted to a 100 m^2 roof connection on each allotment, however it was reasonable to assume that all impervious surfaces drain to street scale bioretention as runoff from contributing catchment area roofs, driveways and roads will drain to the street scale bioretention when installed at 100 m intervals across the Frederick Street catchment.

Management of Flood Volume by Street Scale Bioretention

Figure 6-30 shows the impact of street scale bioretention on the ARI of the critical flood depth at the case study location in the Frederick Street catchment. The results indicate that some improvement of the flood frequency is possible using bioretention at the street scale. The ARI of critical flood depth was improved beyond the 1993 level when applied to the 2013 catchment, but the systems were less effective in 2040.

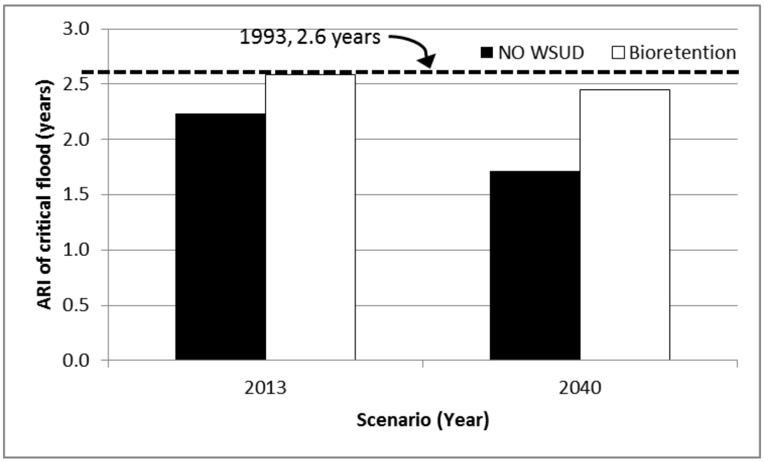


Figure 6-30 – The ARI of critical flood depth at the Frederick Street catchment flood assessment point with and without street scale bioretention for the 2013 and 2040 development scenarios

6.5 Discussion

The results indicated that infill development increased peak flow, runoff volume and flooding in the catchment. Comparing the flow in 1993 to that projected in 2040, mean annual flow volume increased by 30%, the one year ARI peak flow increased by 23% and the frequency of floods in excess of the theoretical design capacity had reduced from 2.6 years to 1.7 years (based on a single flood assessment point).

The effectiveness of WSUD varied when installed in conjunction with new development. Retention tanks had beneficial impacts on runoff volume, peak flows and flooding in the catchment. The adoption of 1 kL and 5 kL retention tanks on new homes reduced catchment mean annual runoff by 4.3% to 6% in 2013 and 7.1% to 9.9% in 2040, but these values do not reach the 12.4% reduction required in 2013 or the 22.9% reduction required in 2040 to restore 1993 flows. Similarly, while peak flow rates from the catchment were improved by retention, the simulated results indicated that 1 kL and 5 kL retention tanks had limited capacity to reduce peak flow rates to levels observed in 1993 under the assumed conditions. Retention tanks when installed with new homes (2 per redeveloped allotment) with a connected roof area of 100 m^2 per tank and 100 L/day demand (see Section 6.3.8 what this usage represents) reduced the 1 year ARI peak flow by 3.6% and 6% in 2013, respectively, compared to the 12% required to restore 1993 peak flow rates. The frequency and severity of flooding was improved by the retention tanks. In 2013, the 1993 flood frequency was restored by implementing 1 kL and 5 kL tanks, but this was not achieved in the 2040 scenario.

Detention tanks showed an ability to reduce the peak flow rate and flood frequency in the catchment when applied to new housing, with results comparable to retention tanks. The application of tanks in the size range of 1 kL to 5 kL would appear to be the most effective, with an orifice of 20 mm to 40 mm. Detention tanks were not able and nor were they intended to reduce stormwater runoff volume. The adoption of 5 kL detention tanks on new homes with a 30 mm orifice reduced the 1 year ARI peak flow rates of the 2040 scenario infill scenario by 9.9% compared to the 21.7% reduction required to restore the pre-infill peak flow rate in 1993. The influence on flooding was also similar to retention tanks. For example, detention tanks were able to restore the ARI of critical flooding at the flood assessment point in in 2013 when applied to new homes, but not in 2040.

Generally speaking, the benefits of increasing retention and detention tanks to volumes greater than 5 kL yielded little extra benefit with the 100 m^2 connected impervious area. However this relationship may change should it be reasonable to assume larger allotment impervious areas can be connected to the tank, or if tank overflows were reasonably assumed to drain over a pervious area before reaching the street drain. It was also found that the influence of retention tank demand was not significant above approximately 330 L/day. However, this volume may also be influenced by the connected impervious area to the tanks.

When retention and/or detention tanks were applied to all homes in the catchment (existing and new housing) in 2040, results indicated that the retention and detention systems were highly effective. For example 1 kL retention tanks were almost able to restore runoff volumes to levels in the 1993 scenario, while 5 kL tanks were less than the 1993 scenario. A complete retrofitting with 1 kL retention tanks could not restore the peak flow rates or flooding at the assessment point to 1993 levels but 5 kL retention tanks reduced peak flows to levels lower than those in 1993 for all ARIs up to and including the 5 year ARI, and also increased the ARI of the critical flood (reducing the flood frequency) to levels better than 1993. A complete retrofit with detention tanks had similar results. The assumption of 1 kL detention tanks on all homes in 2040 could not restore peak flow rates or the occurrence of flooding to 1993 levels, but 5 kL tanks produced peak flow rates lower than those in 1993 and improved the flood frequency.

A comparison of results for the on-site retention and detention tank scenarios with equal tank size indicated that there was little difference between the resulting peak flow rates or occurrence of flooding. Ignoring any cost and maintenance implications, the benefits of flow volume reduction and potential provision of an alternative water source to households (in the case of a rainwater tank) indicates that retention tanks have a greater improvement on catchment runoff management. However it should be stressed that this assumption is dependent on the nature of the assumed connected impervious area to tanks, and the assumed retention tank demand. Reduction in demand to levels lower than 100 L/day or reductions in the connected impervious area to levels less than 100 m^2 may influence this finding. In addition, due to the absence of any concern of surface runoff quality in a detention system, it may be easier to increase the connected impervious area of underground detention tanks than rainwater tanks. The inclusion of surface runoff in a rainwater tank introduces higher risk which may not be acceptable for end use in the home without greater consideration of water quality risks (NRMMC, EPHC and NHMRC, 2009).

The application of street scale bioretention on each side of the road at 100 m intervals was generally effective at restoring the 1993 peak flow rates and flood volumes in the 2013 development scenario, especially in 2013. In the 2040 scenario, the flood frequency was maintained at 1993 levels, however, the peak flow rates of the one, two and five year ARI were not maintained. The relative effectiveness of bioretention may be attributed to the connected impervious area to each system, which was much higher than that of the on-site retention and detention systems which were assumed to be restricted part of the allotment roof.

There are several opportunities for further research based on the findings of the Frederick Street simulation. Firstly, there was little difference between the performance of retention and detention tanks for preserving peak flow rate and flooding. The selection of either means for flow management would therefore benefit from a detailed assessment of the cost of implementing and operating on-site retention and on-site detention with new infill development. Furthermore, the cost of retrofitting the catchment with these solutions could also be explored, to compare the cost of this with an upgrade to the existing drainage system. An assessment should consider the cost effectiveness of street scale rain gardens, especially where they may be installed in conjunction with scheduled road works. Further research should also be undertaken to ensure that these results apply to other catchments: for example, catchments with higher slope, larger catchment area (e.g. greater than 100 Ha) or with different rainfall and evapotranspiration conditions. The results in this analysis could also be improved by maximising the assumed connectivity of new impervious areas to on-site retention and detention. Opportunities also exist to improve street scale rain garden design specifically for providing flow management outcomes.

The practicality of on-site retention or detention measures for a catchment should also be considered. For example, the space required for these systems on redeveloped lots may require the consideration of underground retention and detention systems, which will impact on cost. Likewise, a detention system is not always simply going to be able to drain by gravity to the existing street system, as has been assumed in this study. For a given catchment, the cost of implementing retention and detention systems should consider the means of connecting systems to in house reuse or to drainage systems.

It should also be noted that there is an additional pipe located in the catchment which was not present in 1992 when the initial monitoring of the catchment was conducted. Should there be additional monitoring undertaken at the Frederick Street drain, it is recommended that the true outflow point of this pipe and the condition of the inlet pits is inspected before undertaking analysis of any outflow rates or volumes with respect to catchment characteristics.

6.6 Summary

The impact of infill development was assessed using the Frederick Street catchment in Glengowrie as a case study. The site is a low gradient, fully urbanised catchment. The mean annual runoff, peak flow rate and frequency of flooding at key points in the 44.7 Ha catchment was determined based on simulating 19 years of flow from a calibrated model representing the 1993 development scenario, and the change in these values was quantified based on increases in the catchment imperviousness due to infill development levels observed in 2013 and projected to 2040. Results indicated that infill development increased the mean annual runoff, increased the peak flow rate at the end of the catchment and reduced the flood capacity of the catchment. Retention and detention on new homes of subdivided allotments, with typical impervious area connections (100 m^2 roof), contributed to but could not fully restore the pre-infill development flow regime of a catchment. Higher levels of connected impervious area, achieved by implementing tanks to all existing and new homes, did restore the flow regime to pre-infill development levels. There was little difference between the peak flow and flood reduction benefits achieved by on-site retention and detention storages. Retention systems may be considered to provide additional benefits based on their ability to reduce flow volume. Street scale rain gardens, which may be assumed connected to all upstream impervious area, were effective at restoring the flow regime up to a limited extent of infill development, but their effectiveness was restricted by storage capacity.

7 Case Study 4 – Paddocks Catchment

7.1 Introduction

The Paddocks catchment is located in Para Hills, part of the City of Salisbury local government area. The catchment is approximately 76 Ha in size. Runoff from the catchment drains into engineered wetlands at the base of the Para Hills escarpment. This water may currently be harvested and stored via an aquifer storage and recovery scheme at the Paddocks wetlands. The catchment layout and general location are presented in Figure 7-1.

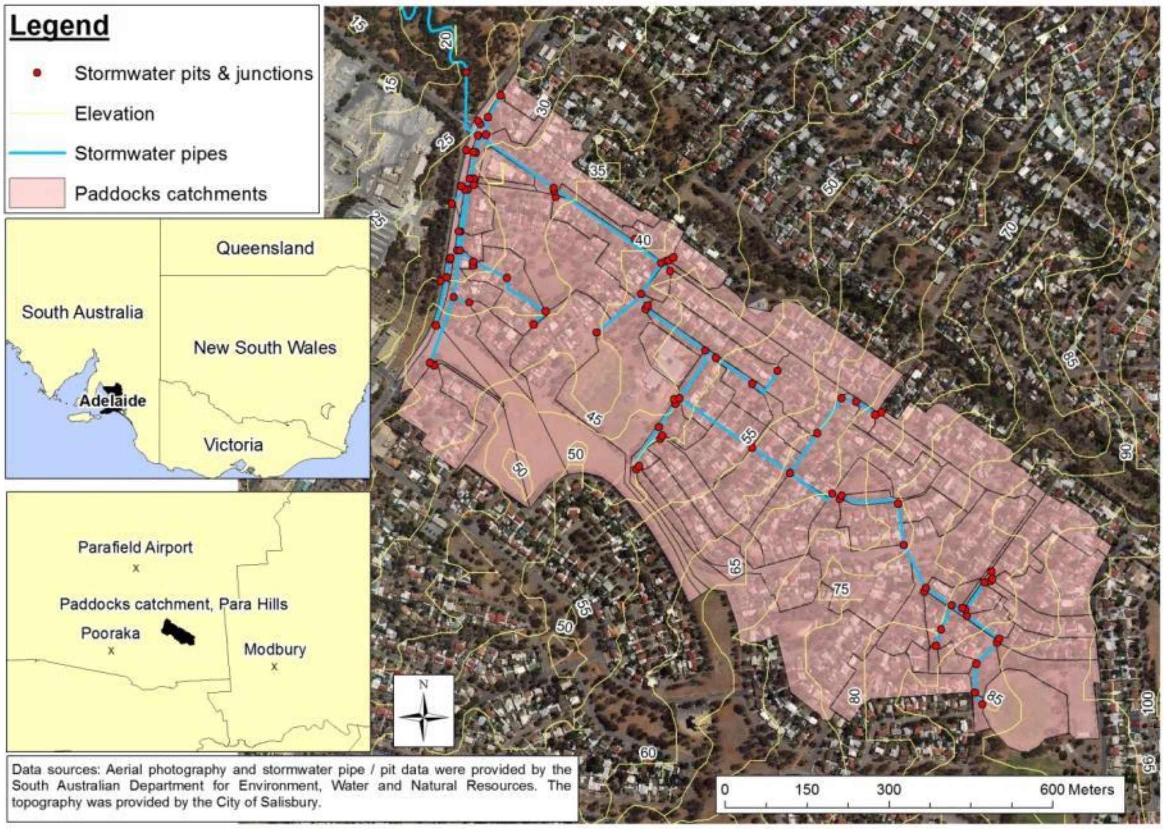


Figure 7-1 – Location of the Paddocks, indicating surrounding suburbs

The nature of the catchment has been described by previous authors (Kemp, 2002; Tomlinson et al., 1993). In brief, the Paddocks catchment has been fully urbanised with most development occurring in the 1950s and 1960s. Development has been largely residential with a small area of commercial land use. The underlying soils have been described as sandy to clay soils with abundant lime. The catchment is at a greater slope than the Frederick Street catchment, with an average slope toward the north west of approximately 5%, characteristic of its location on the escarpment of the Adelaide Hills. A review of aerial photography at the site between 1993 and 2007 indicated there has been little redevelopment since this time. Despite the lack of redevelopment occurring at present, the site was selected as an indicator of locations where infill development may occur on sloped catchments. The Paddocks catchment was selected as an indicator for this condition because it was the only confined, urbanised catchment area from which rainfall and runoff data was available for the production of a calibrated model.

It should be noted that the Paddocks is immediately upstream of an aquifer storage and recovery scheme at the Paddocks wetlands, immediately West of the catchment. As such, for this particular catchment, a reduction in stormwater runoff volume is not necessarily considered a goal of the study. However control of runoff flow rate is a priority to prevent scouring or overflow from the wetlands. In addition, volume reduction scenarios were also explored to identify opportunities for this on catchments with high slopes in other areas of Greater Adelaide.

7.2 Aims

There were two aims to the infill development case study of the Paddocks catchment:

- To estimate the impact of infill development on a medium sized catchment with a high slope
- To explore the opportunities to overcome these impacts with WSUD tools

7.3 Methodology

To examine the impact of the increased development density on peak flow, flooding and runoff volume from the Paddocks catchment, and potential WSUD solutions to ameliorate this impact, the eight steps in Section 3.4 were followed. Site selection (Step 1) involved a review of available flow data in Greater Adelaide, with the selection of the Paddocks to represent a medium size, high slope catchment. Further information on the site selection and characteristics is provided in Section 7.3.1. The computer software for the analysis (Step 2) was PCSWMM, selected in accordance with Section 3.4.2. The model was assembled based on the data and assumptions outlined in Section 7.3.2. The model was calibrated and verified in accordance with the procedures in Section 3.4.4, with the results of calibration and verification provided in Section 7.3.3 and Section 7.3.4. The scenarios used to assess the impact of infill development on the catchment flow, and the potential for WSUD to overcome these changes (Step 5) are described in Section 7.3.6. Details of the assumed nature of potential WSUD solutions were identical to those in Frederick Street, but are briefly described in Sections 7.3.8 (retention based systems) 7.3.9 (detention based systems) and 7.3.10 (street scale rain gardens). The long term continuous rainfall data applied in hydrological modelling (Step 6) was the 19 years of data from Parafield Airport described in Section 3.4.6. Peak flow and runoff volume analysis of simulated flows (Step 7) was conducted in accordance with the procedures in Section 3.4.7. The method used to assess the current levels of flooding in the catchment and ways this may be ameliorated using WSUD is described in Section 7.3.5. The results of the analysis (Step 8) are presented and compared in Section 7.4 with a summary in Section 7.5.

7.3.1 Site Selection

The Paddocks catchment was selected for Case Study 4 because:

- It has a catchment size of 76 Ha, which represents a manageable size model,
- it has a medium slope, and

- is one of few sites in the greater Adelaide region where flow data was available for the production of a calibrated model for continuous simulation of an urbanised catchment

7.3.2 Model Assembly

Previous Modelling

Previous modelling of the Paddocks catchment was available from Kemp (2002) and Scott (1994), both of which used the ILSAX model to simulate isolated runoff events. Scott (1994) utilised drainage maps, aerial photography and site visits to delineate the Paddocks catchment into subcatchments. Work was also undertaken in the field to determine characteristic values of directly connected impervious area (A_{DCIA}), indirectly connected impervious area (A_{ICIA}) and pervious area in residential allotments. This data was compiled to produce a valuable pool of data for future modelling.

Kemp (2002) acquired data from the City of Salisbury, including the information from Scott (1994), and applied this information to produce an ILSAX model for the Paddocks catchment. The model was used to verify the suitability of ILSAX to simulate runoff from urban catchments. Kemp (2002) found that the ILSAX model did not provide a good estimate of volume and peak flow initially, but that this was improved by increasing the directly connected impervious area of the overall catchment by 8.6%, reducing the pipe Manning's 'n' value from 0.012 to 0.011 and adjusting the GUT factor of the ILSAX model. There was no runoff found to occur in the available flow data so no loss model parameters could be determined. Parameter fitting was carried out using the PEST parameter estimation program (Doherty, 2010). Other key parameters of the model discussed by Kemp (2002) are provided in Table 7-2.

Table 1 – Key properties of the Paddocks catchment model developed by Kemp (2002)

Model parameter	**Value**
Impervious area depression storage (loss)	0 mm
Pervious area depression storage (loss)	5 mm (not calibrated)
Pervious area roughness, N_{perv}	0.03 (no units, not calibrated)
Pipe roughness, N_{pipe}	0.011
Catchment slope	5 % (average value, not calibrated)
GUT Factor	9.51
Soil infiltration rate – Initial	130.1 mm/h (not calibrated)
Soil infiltration rate – Final	13 mm/h (not calibrated)
Shape factor	2 h^{-1} (not calibrated)

These parameters were used as a starting point for a calibration of equivalent parameters in the SWMM model of the Paddocks catchment. For modelling purposes, the slope of each catchment was individually determined based on contours. The percentage of connected impervious area, indirectly connected impervious area and pervious area in each subcatchment was adopted based on the recommendations of Kemp (2002). There was no GUT factor in the SWMM model, however the GUT factor may be considered as somewhat comparable to the catchment width in SWMM. In DRAINS, the GUT factor accounts for overland travel time in combination with a lag parameter, both of which are lumped into the catchment width parameter in SWMM. A full description of catchment width is provided by Rossman (2010). Briefly however, for the Paddocks catchment, width represents the distance that water must flow to a drainage point in a catchment. It was estimated using the procedures recommended by Rossman (2010), based on the catchment area and the length of overland flow, which was assumed to be 25 m (based on the approximate distance of travel of runoff from housing lots to gutters).

Catchment characteristics

Situated in the foothills north-east of Adelaide, the 75 Ha Paddocks catchment had a mean slope of approximately 5%. The catchment area was almost wholly residential with a school and commercial development in the north west. Originally developed in the 1960s, only one allotment appears to have undergone subdivision in the period of 1993 to 2013. Table 6-2 summarises the characteristics of the catchment based on the site analysis in 1993.

Table 7-1 – Summary of catchment properties for the Paddocks catchment

Catchment property	Value
Total area (Ha)	75.3
Directly connected impervious area (%)	26
Indirectly connected impervious area (%)	16
Pervious area (%)	58

The layout of stormwater pipes, junctions and side entry pits was derived from mapping data available from the South Australian Department for Environment, Water and Natural Resources (DEWNR) and is also illustrated in Figure 7-1. When this data was cross checked with the data from previous field work in the catchment by Scott (1994) and modelling work by Kemp (2002) there were constant discrepancies in the pit and pipe elevation data and minor disagreements on the drainage system layout. The pit, pipe and surface levels provided by DEWNR and the slope data in the model from Kemp (2002) tend to differ, and may be a result of using different sources of information in model compilation. Where data was available, the work of Scott (1994) and Kemp (2002) was used for model construction as this was based on documented field verification work. The verified surface levels provided by Scott (1994) were also applied where available. Although the surface elevation data was not strictly required for the previous cases of ILSAX modelling, it was important in the SWMM model for the determination of surface slope and simulation of major flow paths (road surfaces).

Climate and Flow Data

There were three monitoring stations used during the period over which flow and rainfall was monitored in the Paddocks catchment. These monitoring stations are described in Table 7-2. The location of these gauges is shown in Figure 7-2. Most notably, all gauges lie within the catchment boundary.

Table 7-2 – Description of flow and rainfall gauges in the Paddocks catchment

Station	Number*	Data available	Dates	Missing/Errors
Para Hills Drain at Paddocks inlet	*AW504546* **A5040546**	Water level	31/08/1990 – 22/10/2004	15.4%
Leichardt Avenue	*AW504566* **M523006**	Rainfall	14/04/1992 – 31/12/2002	0.45%
Paddocks catchment pluviometer at Joslin Avenue	*AW504567* **A5040567**	Rainfall	14/04/1992 – 08/05/2002	2.9%
AWXXXXXX* indicates a now defunct site code which may be used in previous literature; **AXXXXXXX represents a site code current at time of writing.				

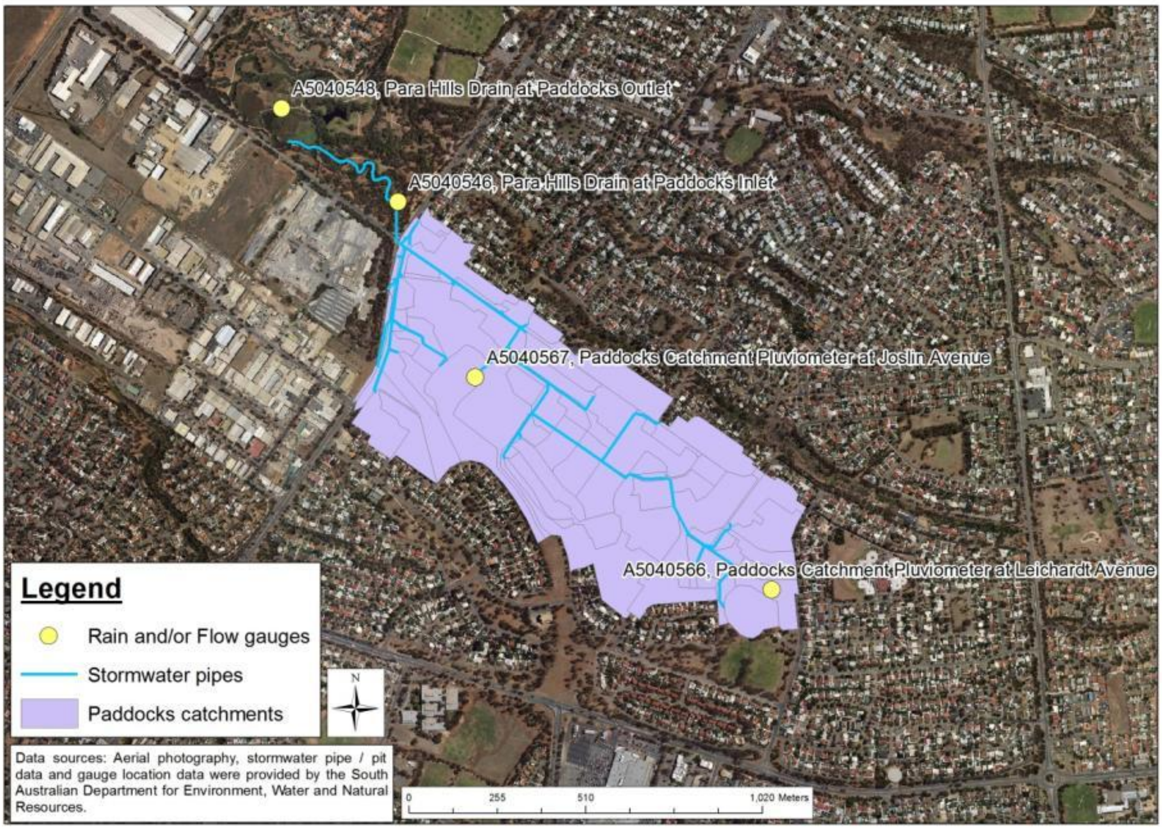


Figure 7-2 – Location of gauging stations within and near the Paddocks catchment

The long-term and quality controlled climate station nearest to the Paddocks catchment is the Bureau of Meteorology station at Parafield Airport (023034). As background information, the mean annual rainfall at the Parafield Airport station has been documented as 453 mm, with the mean monthly rainfall shown in Figure 7-3. The mean annual rainfall of the two rainfall gauges located inside the catchment boundary are also shown for comparison with the Parafield Airport data in Figure 7-3. In each case, mean monthly rainfall is based on the mean of monthly rainfall over a 10 year period. The Parafield Airport rain gauge is situated at an elevation of 10 m and approximately 3 km north-west of the catchment boundary. The elevation of the Paddocks catchment varies from approximately 20 m at the western boundary to 95 m at the eastern boundary (Figure 7-1), and variation in rainfall may be due to this elevation change. The nearest evaporation gauge is located at Parafield Airport, with a mean annual evaporation of 2080 mm. Mean monthly evaporation shown in Figure 7-4. The mean monthly evaporation data from Parafield Airport was used to simulate the effect of evaporation on catchment surface storages in the SWMM model for the Paddocks.

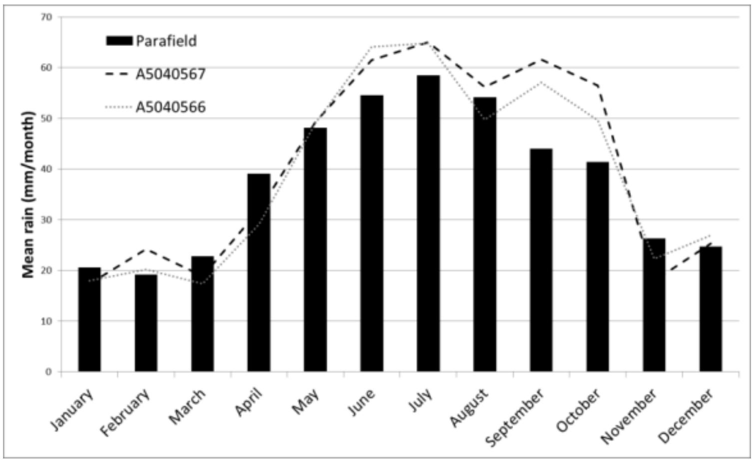


Figure 7-3 – Mean monthly rainfall at the Parafield Airport BOM gauge (023013) and for the two Frederick Street catchment gauges

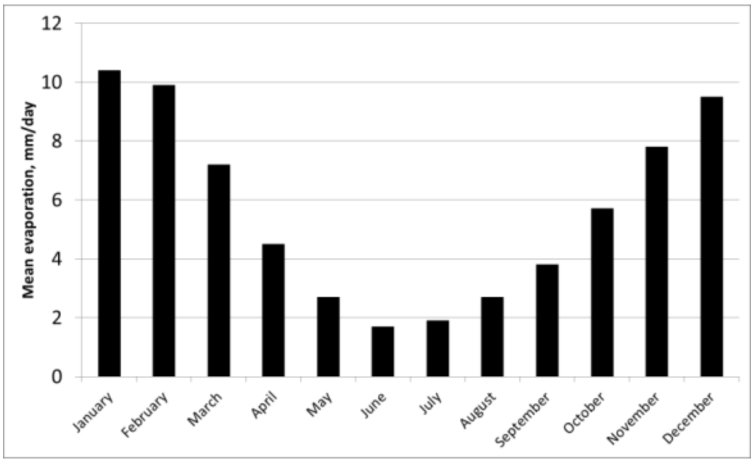


Figure 7-4 – Mean daily evaporation at the Parafield Airport BOM gauge (023013)

Observed flow for the Paddocks catchment was available from the flow gauge at the entry to the Paddocks wetlands (A5050546, Table 7-2). The quality of data from this flow gauge was assessed with respect to rainfall and found to be generally good, although 15% of the data was noted to be of compromised quality. Data from these periods were excluded from the calibration and verification events.

Other Assumptions

- The calibration and verification of the model was dependent on the accuracy of the flow gauge at the entry to the Paddocks wetlands. It was assumed that this data was fit and proper for model calibration, despite some concerns regarding the operation of the gauge. Concerns included the lack of any evident runoff from pervious areas, and competing arguments regarding over-estimation and under-estimation of flow by the gauge reported by Tomlinson et al (1993).

7.3.3 Model Calibration

For comparison with previous work, calibration and verification events were selected from the events used by Kemp (2002) for calibration of an ILSAX model of the Paddocks catchment. In some cases, the event period used in this report was extended to capture full event hydrographs. Events from October 1992 to December 1993 were used for calibration, which provided fifteen events. The characteristics of these events are shown in Table 7-3.

Table 7-3 – Calibration events for the Paddocks catchment model

#	Date	Time	Observed Peak Flow (m^3/s)	Rainfall (mm)		Observed runoff volume (m^3)
				A5040566	A5040567	
Cal 1	3/10/1992	1600 to 2100	1.407	11.4	11	1732
Cal 2	8/10/1992	0200 to 1800	0.964	31.8	28.6	5148
Cal 3	8/10/1992	1930 to 0000	1.286	9.2	11.6	2437
Cal 4	17/11/1992	1130 to 1600	2.239	22.8	22.6	3417
Cal 5	20/11/1992	2200 to 0400	0.772	14.4	13.4	1900
Cal 6	18/12/1992	1600 to 2200	1.453	17.2	12.6	2294
Cal 7	19/12/1992	1300 to 1500	2.465	19.2	19.4	3518
Cal 8	27/02/1993	2200 to 0100	0.866	9.2	9.2	1551
Cal 9	21/05/1993	1200 to 1700	1.378	20.8	17.6	2325
Cal 10	3/06/1993	1630 to 1830	1.144	11.2	10.2	1564
Cal 11	11/06/1993	1400 to 1600	0.943	3	5	672.6
Cal 12	30/08/1993	1700 to 1830	1.391	10.2	11.2	1834
Cal 13	17/10/1993	0800 to 1400	1.048	14	14.4	2111
Cal 14	18/10/1993	0600 to 1100	1.054	10.2	8.4	1220
Cal 15	13/12/1993	2230 to 0000	1.67	13.2	8	1467

Figure 7-5 shows a plot of the total observed flow and rainfall volume for each of the calibration events, as well as the verification events described in Section 7.3.4.

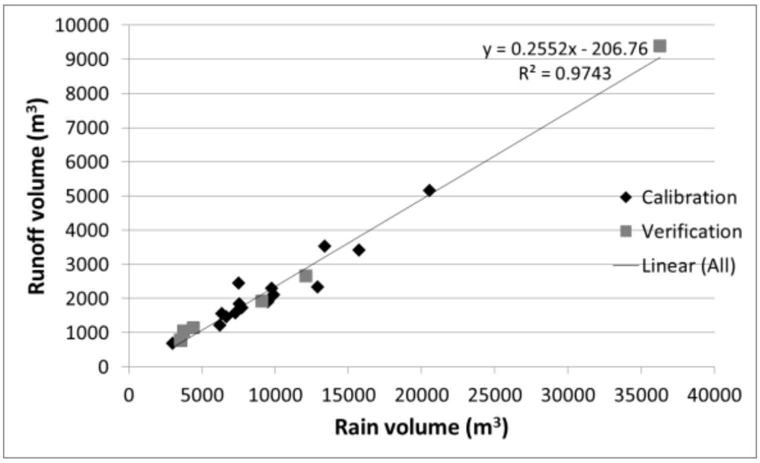


Figure 7-5 – Comparison of observed rainfall and runoff volumes for calibration and verification events in the Paddocks catchment

A detailed explanation of catchment parameters is provided by Rossman (2010). In summary, during calibration, adjustments were made to the assumed values of the following parameters, ensuring that values stayed within reasonable limits based on the known catchment characteristics and the recommendations of Rossman (2010):

- Manning's N values of impervious area (N Imperv)
- Manning's N value of pervious areas (N Perv)
- Manning's N value of pipes
- Catchment width (not included in ILSAX model)
- Horton Infiltration parameters:
  - Maximum infiltration rate
  - Minimum infiltration rate
  - Decay constant
  - Drying time (not included in ILSAX model)
  - Maximum volume (mm)

Initial attempts at calibration showed that the model was capable of replicating events to a reasonable degree, with results rated well by the Nash Sutcliffe coefficient of fitness. However, there was difficulty in simulating peak flows effectively, which was considered an important capability for the peak flow analysis for which the model was intended. By applying the automatic calibration tools in PCSWMM, which provides an estimate of the sensitivity of the predicted hydrograph to model parameters, the parameters adopted are those shown in Table 7-4.

Table 7-4 – Calibrated parameters of the PCSWMM model of the Paddocks catchment

PCSWMM Parameter	Final value
Percent impervious (%)	8% less than estimated
Width (m)	Based on flow length = 15 m
Manning's 'n' – Impervious area	0.012
Manning's 'n' – Pervious area	0.15
Manning's 'n' – Pipes	0.012
Impervious area storage (mm)	1.0
Pervious area storage (mm)	3.0
Maximum infiltration rate (mm/hr)	100
Minimum infiltration rate (mm/hr)	6
Decay (1/hr)	2
Drying time (Days)	7
Maximum volume (mm)	50

The results of the calibration procedure for the events previously described in Table 7-4 are presented in Table 7-5. It includes the percentage error in peak flow (PEP), percentage error in volume (PEV), Nash Sutcliffe efficiency (r^2) and the sum of squared residuals (G) for each individual event in the continuous time series.

Table 7-5 – Fit of the simulated to observed flow data for calibration events of the Paddocks model

Event	Simulated peak	Observed peak	PEP	PEV	R^2	G
Cal 1	1.30	1.41	-7.6	1.9	0.90	0.75
Cal 2	0.82	0.96	-15.1	-0.5	0.88	0.57
Cal 3	1.17	0.00	-9.3	8.5	0.91	0.78
Cal 4	2.22	2.24	-0.8	13.6	0.97	0.74
Cal 5	0.76	0.77	-1.3	18.4	0.89	0.35
Cal 6	1.52	1.45	4.5	-4.9	0.96	0.36
Cal 7	2.76	2.47	11.8	-4.2	0.95	1.89
Cal 8	0.75	0.87	-13.8	-7.3	0.97	0.13
Cal 9	1.37	1.38	-0.7	16.4	0.95	0.42
Cal 10	1.15	1.14	0.6	4.4	0.96	0.24
Cal 11	1.02	0.94	7.8	14.7	0.91	0.20
Cal 12	1.40	1.39	0.5	-2.8	0.79	2.10
Cal 13	1.26	1.05	19.9	8.0	0.88	0.80
Cal 14	1.00	1.05	-5.0	14.1	0.91	0.38
Cal 15	1.83	1.67	9.5	11.5	0.86	1.37

The fit of the simulated to the observed hydrograph is illustrated in Figure 7-6 and Figure 7-7. Remaining results are shown in Appendix B.

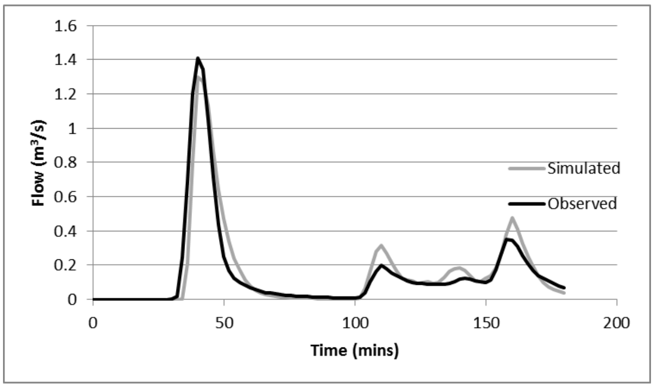


Figure 7-6 – Comparison of the observed and simulated flows for Paddocks event Cal 1 (3 October 1992)

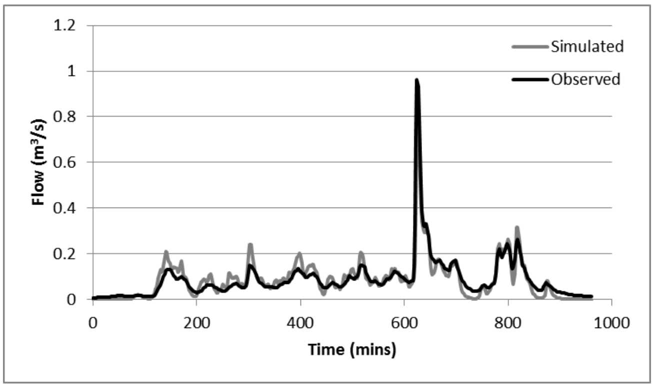


Figure 7-7 – Comparison of the observed and simulated flows for Paddocks event Cal 2 (8 October 1992, #1)

7.3.4 Model Verification

Model validation was conducted to check that the model calibration had not provided a fit suitable only to the calibration period. In the process of model validation, events were selected from the observed flow time series (from June to December 1993) to examine whether the calibrated model was able to adequately predict peak flows outside the initial calibration period without any further adjustment. Peak flow was again prioritised by using the r^2 and PEP values as primary indicators. For this reason, observed flow events for validation were selected where observed flow was in excess of 0.8 m^3/s. Events were selected such that flows 3 hours prior to and following the peak flow event were examined. There were nine events selected in the observed time series, with the characteristics shown in Table 7-6.

Table 7-6 – Verification events for the Paddocks catchment model

#	Date	Time	Observed Peak Flow (m^3/s)	Rainfall (mm)		Observed runoff volume (m^3)
				A5040566	A5040567	
V1	14/05/1994	0200 to 0530	1.766	14.6	12.4	1908
V2	14/06/1994	0130 to 0400	1.237	4.6	5.8	1049
V3	2/05/1995	1100 to 1230	1.098	4.2	5.6	761.7
V4	25/05/1995	0600 to 1530	1.358	14.6	18.8	2643
V5	21/07/1995	1530 to 1830	1.053	5.9	6.6	1131
V6	1/08/1995	1700 to 2000	0.836	4.5	5.4	773.1
V7	31/12/1995	1430 to 1700	2.761	47.2	54.7	9380

The results of the validation check are shown in Table 7-7, with selected hydrographs shown in Figure 7-8 and Figure 7-9. The results show that the model still provides a good prediction of peak flows outside of the calibration period, with the exception of events V2, V4 and V5.

Table 7-7 – Fit of the simulated to observed flow data for verification events of the Paddocks model

#	Observed Peak Flow (m^3/s)	Simulated Peak Flow (m^3/s)	PEP	PEV	R^2	G
V1	1.75	1.77	-0.8	9.5	0.99	0.21
V2	0.87	1.24	-29.8	-20.1	0.92	0.45
V3	1.20	1.10	9.7	8.7	0.95	0.16
V4	1.09	1.36	-19.7	6.7	0.95	0.52
V5	0.81	1.05	-22.7	-11.6	0.94	0.26
V6	0.76	0.84	-9.6	3.7	0.93	0.14
V7	2.54	2.76	-8.0	-16.9	0.95	4.00

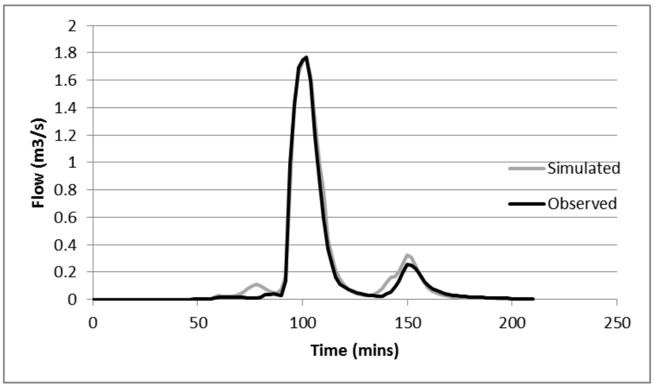


Figure 7-8 – Comparison of the observed and simulated flows for Paddocks event V1

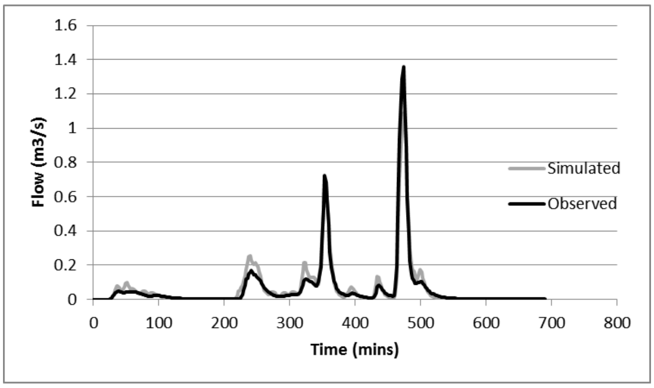


Figure 7-9 – Comparison of the observed and simulated flows for Paddocks event V4

For event V4, the rainfall gauge is considered to have had some influence over the prediction of peak flow. There is a cessation of rainfall at gauge A5040556 corresponding with a peak rainfall intensity at gauge A5040567. This is clearly evident in the excerpt of event V4 corresponding with the peak flow value, shown in Figure 7-10.

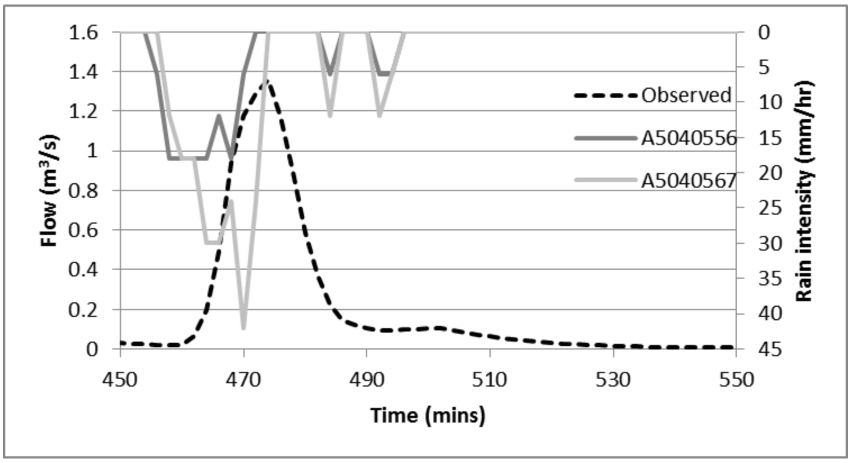


Figure 7-10 – Excerpt of event V4, showing the suspicious cessation of rainfall at one gauge

The difference in rainfall readings may be attributable to potential errors like these at the rainfall gauges. It may also be attributable to the limitation of the model to only two rain gauges. Each catchment has been attributed a single rainfall gauge based on proximity, and in reality the boundaries of this rainfall event at peak intensity might have differed from the assumed boundary. A similar effect may also explain the difference for events V2 and V5. However, as all events showed an excellent value for the Nash Sutcliffe efficiency (r^2), and a majority of events had an reasonable level of accuracy for the peak flow rate and volume prediction (< 10% difference in both cases), the model was accepted as sufficiently calibrated.

7.3.5 Assessing Flooding in the Paddocks Catchment

Flooding in the Paddocks catchment was examined at a representative location in the catchment, namely a single grated drainage pit on Bridge Road at the base of the catchment. The occurrence of flooding in the PCSWMM model was indicated by temporary ponding of water above stormwater pits unable send water into subsurface pipes (due to an at-capacity inlet, an at capacity drainage pipe or a pipe surcharge) nor send water downstream via the road carriageways (due to the location of the pit at a sag point). The extent and frequency of flooding was recorded throughout the catchment and key areas of flooding were identified using this data. Of these locations, the 'indicator' location was selected based on the frequency of events which occur above a volume threshold considered likely to disturb local traffic, and the exposure of this location to catchment wide conditions. The tendency of this location to flood was reinforced by site visits during rainfall in February 2014. The impacts of flood volumes along Bridge Road cause ponding on the road surface and a hazard to passing traffic as shown in Figure 7-11.

Figure 7-11 – Nuisance flooding at the Paddocks catchment case study location in February 2014

The selected flow threshold in the Paddocks catchment was selected using the same criteria as Frederick Street described in Section 6.3.5, where the goal was to preserve one dry lane available for traffic to pass. At this location, the depth of water that causes flooding to occur in excess of this criteria was 150 mm – higher than the acceptable depth in Frederick Street case because Bridge Road is a dual carriageway at this location with two lanes available for traffic in each direction. The corresponding flood volume was 9.4 m^3.

7.3.6 Model Scenarios

There were a large number of scenarios used to explore runoff characteristics of the Paddocks catchment. These can generally be divided into two main groups. All scenarios were undertaken using one of three *development scenarios*. These development scenarios were used to identify the impact of infill development on the peak flow and runoff volume from the catchment. The details of each development scenario are provided in Section 7.3.7. The development scenarios were then used as the basis for *WSUD scenarios* to investigate the impact that WSUD systems might have on mitigating the changes in peak flow and runoff volume due to the infill development. The assumptions behind WSUD scenarios are detailed in Section 7.3.8 for onsite retention systems (e.g.

rainwater tanks or infiltration systems), Section 7.3.9 for on-site detention systems (e.g. detention tanks) and Section 7.3.10 for street scale bioretention systems.

7.3.7 Development Scenarios

Since 1993, there is only one location where subdivision has been found to occur in the Paddocks catchment. Due to the lack of infill development, theoretical scenarios were developed to simulate the impact of infill development in the Paddocks catchment. The three scenarios studies were:

1. 1993 Development – the calibration case of the model based on the known catchment properties measured in the field studies described by Scott (1994), Kemp (2002) and Tomlinson et al (1993).
2. 1993 + 25% – the calibration case model with 25% of allotments subdivided into two allotments and rebuilt with two separate dwellings
3. 1993 + 50% - the calibration case model with 50% of allotments subdivided into two allotments and rebuilt with two separate dwellings.

The key properties of each scenario are shown in Table 7-8. The following paragraphs describe the assumptions behind the redevelopment of lots in the Paddocks catchment.

Table 7-8 – Development scenarios for the Paddocks catchment

Case	Description	Mean impervious area (%)	Mean connected impervious area (%)	Mean indirectly connected impervious area (%)
A	1993 development (calibration case)	43.3	25.1	18.2
B	1993 + 25%	48.4	29.4	18.9
C	1993 + 50%	53.5	33.9	19.7

The 1993 Paddocks development scenario was identical to that outlined by Scott (1994), Kemp (2002) and Tomlinson et al (1993). This data was subject to some alterations during calibration as outlined in Section 7.3.3. Using the data in 1993 as a basis, the changes in the catchment were then determined using the following steps:

1. Examine the nature of development in the Paddocks catchment that currently exists (e.g. lot size)
2. Project future development based on subdivision of homes on a ‘1 into 2’ basis.

The number of homes in the catchment for the 1993 scenario and the two theoretical development scenarios are shown in Table 7-9.

Table 7-9 – Current and assumed future allotment numbers in the Paddocks catchment.

	Allotments	Subdivisible allotments	New houses	No. subdivisions
1993	539	539	-	-
1993 + 25%	677	401	276	138
1993 + 50%	817	261	556	278

The second step of the process was conducted using the data for the catchment in 1993, and current aerial photography, to examine allotments and determine the mean level of:

- directly connected impervious area,
- indirectly connected impervious area, and
- pervious area.

This data was then used to produce a representative allotment from 1993 and a representative infill allotment (following subdivision). The characteristics of the current allotment and assumed future allotments are presented in Table 7-10. The current allotment characteristics were based on selection of a portion of the Paddocks catchment. The portion had 181 homes in total. The redeveloped allotment was based on the characteristics of redeveloped allotments undertaken for the Frederick Street catchment in Section 6.3.7.

Table 7-10 – Assumed characteristics of allotments in the Paddocks catchment area

	Existing Allotment		New Allotment	
	Area (m^2)	%	Area (m^2)	%
Directly connected impervious	247.9	33.2	522.3	70.0
Indirectly connected impervious	63.9	8.6	111.9	15.0
Pervious	434.4	58.2	111.9	15.0
Impervious (total)	311.8	41.8	634.3	85.0
Total Area	746.2	-	746.2	-

Following the simulation of the 1993 scenario and the +25% and +50% version of the catchment without any WSUD features, retention and detention based WSUD scenarios were explored to determine their impact on catchment peak flow rates when applied to new dwellings.

7.3.8 On-site Retention Scenarios

The retention systems explored for the WSUD scenarios of the Paddocks catchment were identical to those examined for the Frederick Street catchment, namely rainwater tanks and infiltration systems. Each system was simulated in an identical manner, and with the same variables as identified in Section 6.3.8. The scenarios were simulated with variation in the same four key variables: storage volume, water demand or infiltration rate, connected roof area and number of tanks/systems per property. Storage volume assumptions were identical to those described in Section 6.3.8. Tank demand or infiltration assumptions were identical to those described in Section 6.3.8. The connected roof area to each tank was identical to that described in Section 6.3.8.

The number of retention tanks assumed present in the catchment was explored with two scenarios. The first was on the assumption of one retention tank per new home, and where each redevelopment site was assumed to be broken into two allotments - thus 2 tanks per redevelopment site. This resulted in 276 tanks for the 1993+25% scenario, and 556 for the 1993+50% scenario. The second was on the assumption a complete retrofit of all houses in the catchment with retention tanks. This resulted in 677 tanks for the 1993+25% scenario and 817 for the 1993+50% scenario.

7.3.9 On-site Detention Systems

The detention systems explored using the Paddocks model were on site detention tanks, identical to those outlined for the Frederick Street catchment in Section 6.3.9. The four variables used in the simulation of detention tanks were tank size (volume), tank shape, orifice size and number of detention tanks. All detention tank scenarios assume a connected roof area of 100 m^2. The detention tank volumes simulated were 1 kL, 5 kL and 10 kL, selected and applied as described in Section 6.3.9. The outflow orifice size examined was between 10 mm and 50 mm, in 10 mm increments, selected as described in Section 6.3.9. The number of detention tanks assumed was identical to that for retention tanks in Section 7.3.8 (two tanks per redeveloped allotment).

7.3.10 Street Scale Bioretention

The implementation of street scale bioretention was explored based on the arrangement of bioretention systems in Mile End (Section 5). Based on an examination of the Mile End rain gardens in Mile End, it was found that rain gardens were situated on each side of the road and at intervals of approximately 100 m. Therefore, to examine the impact of bioretention gardens in the Paddocks catchment, rain gardens were placed on road sides at intervals of approximately 100 m throughout the catchment.

In Section 6, the Frederick Street catchment was provided with one type of rain garden which connected to the existing drainage system via an underdrain. This was not possible in the Paddocks catchment because the drainage system does not generally follow roadways. As such, there were two types of rain garden applied in the Paddock catchment based on proximity to existing stormwater drainage. Rain garden type 1 was proposed in locations where it could readily connect to the existing stormwater drainage pipes. This garden was identical to rain garden RG5 (Section 5.3.2), with surface properties previously presented in Table 5-2 and drainage properties as presented in Table 5-3. The second garden type was proposed in locations where there was no nearby stormwater drainage pipe. It was not sealed like the first, and allows infiltration to occur through the base of the system only. The location of the proposed rain gardens are shown in Figure 7-12.

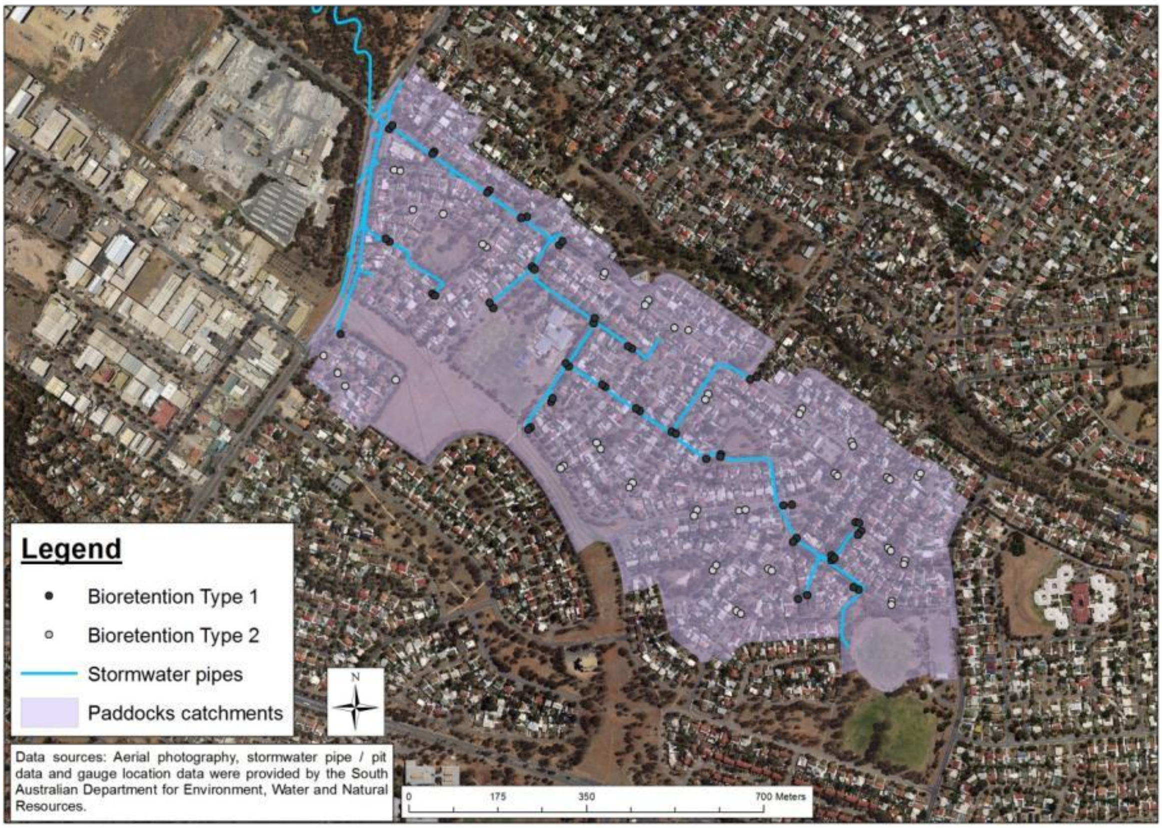


Figure 7-12 – Proposed location of bioretention in the Paddocks catchment

7.4 Results

The results of the Paddocks simulation are provided in the following sections. Section 7.4.1 describes the impact of increased development density on runoff from the Paddocks catchment. Section 7.4.2 describes the impact of retention tanks on runoff when fitted to new homes. Section 7.4.3 describes the impact of detention tanks on runoff when fitted to new homes in the catchment. Section 7.4.4 describes the impact of street scale bioretention systems on runoff from the catchment.

7.4.1 The Impact of Increased Development Density in the Paddocks Catchment

Impact of Infill Development on Runoff Volume

The simulation of the same 19 years rainfall data over the Paddocks catchment for the three development scenarios indicated that the volume of runoff from the catchment was 60.6 ML/annum in 1993, increasing by 17% to 70.9 ML/annum for the 1993 + 25% scenario. The 1993 flow volume increased by 42% to 81.3 ML/annum in the 1993 + 50% scenario. These results are illustrated in Figure 7-13.

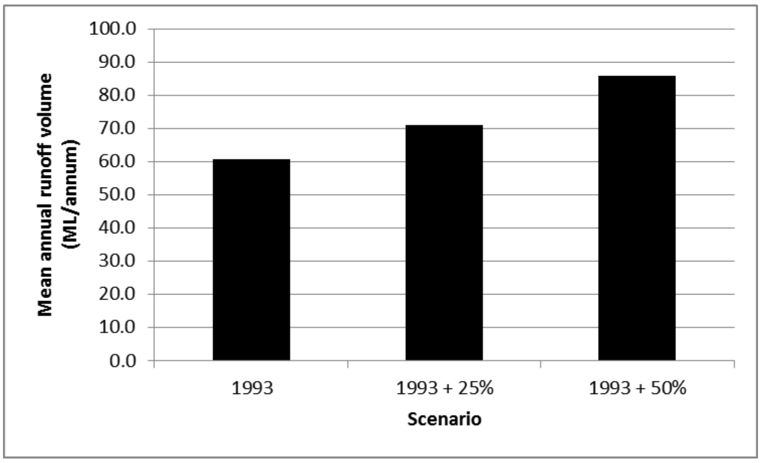


Figure 7-13 – Mean annual runoff volume of the Paddocks catchment in the 1993 scenario and the +25% and +50% scenarios

Impact of Infill Development on Peak Flow Rates

Table 7-11 shows the changes in the estimated 6 month, 1, 2 and 5 Year ARI peak flows at the outlet of the Paddocks catchment for the calibration case (pre-infill development scenario, 1993), the +25% infill scenario and the +50% infill scenario. The results in Table 7-11 indicate that the 6 month, 1, 2 and 5 year ARI peak flow rates increase as infill development takes place across the catchment. In percentage terms, the increase was higher for more frequent events. For example, the percentage increase in the 0.5 year ARI in 1993 was 14.5% and 28.5% for the 25% and 50% infill scenarios, respectively, however the percentage increase in the 5 year ARI was 3.6% and 4.1%. It is also worth indicating that up to the 2 year ARI value, flow was conveyed by the underground pipe system. However the 5 year ARI peak flow includes overland flow. The maximum flow conveyed by the pipe system during the simulation was 2.9 m^3/s.

Table 7-11 – Predicted peak flows from the Paddocks catchment in the 1993 scenario and the +25% and +50% infill scenarios based on partial series analysis

Case	0.5 Year ARI (m^3/s)	1 Year ARI (m^3/s)	2 Year ARI (m^3/s)	5 Year ARI (m^3/s)
1993	1.02	1.29	1.56	3.23
1993 + 25%	1.19	1.45	1.71	3.62
1993 + 50%	1.43	1.67	1.91	4.14

Impact of Infill Development on Flood Frequency

Based on the methodology in Section 6.3.5, flooding was assessed based on the frequency of a flood volume of 9.4 m^3 at the Bridge Street ponding location. It was also assumed that the drainage system was able to preserve road widths by ensuring flooding remains less than this value up to and including the 5 year ARI flows. The events which plotted closely to the 5 Year ARI in the partial series were those with a return period estimate of 4.2 and 5.3 years. For the 1993 case, both of these events caused flooding in excess of the threshold (250 m^3 and 122 m^3, respectively). All events in the partial series above the 0.9 year ARI caused flooding in excess of this threshold. In the +25% infill scenario, this frequency increased to include all events above the 0.5 year ARI. This increased to include all events above the 0.4 year ARI for the +50% infill scenario. It should be stressed that these flood frequency values are estimates and stated for comparative purposes in this report.

7.4.2 Effect of Retention Tanks on Stormwater Runoff when fitted to New Homes

Management of Stormwater Runoff Volume Using Retention Tanks

Runoff volume increases of up to 34% were projected to occur due to infill development as shown in Figure 7-13. The mean annual runoff volume from the catchment outlet when 1 kL, 5 kL and 10 kL onsite retention tanks were introduced in conjunction with new housing is presented in Figure 7-14 for the +25% infill and +50% infill scenarios. These scenarios represent the case where tanks were installed on all new homes (e.g. 2 tanks per redeveloped allotment) with a connected roof area of 100 m^2.

(a) (b)

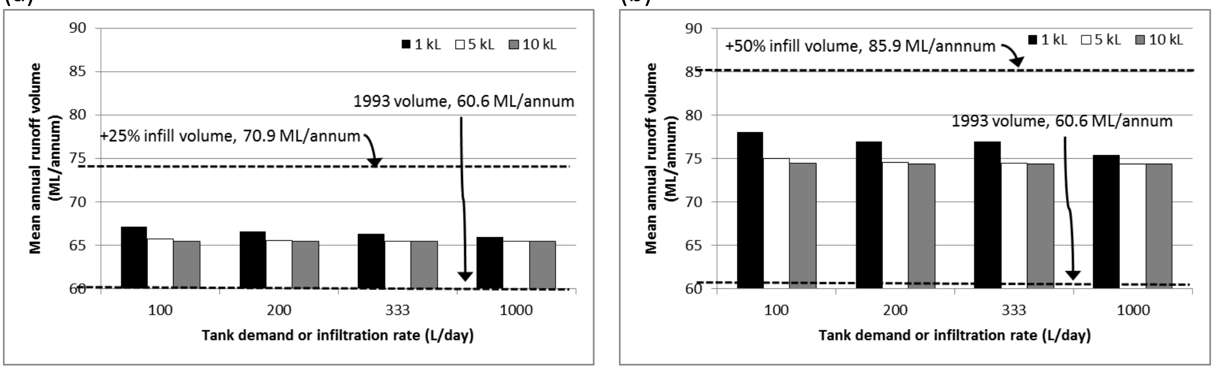


Figure 7-14 – The total runoff volume from the Paddocks catchment when applying 1 kL, 5 kL and 10kL rainwater tanks to new homes in the (a) +25% infill scenario and (b) +50% infill scenario

These results indicate that the volume of runoff in for the +25% scenario could have been reduced by approximately 5.4% to 7.4% using rainwater tanks of 1 kL to 5 kL volume, respectively, with typical levels of usage (100 L/day). The volume of runoff in the +50% infill scenario could be reduced by 9.2% to 12.7% using rainwater tanks of 1 kL to 5 kL volume, respectively. The results also indicate that there was very little benefit from increasing the tank size from 5 kL to 10 kL, with the greatest reductions occurring between 1 kL and 5 kL. In addition, there was little benefit to very high levels of use for the 5 kL and 10 kL rainwater tanks. The annual runoff volume was reasonably similar for demand/infiltration between 333 L/day and 1666 L/day cases, indicating that the optimal level of demand (or infiltration) is between 200 L/day and 333 L/day. The amount of water that may be harvested or disposed of via on site infiltration is also shown in Figure 7-15.

(a) (b)

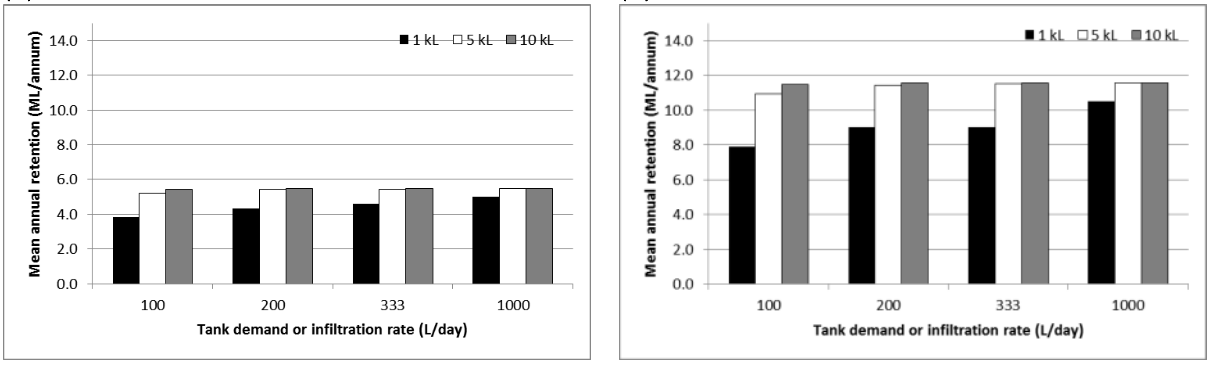


Figure 7-15 – The total retention in the Paddocks catchment when applying 1 kL, 5 kL and 10kL rainwater tanks to new homes in the (a) +25% infill scenario and (b) +50% infill scenario

The results for runoff volumes from the Paddocks catchment are notably similar to the results for the Frederick Street catchment in Section 6.4.2. There was an unusual result for the 1 kL tank scenario with high levels of use/infiltration (1667 L/day) which was not considered accurate – the routing error of the SWMM model was typically 0.1% in other scenarios, but was 0.9% for this scenario, which is suggested to be the reason for the unusually good performance of the 1 kL tank with this level of usage.

Management of Peak Flow Rates Using Retention Tanks

The peak flow rates from the catchment in 1993, for the +25% and +50% infill scenarios were presented in Table 6-13. The effect of retention systems on these peak flow rates are presented in Figure 7-16 for the +25% and +50% infill scenarios. Each diagram also compares the effect of tank size (1 kL, 5 kL or 10 kL) and tank demand (from 100 L/day to 1666 L/day). These results assume two tanks at each redeveloped site, where each has a connected roof area of 100 m^2.

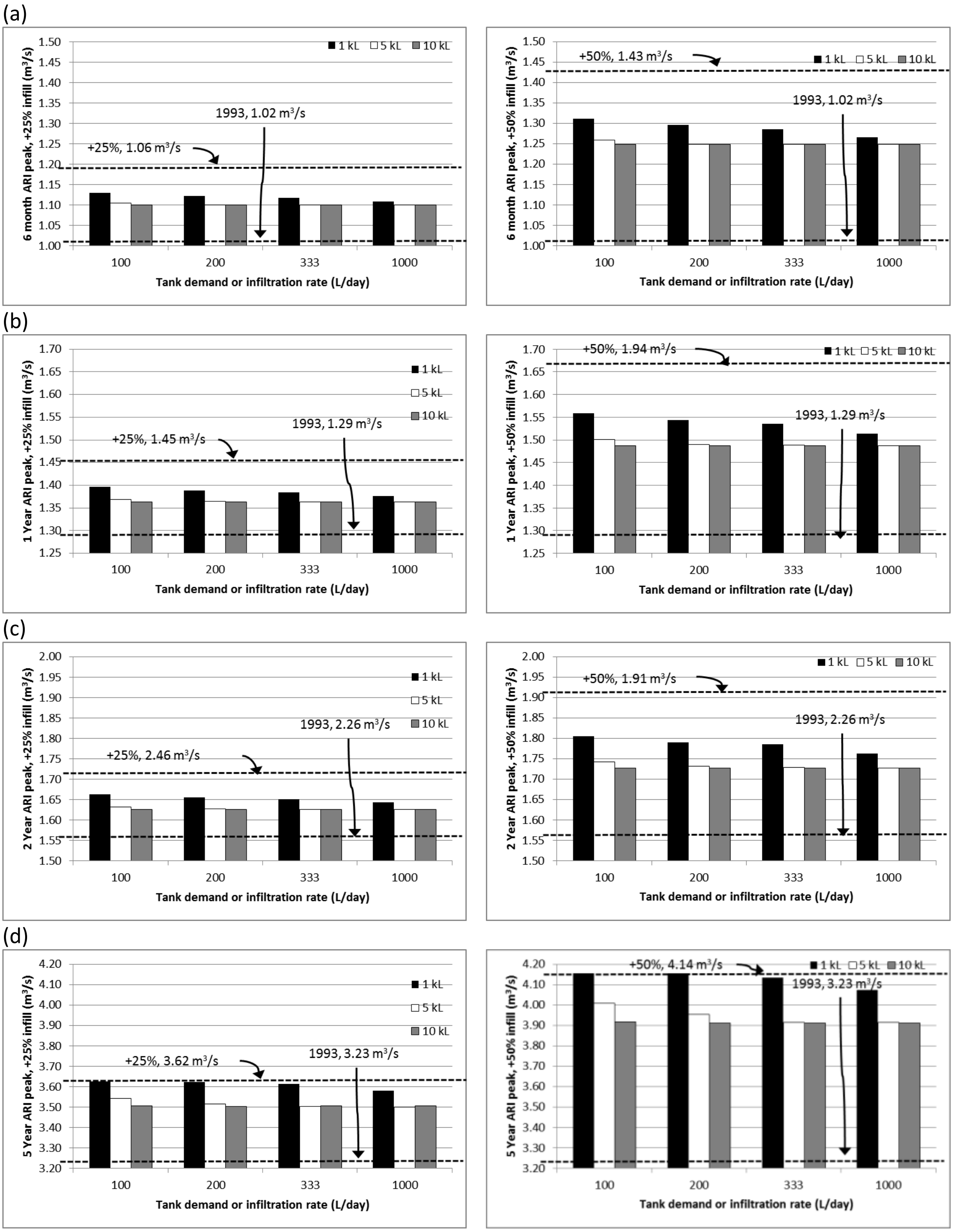


Figure 7-16 – The effect of retention systems on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI of peak flows from the Paddocks catchment +25% infill and +50% infill scenarios

The impacts of retention on the 6 month, 1, 2 and 5 year ARI when applied to new housing in the +25% infill scenario in Figure 7-16 was not able to restore pre-infill development peak flow rates. Similar results were found for the +50% infill case where the peak flow reductions were even less effective. The reduction in the peak flows was lower when the frequency of peak flows reduced. For example, for any given scenario, the percentage reduction of 0.5 year ARI peak flows was higher than the percentage reduction of 5 year ARI peak flows.

Management of Flood Volume

Flood management was assessed at the single case study location using the procedures in Section 7.3.5. A 150 mm flood depth (flooding of 9.4 m^3) was assumed to be critical because it would restrict traffic flow. In the 1993 scenario, this flood depth was exceeded with an ARI of 0.93 years. Figure 7-17 shows the impact using 1 kL, 5 kL and 10 kL retention tanks with varying levels of demand or infiltration to restore the pre-infill development flood frequency. The results shown indicate that some improvement in the flood volume was made by adopting the retention systems. However, there was little benefit progressing from a 5 kL retention tank to a 10 kL retention tank regardless of the level of reuse. The optimum tank size for this catchment would appear to be between 1 kL and 5 kL to reduce flood volumes at this ARI.

(a) (b)

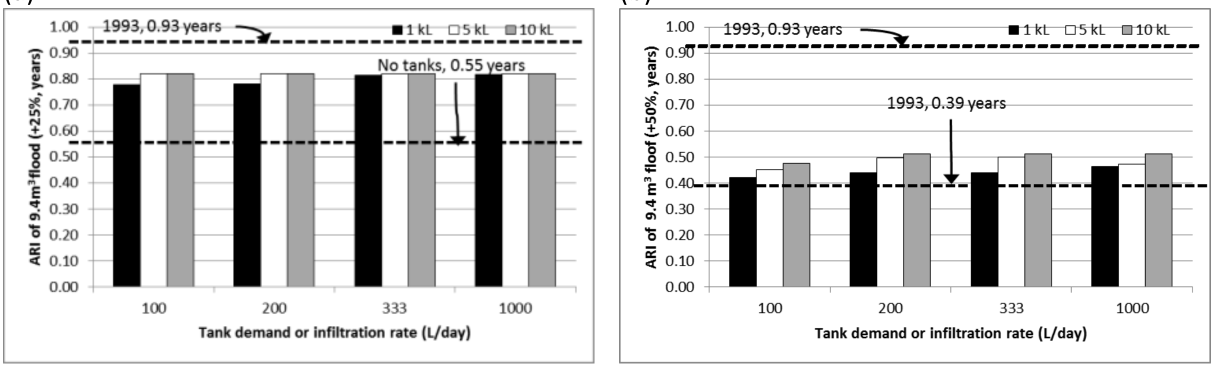


Figure 7-17 – The ARI of critical flood depth at the flood assessment point when retention tanks were applied to the Paddocks catchment (a) +25% infill and (b) +50% infill scenarios

7.4.3 Effect of Detention Tanks on Stormwater Runoff when fitted to New Homes

Management of Stormwater Runoff Volume Using Detention Tanks

Detention systems did not have an impact on runoff volume from the Paddocks catchment for the reasons outlined in Section 6.4.4.

Management of Peak Flow Rates Using Detention Tanks

The peak flow rates from the catchment in 1993, for the +25% infill scenario and the +50% infill scenario were presented in Table 7-11. The effect of detention systems on these peak flow rates from the catchment are presented in Figure 7-18 for the +25% and +50% infill scenarios. These diagrams also compare the effect of detention tank size (1 kL, 5 kL or 10 kL) and demand (from 100 L/day to 1000 L/day). These results assume two detention tanks at each redeveloped site, where each tank has a connected roof area of 100 m^2.

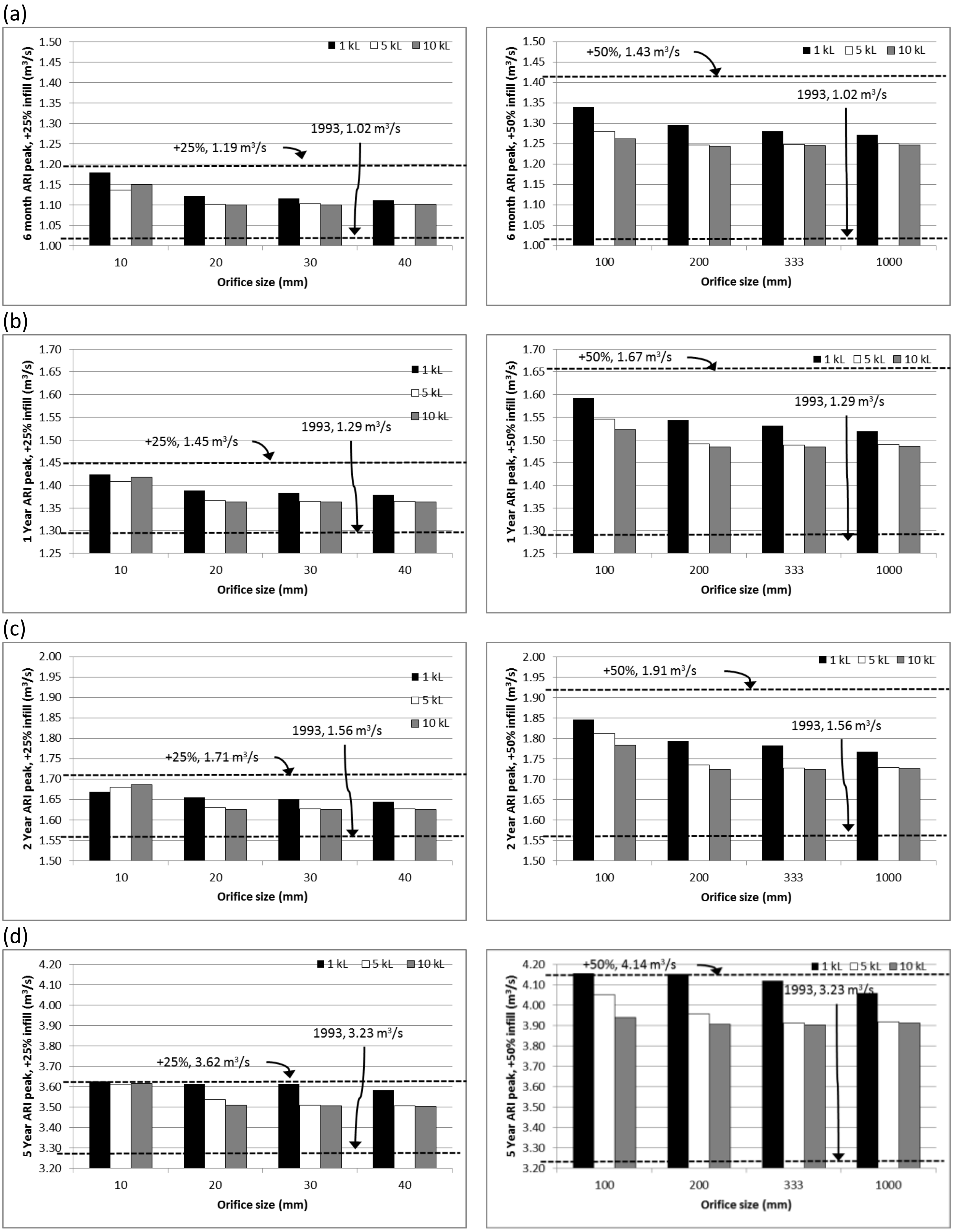


Figure 7-18 – The effect of 1 kL, 5 kL and 10 kL detention tanks on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI peak flows from the Paddocks Catchment +25% and +50% infill scenarios

The results for detention tanks applied to new homes in the +25% and +50% infill scenarios indicate that none of the proposed detention tank sizes were able to restore the 0.5, 1, 2 and 5 year ARI peak flow rates, however larger capacity tanks were most effective. In general, an orifice size between 20 mm to 40 mm performed best in the Paddocks catchment. Similar to results for retention and detention in the Frederick Street catchment, the reduction in the peak flows was lower when the frequency of peak flows reduced. For any given scenario, the percentage reduction of 0.5 year ARI peak flows was higher than the percentage reduction of 5 year ARI peak flows. It should also be noted that the 10 mm orifice size assumption produced higher levels of routing error in the simulation, as was discussed previously in Section 6.4.4.

Management of Flood Volume Using Detention Tanks

The assessment of flood volume was undertaken using the procedures in Section 7.3.5. As stated previously, the average recurrence interval of the 150 mm flood depth was 0.93 in the 1993 scenario, which was reduced as flooding became more frequent following infill development. Figure 7-19 shows the impact of using 1 kL, 5 kL and 10 kL detention tanks with varying orifice sizes to increase the ARI (or reduce the frequency) of flooding. The results indicate that improvement on the flood volume was evident by applying detention systems to new allotments. Larger tanks tended to reduce flood volumes at the case study location more effectively, but there was little improvement by adopting a tank greater than 1 kL. Like the end of catchment flow management results, orifice sizes tended to be most effective in the range of 20 mm to 40 mm.

(a) (b)

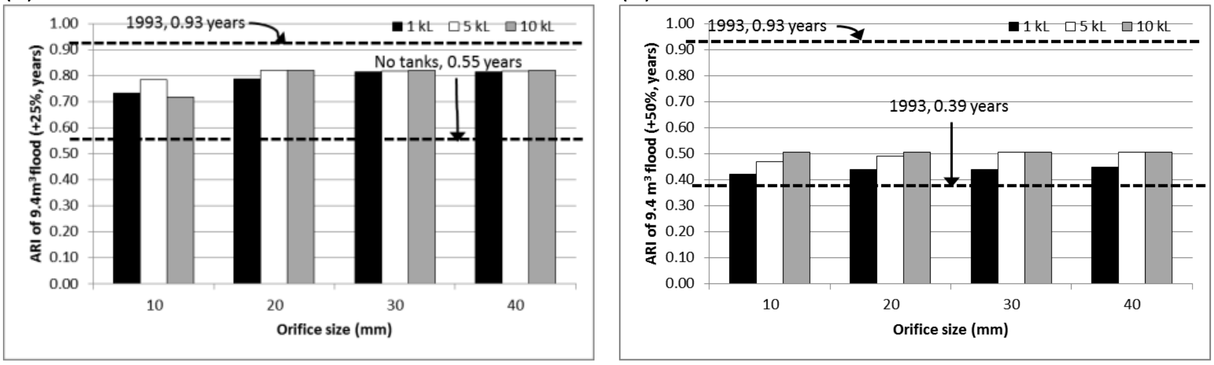


Figure 7-19 – The ARI of critical flood depth at the Paddocks catchment flood assessment point for the (a) +25% infill and (b) +50% infill scenario

7.4.4 Comparison of Retention and Detention – Peak flow and Duration

Using the data above, it was possible to examine the effectiveness of detention tanks and retention tanks for peak flow reductions in the Paddocks when applied in equal numbers across the catchment. A comparison for the 6 month, 1 year and 2 year peak flow rates are presented in Figure 7-20. It compares the impact of 1 kL retention tanks with a 100 L/day or 200 L/day demand, and a 5 kL detention tank with a 20 mm orifice. The results were similar for the 5 kL case. Results were also similar for the 5 year ARI, but the data was not shown because the y-axis scale detracted from interpreting the results at the lower ARI values.

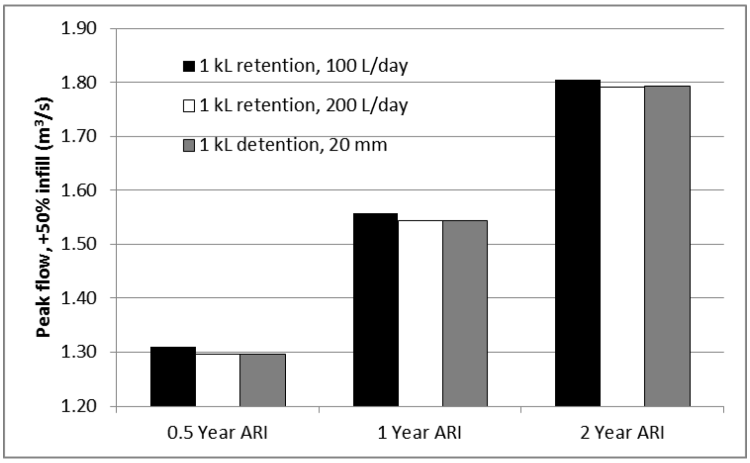


Figure 7-20 – Comparison of the peak flow resulting from 1 kL retention (100 L/day & 200 L/day demand) and 1 kL detention (20 mm orifice) on new homes in the +50% infill scenario

The results indicate that there was little difference between the peak flow rates resulting from identical scenarios with comparable detention and retention when tanks were applied to new homes in the Paddocks catchment. When results for the flood volumes of these scenarios were plotted, there was little difference between the flood volumes. The results concur with those previously presented in Section 6.4.6.

7.4.5 Effect of Street Scale Bioretention on Stormwater Runoff

Management of Stormwater Runoff Volume using Street Scale Bioretention

Only those bioretention systems which could not connect to the stormwater drainage system were able to reduce runoff volumes, as it was assumed that these were draining from the base of the system. The impact of these bioretention systems on mean annual runoff are shown in Table 7-12.

Table 7-12 – The impact of street scale bioretention on mean annual runoff in the Paddocks catchment

	Mean annual runoff (ML)		
Scenario	No bioretention	With bioretention	% reduction
Current	60.6	-	-
+25% infill	70.9	65.3	7.9
+50% infill	85.9	80.2	6.7

Management of Peak Flow Rate using Street Scale Bioretention

The effectiveness of connected and disconnected street scale bioretention systems for the management of peak flow rates across the Paddocks catchment are shown in Figure 7-21.

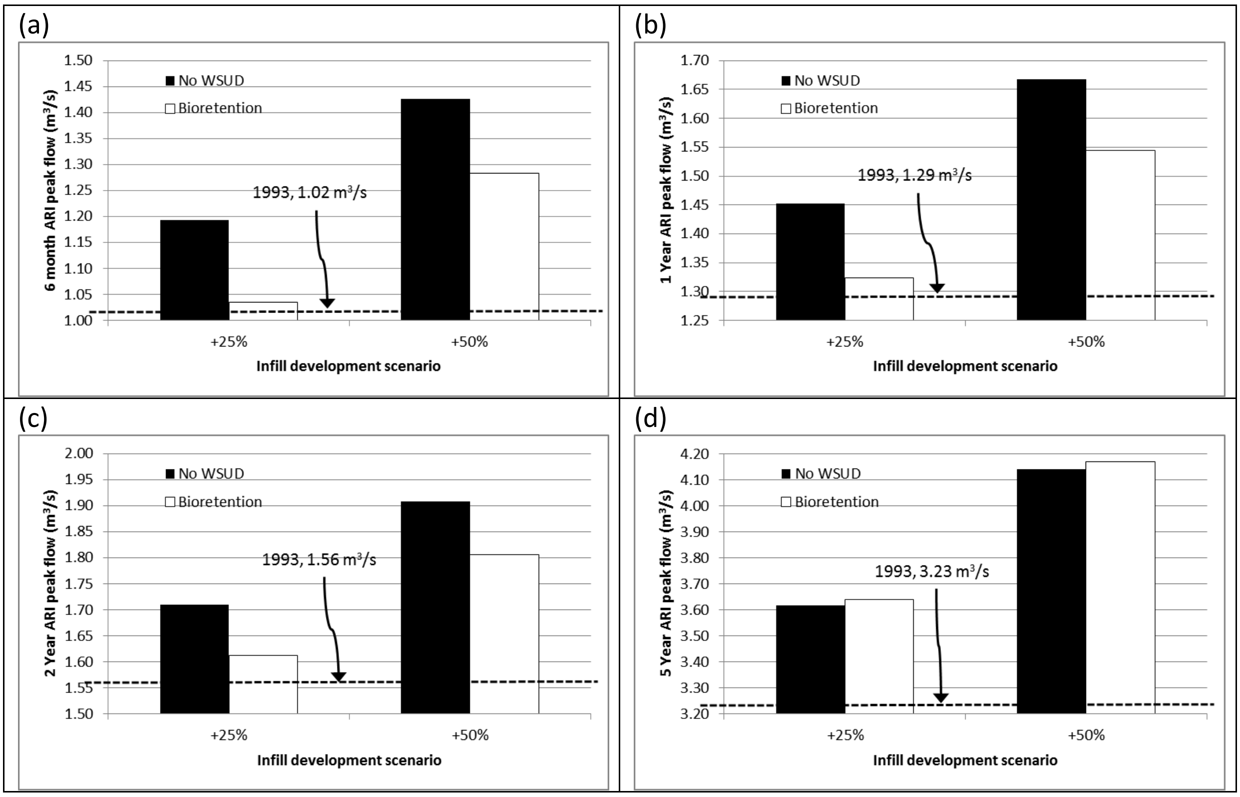


Figure 7-21 – The impact of street scale bioretention on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI peak flows from the Paddocks catchment in the +25% and +50% infill scenarios

Like the case of Frederick Street, the effectiveness of street scale bioretention may be attributed to the detention of flows during the ponding and filtration process. Effectiveness is also considered to be improved by the large connected impervious area that is possible at the street scale. Retention and detention tank scenarios have been restricted to a 100 m^2 roof connection on each new home, however it was reasonable to assume that all impervious surfaces drain to street scale bioretention as runoff from contributing catchment area roofs, driveways and roads will drain to the street scale bioretention when installed at 100 m intervals across the Paddocks catchment.

Management of Flood Volume using Street Scale Bioretention

Figure 6-30 shows the 2.6 year ARI of flows achievable using street scale bioretention in the Paddocks catchment. The results indicate that some improvement of the flood volume was evident from the application of bioretention. The original flood volume in 1993 was almost restored when applied to the 25% infill scenario, but the systems were less as infill development progressed.

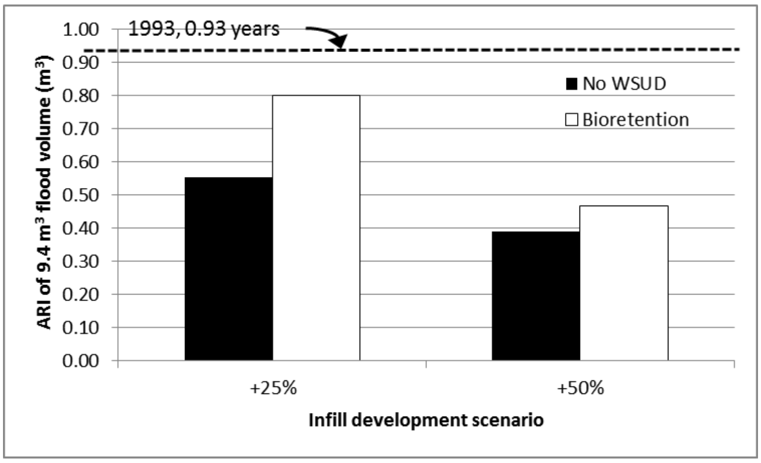


Figure 7-22 – The ARI of critical flood depth at the Paddocks catchment flood assessment point with and without street scale bioretention for the +25% and +50% infill development scenarios

7.5 Discussion

The results indicated that as infill development progressed across the catchment, the runoff volume, peak flow and flood frequency increased. The runoff volume from the catchment was 60.6 ML/annum in 1993, increasing by 17% and 34% for the 25% and 50% infill scenarios. For peak flow rates, the pre-infill development 5 year ARI increased by 10.5% and 21.9% for the 25% and 50% infill scenarios, respectively. In percentage terms, the increase was higher for more frequent events. Flooding in excess of the determined threshold also occurred more frequently, with the ARI of design flood conditions reducing from 0.93 in the pre-infill development case to 0.55 and 0.39 in the 25% and 50% infill development scenarios.

Retention tanks had some beneficial impact on the runoff volume, peak flow and flooding. The mean annual runoff volume was reduced by 5.4% to 7.4% using rainwater tanks of 1 kL to 5 kL volume, respectively, with typical levels of usage (100 L/day) in the 25% infill scenario. The volume of runoff in the 50% infill scenario could be reduced by 9.2% to 12.7% using these tanks. However, a reduction of more than 14% and 29%, respectively, was required to restore pre-infill runoff volumes. The peak flow rates from the catchment were improved by retention systems, but the original peak flows were not restored to pre-infill development levels. The frequency of flooding was improved by the 1 kL to 10 kL retention tanks, but the retention was less effective in the 50% infill scenario compared to the 25% infill scenario. For example, the ARI of flooding in the 25% infill scenario increased from 0.55 years without 5 kL tanks to 0.85 years with the tanks installed on new homes and with typical demand (100 L/day), which was close to the pre-infill development 0.93 years. However the pre-infill development flooding conditions were not restored in any retention scenario. It should be noted that these results were restricted to the assumption of a 100 m^2 connected roof area per tank. The results also suggest that under these conditions, there was little impact of increasing the tank size from 5 kL to 10 kL to reduce the runoff volume, peak flow or flood frequency. There was however benefits by increasing the tank size from 1 kL to 5 kL. The assumed 100 m^2 connected impervious area may influence this outcome, however.

Detention tanks showed an ability to reduce the peak flow rate and flood volume of stormwater runoff when applied to new housing. The application of tanks in the size range of 1 kL to 5 kL would appear to be the most effective, with an orifice of 20 mm to 40 mm. The adoption of 5 kL detention tanks on new homes with a 30 mm orifice reduced the 1 year ARI peak flow rates of the 50% infill scenario by 4.0% compared to the 19.3% reduction required to restore the pre-infill peak flow rate. The frequency of flooding was also improved by detention tanks in the Paddocks catchment. For example, a 5 kL detention tank with a 30 mm orifice on new homes in the 25% infill scenario increased the ARI of the selected flood condition from 0.55 years to 0.82, which was equivalent to a rainwater tank of the same size with modest usage. Similar to the retention tank scenario, there was little improvement in adopting tank sizes greater than 5 kL, although this assumption was likely contingent on the connected roof area. Detention tanks were not able, nor intended to reduce stormwater runoff volumes.

The results indicate that street scale bioretention was effective at reducing peak flow rates and flood volumes when applied across the catchment on each side of the road at 100 m intervals. However, the improvement was mainly observed in the 25% infill scenario, and improvement was also not strong in the 50% infill scenario, indicating that the capacity of the systems to restore flows reached a limit in the when the extent of infill was approximately 25%. This effectiveness may be attributed to the connected impervious area to each system, which was much higher than that of the domestic scale retention and detention systems restricted to a partly connected allotment roof.

A key finding from this analysis was that the results for the Paddocks match well with the results for the Frederick Street study, indicating that the relationships for the effectiveness of WSUD seem consistent regardless of slope (approximately 0.5% in Frederick Street, and approximately 5% in the Paddocks) however the catchments were similarly sized (45 Ha for Fredrick Street, 76 Ha for the Paddocks catchment). Based on this, the recommendations previously stated in Section 6.5 (for the Frederick Street catchment results) are also applicable to the Paddocks catchment results. It is suggested that research is undertaken on a much larger catchment (greater than 100 ha) to ensure that scale does not influence the finding, and since both studies were based on the same rainfall time series, there is more research recommended for catchments with differing climate conditions. It also remains important to explore opportunities to maximise the amount of impervious area connected to retention and detention in redeveloped allotments of a catchment.

7.6 Summary

The impact of infill development in an existing urban catchment was assessed using a high gradient fully urbanised case study catchment in Para Hills, SA. The mean annual runoff, peak flow rate and flood frequency of the 76 Ha catchment was determined based on simulating 19 years of flows resulting from a calibrated model of the catchment in 1993. The change in these values was quantified based on increases in the catchment imperviousness due to the occurrence of theoretical scenarios representing 25% infill and 50% infill development. Results indicated that infill development increased the mean annual runoff, increased the peak flow rate at the end of the catchment and reduced the flood capacity of the catchment. Retention and detention on the new homes of subdivided allotments, with typical impervious area connections (100 m^2 roof), contributed to but could not fully restore the pre-infill development flow regime of a catchment. Higher levels of connected impervious area, achieved by implementing tanks to all existing and new homes, did restore the flow regime to pre-infill development levels. There was little difference between the peak flow and flood reduction benefits achieved by on-site retention and detention storages. Retention systems may be considered to provide additional benefits based on their ability to reduce flow volume. Street scale rain gardens, which may be assumed connected to all upstream impervious area, were effective at restoring the flow regime up to a limited extent of infill development, but their effectiveness was restricted by storage capacity.

8 Case Study 6 – Flagstaff Pines Catchment

8.1 Introduction

Flagstaff Pines is a recent development which has been undertaken immediately west of the intersection of Flagstaff Road and Black Road, Flagstaff Hill. The development is wholly located in the City of Onkaparinga. The Flagstaff Pines catchment being considered in this report is approximately 16 Ha, and represents the portion of the development that drains directly to Flagstaff Creek before reaching a detention basin (Stages 1 to 4, and part of Stage 5). The location of the Flagstaff Pines catchment is shown in Figure 8-1, indicating nearby major roads. It should be noted that the aerial photo is from 2007 prior to construction of the residential development. It also shows the total catchment area of the detention basin, approximately 16 Ha. Unless otherwise indicated, the main area of interest in this study is the flow occurring at Flow Point 1, indicated in Figure 8-1, which is the flow through Flagstaff Creek upstream of the detention basin.

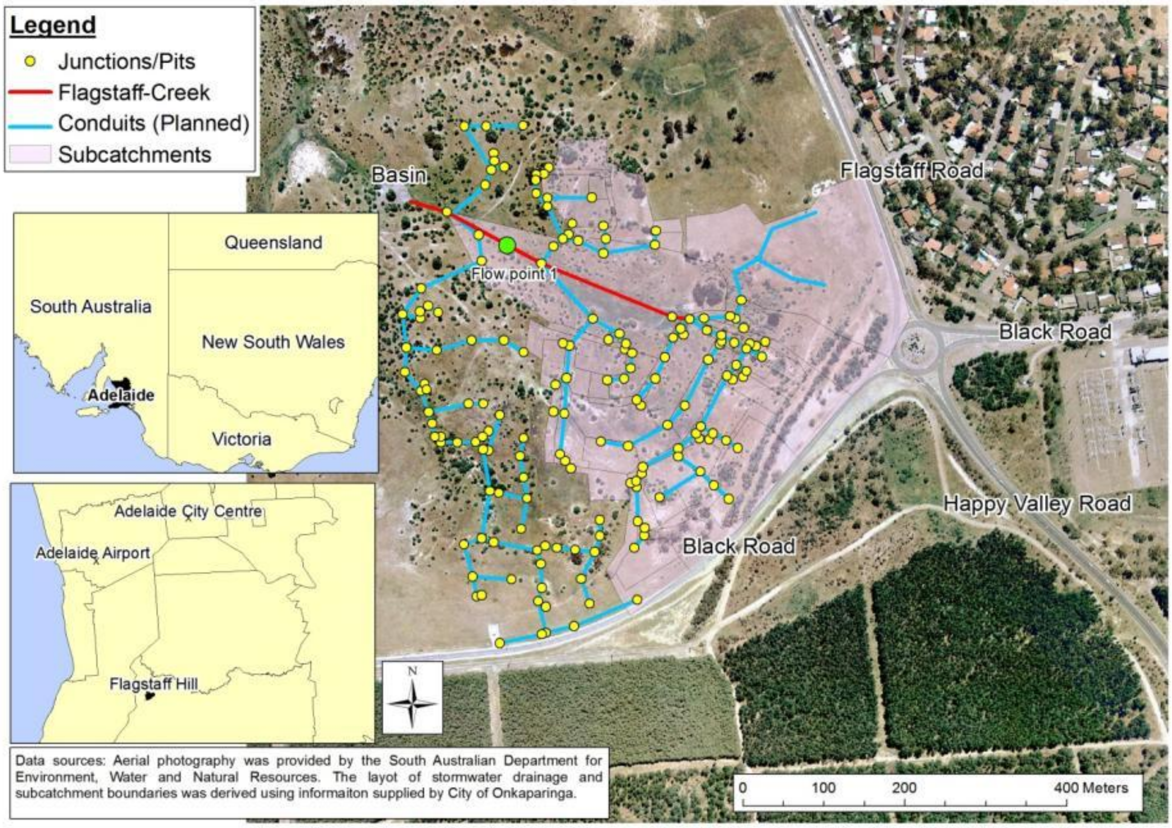


Figure 8-1 – Location of the Flagstaff Hill catchment, indicating selected roads

There is known to have been expenditure on flow management measures by local government both upstream and downstream of the detention basin as flows from the catchment area scour the existing drainage line. Despite this expenditure, examples of creek bed degradation was still evident in 2013 with scouring occurring just upstream of the detention basin in Figure 8-2 (image taken just upstream of Flow point 1 in Figure 8-1). This was considered an important impact because scouring of the creek bed disturbs natural habitat and enables transport of sediment downstream (Walsh et al., 2005a). Flow from the Flagstaff Pines catchment eventually spills into the Sturt River and to Adelaide's coast, which has subsequent effects on Adelaide's coastal environment (McDowell and Pfennig, 2013). However there may be opportunities to use WSUD techniques to reduce the overall effective impervious area of a developed catchment and potentially restore the natural flow regime of urban streams (Walsh et al., 2005b). This may involve the use of retention or detention based mechanisms which intercept a portion of runoff in the catchment to reduce the volume of flow, as well as the magnitude and frequency of peak flows.

(a) (b)

Figure 8-2 – Examples of (a) creek scouring and (b) measures taken by local government to overcome creek scouring via installation of rock gabions to reduce flow velocity

8.2 Aims

There were two aims to the greenfield development case study in Flagstaff Hill:

- To estimate the impact of development frequent flows in the stream at Flow point 1 draining the catchment to a detention basin
- To explore the opportunities to overcome these impacts with WSUD features

8.3 Methods

To examine the impact of development on peak flow and runoff volumes, and potential WSUD solutions to ameliorate this impact, steps in Section 3.4 were followed. Site selection (Step 1) involved a review of greenfield sites in Greater Adelaide, with the selection of Flagstaff Hill to represent a greenfield development site where creek scour was noted and for which information was available. Further information on the site selection and characteristics is provided in Section 8.3.1. The computer software for the analysis (Step 2) was PCSWMM, selected in accordance with Section 3.4.2. Pre development and post-development models of the catchment were assembled based on the data and assumptions outlined in Section 8.3.2. There was no data available for calibration or verification of the model. The scenarios used to assess the pre-development flow rates, post development flow rates and the potential for WSUD to overcome these changes (Step 5) are described in Section 8.3.3. The long term continuous rainfall data applied in hydrological modelling (Step 6) was 19 years of data from Parafield Airport reservoir, for the reasons described in

8.3.2. The characteristics of runoff volume and the magnitude and frequency of peak flows (Step 7) were examined in accordance with the procedures in Section 3.4.7. The results of the analysis (Step 8) are presented and compared in Section 8.4.

8.3.1 Site Selection

The Flagstaff Pines catchment was selected in consultation with local government representatives. The site was selected for study because it represents a greenfield development drained by a natural stream which has been impacted by development. It was also relatively recent and information on the drainage system was available from the local authorities (City of Onkaparinga).

8.3.2 Model Assembly

Previous Modelling

Previous modelling of the Flagstaff Hill catchment was DRAINS modelling conducted for the purposes of stormwater drainage design. The design was undertaken by Maunsell Australia (2004) and Maunsell-AECOM (2005). Design was undertaken in a staged manner, with each design model constructed for a 'stage' of construction and sale. There was no modelling known to have occurred on the entire 'as-constructed' catchment. Input data was based on the drainage design requirements of the City of Onkaparinga. Undertaken prior to development, the modelling assumed that 40 to 50% of each allotment was directly connected impervious area, 10% of the allotment was indirectly connected (or supplementary) impervious area and 40 to 50% grass area. Road catchment areas were assumed to be 60% directly connected impervious area and 40% pervious area. Reserves were assumed to be 100% pervious. Other modelling data assumed by Maunsell-AECOM (2005) is presented in Table 8-1.

Table 8-1 – Key properties of the Flagstaff Hill catchment model developed by Maunsell-AECOM (2005)

Model parameter	**Value**
Impervious area depression storage (loss)	1 mm
Pervious area depression storage (loss)	5 mm
Impervious area roughness, N_{imp}	None (catchment time lag only)
Pervious area roughness, N_{perv}	None (catchment time lag only)
Catchment slope	6 % (average, 188 m to 154m)
Soil infiltration rate – Initial	162.5 mm/h
Soil infiltration rate – Final	9.7 mm/h
Shape factor	2 h^{-1}

Catchment characteristics

The Flagstaff Pines catchment area has been recently urbanised with residential housing and is situated on a hill face. The mean slope in the catchment is approximately 5%. Prior to development, the catchment area drained into two small creeks. The contributing catchment area of Flagstaff Creek, upstream of the detention basin, has remained relatively similar following development - 16.5 Ha prior to development, and 16.0 Ha following development. Flow from the detention basin at the end of the development, with a total catchment area of 23.7 Ha, is released via a fixed outlet and continues downstream, ultimately entering Sturt River.

The entire Flagstaff Pines development consists of a total of 260 allotments constructed in nine separate stages. However this simulation considered only the southern portion of the development which drained into Flagstaff Creek which was Stages 1 to 4 and parts of Stage 5. This included 196 allotments, plus access roads. Of this area, 121 allotments drain into Flagstaff Creek prior to reaching the basin. Allotment sizes varied in this area, from 450 to 1000 m^2, with most allotments approximately 700 m^2 in plan area. The soil in the catchment consists of loam and calcareous loam over clay on rock according to the *Adelaide Metropolitan Soils Data CD* available from Primary Industries and Resources South Australia (PIRSA).

Model Assembly

The layouts of the pre-development and post-development models were adopted based on the modelling data from Maunsell AECOM (2005), provided by the City of Onkaparinga. In the absence of any flow measurement, the modelling parameters from Maunsell-AECOM (2005) were first assumed to be a reasonable estimation of the on-site conditions at Flagstaff Pines. Some adjustments were required during model assembly. In the DRAINS design mode used by Maunsell-AECOM (2005), catchment lag (measured in minutes) was used to simulate the time over which flows travel over a catchment surface before entering a drainage pit. This was reflected in this study (using SWMM) by adjusting the width parameter, which is indirectly related to the lag time. A full description of catchment width is provided by Rossman (2010). Briefly however, for the Flagstaff Hill catchment, width was estimated based on the catchment area and the length of overland flow. In most circumstances, length of overland flow was assumed to be 25 m (based on the approximate distance of travel of runoff from housing lots to gutters). However, for reserves and other open spaces, the maximum length of overland flow was measured as the maximum length of flow in the catchment to its outlet.

Initial modelling results also indicated that the pervious area properties described by Maunsell-AECOM (2005) were generating few runoff events over the 22 year timeframe in the pre-development scenario. As such, pervious area properties were reviewed based on the soil conditions outlined above. The parameters for the Horton infiltration model were those in Table 8-2, where data was adopted from the recommendations of Rossman (2010) for clay-loam soils. In addition to these parameters, SWMM also required a roughness value for the catchment surface. This was based on the recommendations of Rossman (2010), with 0.012 chosen for impervious surfaces and 0.15 for pervious surfaces (low vegetation density, cleared land).

Table 8-2 – Horton Infiltration parameters for the pre-development and post-development model of Flagstaff Pines

Horton model parameter	**Value**
Soil infiltration rate – Initial	25 mm/h
Soil infiltration rate – Final	2 mm/h
Decay constant (Shape factor)	5 h^{-1}
Drying time	5 Days
Maximum volume	Disabled (no maximum)

Climate and Flow Data

There was no rainfall measurement available inside the catchment boundary. The nearest rainfall gauge to the catchment was a DEWNR operated pluviometer located at the Happy Valley reservoir (A5030532). This gauge has operated from 01/10/1988 to the present and is located approximately 800 m from the catchment boundary. This gauge was not used for the rainfall runoff modelling in PCSWMM because the data had gaps many instances of accumulated data between 1992 and 2013. However, the gauge indicates that the average annual rainfall between 1992 and 2013 was 612 mm. In light of the data problems, and to enable a comparison with previous sections, this catchment was simulated with the 19 years of rainfall data from Parafield Airport (Section 3.4.6).

The nearest climate station to Flagstaff Pines is the Bureau of Meteorology station at Adelaide Airport, more than 10 km away. As such, average monthly evaporation data has been adopted from this gauge. This information was previously presented in Section 6.3.2 (Figure 6-5).

There was no flow data available from anywhere in the vicinity of the catchment, however a volumetric estimation was used to determine the suitability of pre-development runoff volumes in Section 8.3.3.

8.3.3 Model Calibration

There was no data available for the calibration of the pre- or post-development case of the Flagstaff Hill catchment. There were a number of assumptions made regarding the nature of flow through the catchment both pre- and post-development. It is however possible to compare the predicted annual runoff data for the Flagstaff Hill catchment with rainfall and runoff characteristics for more than 30 catchments in the Mt lofty Ranges. Tomlinson et al (1993) presented data for these catchments, all of which have been modified for agricultural or urban use. Using the mean annual rainfall of the catchment (612 mm) and comparing it to the values in Figure 8-3, it would appear than the anticipated mean annual runoff volume pre-development would be 40 mm/annum (6.4 ML/annum) up to approximately 100 mm/annum (16 ML/annum). The annual runoff predicted by the PCSWMM model assuming the clay soil properties was 65 mm/annum (10.5 ML/annum), which is within the expected range illustrated on Figure 8-3.

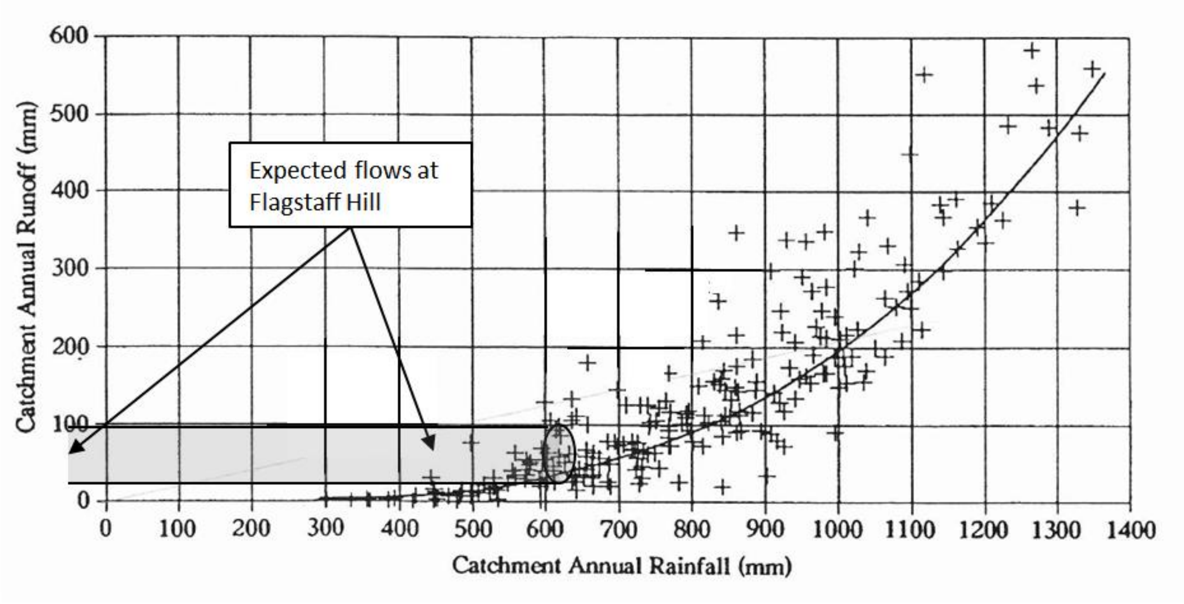


Figure 8-3 – Mean annual rainfall and runoff for 30 catchments in the Mt Lofty Ranges – the mean annual rainfall and potential runoff in Flagstaff Hill is indicated (adapted from Tomlinson et al., 1993)

8.3.4 Modelling Scenarios

There were multiple scenarios used to explore runoff characteristics of the Flagstaff Pines catchment. These included:

- The pre-development scenario
- The post development scenario without WSUD
- Post development scenarios with WSUD alternatives

The pre development site was based on simulating the site as 16 Ha of open space draining to the Flagstaff creek channel. The post development scenario was designed to simulate the as-built site corresponding with the original design for Flagstaff Pines, including parts of Stages 1 to 5 of the Flagstaff Pines development. The post development site was then used as the basis of WSUD scenarios outlined below, to explore the potential of WSUD to preserve 1 and 2 year ARI flows and the existing overall flow regime in the natural channel draining the catchment.

The ability of WSUD scenarios involving retention systems and detention systems to preserve pre-development flow characteristics was then explored. The main characteristics explored were the total runoff volume, the 1 and 2 year ARI peak flow rates, and the overall frequency and duration of flows into the Flagstaff Creek channel upstream of the basin. The retention and detention based WSUD treatment scenarios explored for the Flagstaff Pines catchment are summarised in the following sections.

8.3.1 On-site Retention Scenarios

The retention systems explored for the WSUD scenarios of the Flagstaff Pines catchment were identical to those examined for the Frederick Street and Paddocks catchments, namely rainwater tanks and infiltration systems. Each system was simulated in an identical manner, and with the same variables as identified in Section 6.3.8. The scenarios were simulated with variation in the same four key variables: storage volume, water demand or infiltration rate, connected roof area and number of tanks/systems per property. Storage volume assumptions were identical to those described in Section 6.3.8. Tank demand or infiltration assumptions were identical to those described in Section 6.3.8. The connected roof area to each tank was identical to that described in Section 6.3.8.

The number of retention tanks assumed present in the Flagstaff Pines catchment was assumed to be one per allotment produced in the catchment. The post-development catchment contains 121 allotments.

8.3.2 On-site Detention Systems

The detention systems explored for Flagstaff Pines were on site detention tanks, identical to those applied to the Frederick Street and Paddocks catchments and described in Section 6.3.9. The four variables used in the simulation of detention tanks were tank size (volume), tank shape, orifice size and number of detention tanks. All detention tank scenarios assume a connected roof area of 100 m^2. The detention tank volumes simulated were 1 kL, 5 kL and 10 kL, selected and applied as described in Section 6.3.9. The outflow orifice size examined was between 10 mm and 50 mm, in 10 mm increments, selected as described in Section 6.3.9. The number of detention tanks assumed was identical to rainwater tanks, with one tank fitted to each of the 121 allotments.

8.3.3 Street Scale Bioretention

The implementation of street scale bioretention to mitigate flows in the Flagstaff Pines catchment was explored based on the arrangement of bioretention systems in Mile End (Section 5) and similar to the arrangement in Frederick Street and the Paddocks (Section 7.3.10). Like the Paddocks catchment, drainage does not always run adjacent to public roads in Flagstaff Hill, but a reasonable coverage of street scale systems was possible when trying to place systems in open space at 100 m intervals, and only Type 1 systems (systems connected to stormwater drains, Section 7.3.10) were applied. The location of the proposed rain gardens in Flagstaff Pines are shown in Figure 8-5.

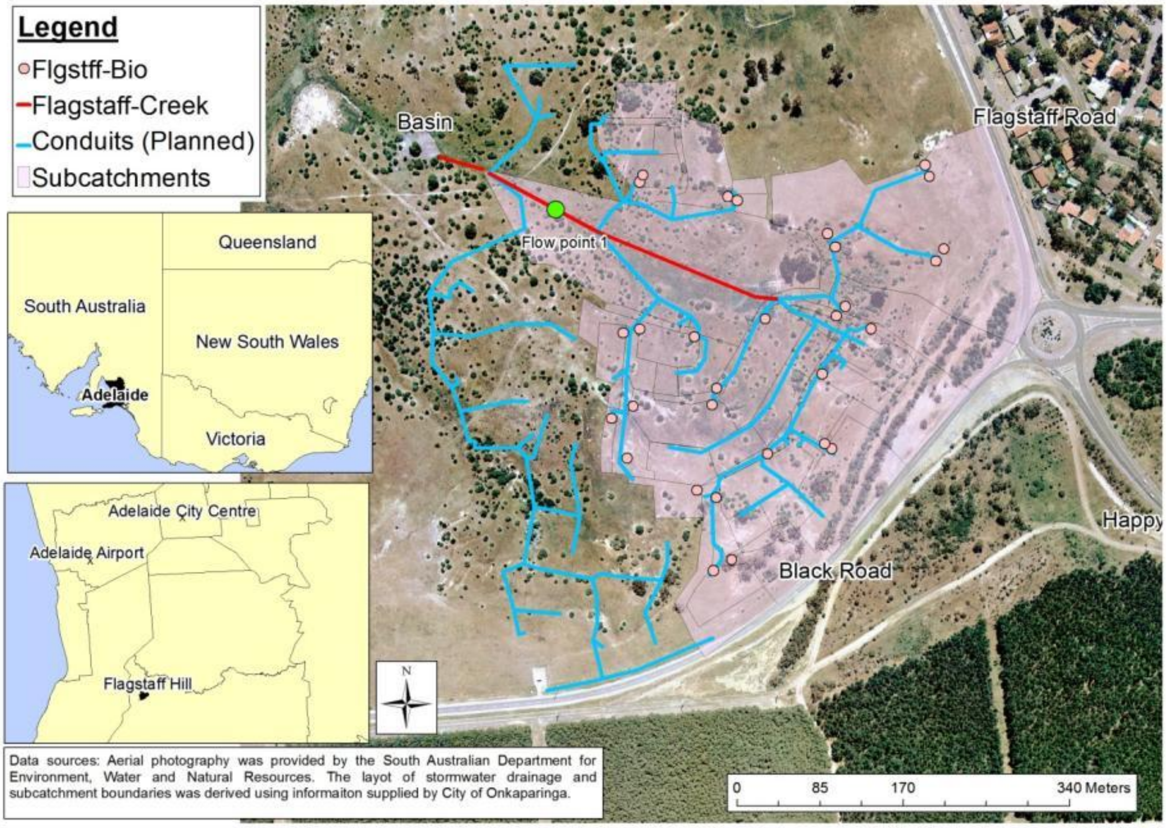


Figure 8-4 – Proposed location of bioretention in the Flagstaff Pines catchment

8.4 Results

8.4.1 Effects of Development on Creek Flows

Runoff Volume and Peak Flow

The effects of development on creek flows are illustrated by comparing pre- and post-development flow data for the catchment. The mean annual flow and the peak flow rates through the Flagstaff Creek are indicated in Table 8-3. The data indicates that the flow volume increases to a value more than 25 times higher than the original flow volume, and that the peak flow rates through Flagstaff Creek also increase.

Table 8-3 – Mean annual flow and peak flow rates estimated for the Flagstaff Hill catchment pre- and post-development

Case	Mean annual runoff (ML)	0.5 Year ARI (m^3/s)	1 Year ARI (m^3/s)	2 Year ARI (m^3/s)	5 Year ARI (m^3/s)
Pre-development	50	0	0.19	0.39	0.64
Post development	1296	0.89	1.15	1.41	1.76

Flow Frequency

Another means of examining the impact of development is to compare the flow duration curve for the catchment. The flow duration curve for pre- and post-development flows is shown in Figure 8-5. The chart indicates that pre-development flow seldom occurred in the 19 year time series, with flow present approximately 1% of the year. Post development, the occurrence of flow was more common, occurring approximately more than 3% of the year.

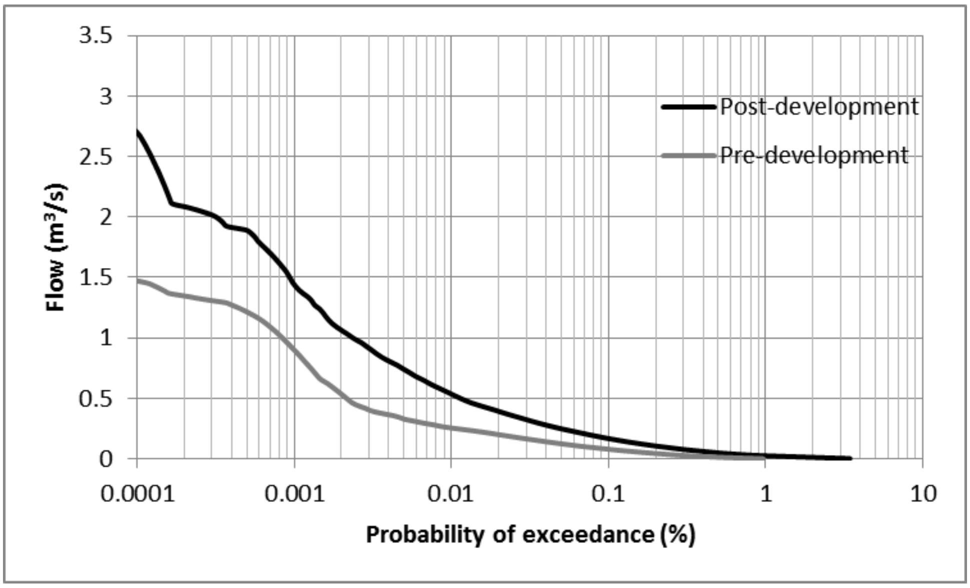


Figure 8-5 – The Pre- and post-development flow duration curve for the Flagstaff Pines catchment prior to the detention basin

8.4.2 Effects of Retention on Post-development Creek Flow

Management of Stormwater Runoff Volume

The impact of retention systems on the mean annual runoff volume are shown in Figure 8-6. The results indicate that the mean annual runoff volume was not restored by implementing retention in the form of rainwater tanks or infiltration systems on each property. Implementing retention tanks of 1 kL, 5 kL and 10 kL reduced post development annual runoff volume by 6.5%, 8.9% and 9.9% respectively with modest reuse rates (100 L/day). With high loss rates (1000 L/day), the mean annual runoff reduction was higher, at 10.9%, 13.2% and 13.8% respectively.

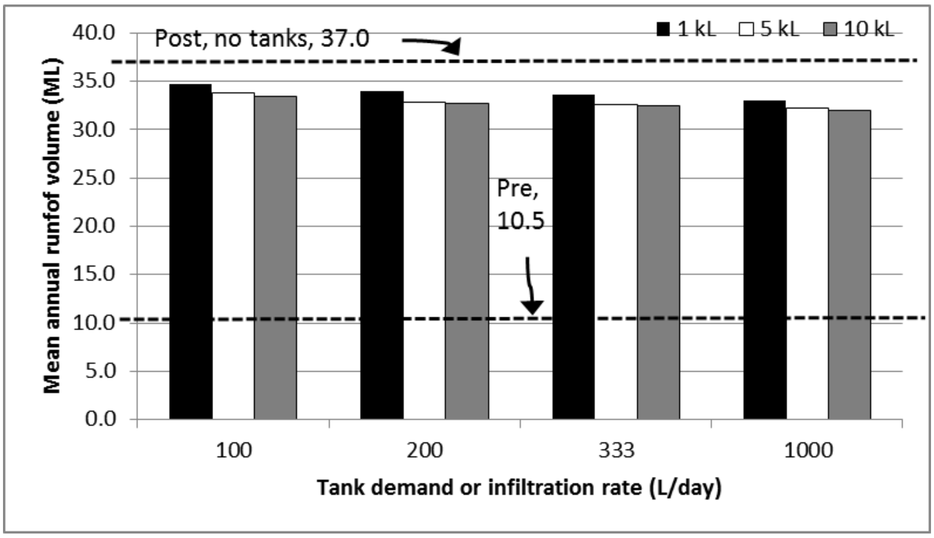


Figure 8-6 – The reduction in the total runoff volume from the Flagstaff Pines catchment when applying 1 kL, 5 kL and 10kL rainwater tanks to each new house post-development

Management of Peak Flow Rates

The peak flow rates from the Flagstaff Hill catchment pre-and post-development were presented in Table 8-3. The effect of retention systems on the post-development peak flows determined via a partial series analysis are shown in Figure 8-7. Each diagram also compares the effect of tank size (1 kL, 5 kL or 10 kL) and tank demand (from 100 L/day to 1000 L/day).

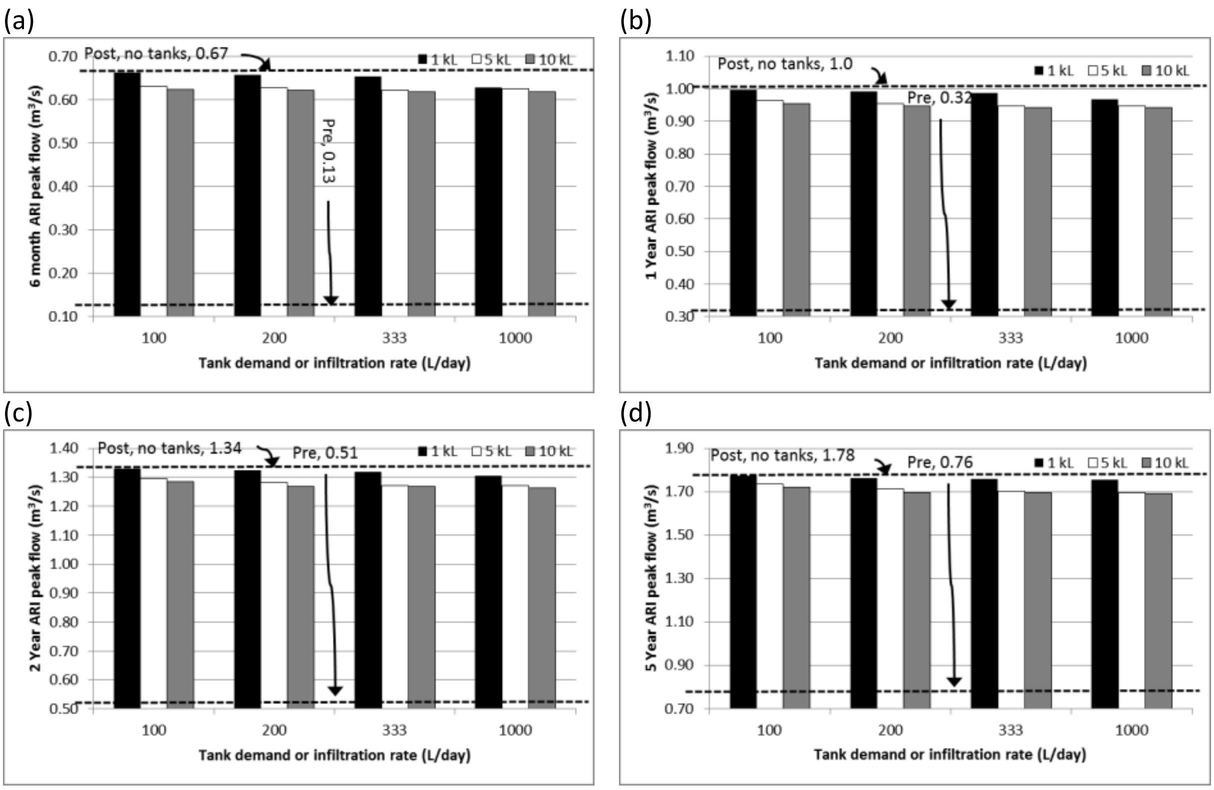


Figure 8-7 – The impact of retention on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI of peak flows from the post development Flagstaff Pines catchment

Management of Flow Duration

The flow duration curve of the catchment pre-development, post development and with the implementation of rainwater tanks of 1 kL, 5 kL and 10 kL and a reuse rate of 100 L/day is shown in Figure 8-8. Similar results were found for other lesser rates of water demand / infiltration. The results indicate that the post-development flow duration curve with retention tanks installed was similar to the post-development catchment with no WSUD in place. Based on these results, retention systems have little impact on the overall flow regime of the catchment.

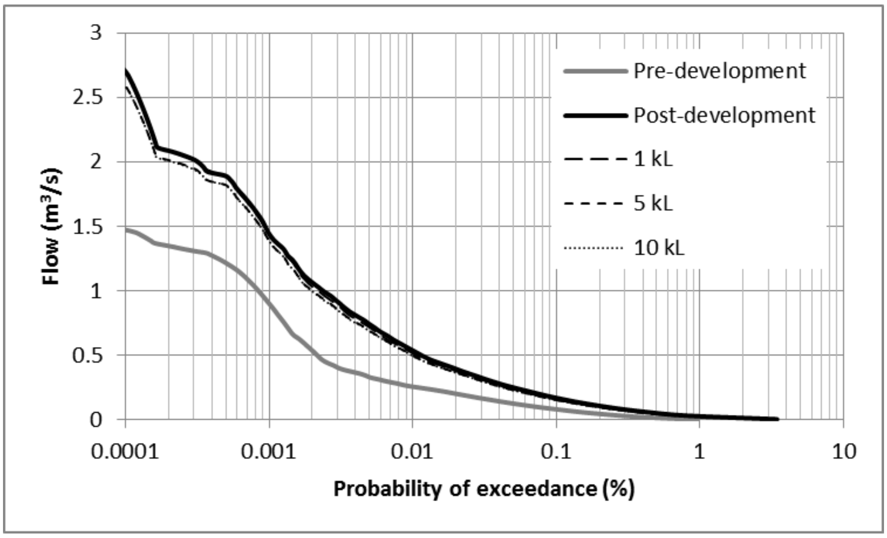


Figure 8-8 – Flow duration curve (or probability exceedance curve) for 1, 5 and 10 kL retention tanks in the Flagstaff Pines catchment (1000 L/day water infiltration)

8.4.3 Effects of Detention on Creek Flows (Post-development)

Management of Peak Flow Rates

The peak flow rates from the Flagstaff Hill catchment pre- and post-development were presented in Table 8-3. The effect of detention systems on the post-development peak flows determined via a partial series analysis are shown in Figure 8-9. Each diagram also compares the effect of tank size (1 kL, 5 kL or 10 kL) and orifice size (from 10 mm to 50 mm). The results indicate that the peak flow rates from the catchment could not be restored from the application of detention tanks when they were fitted to 100 m^2 of roof on each new allotment.

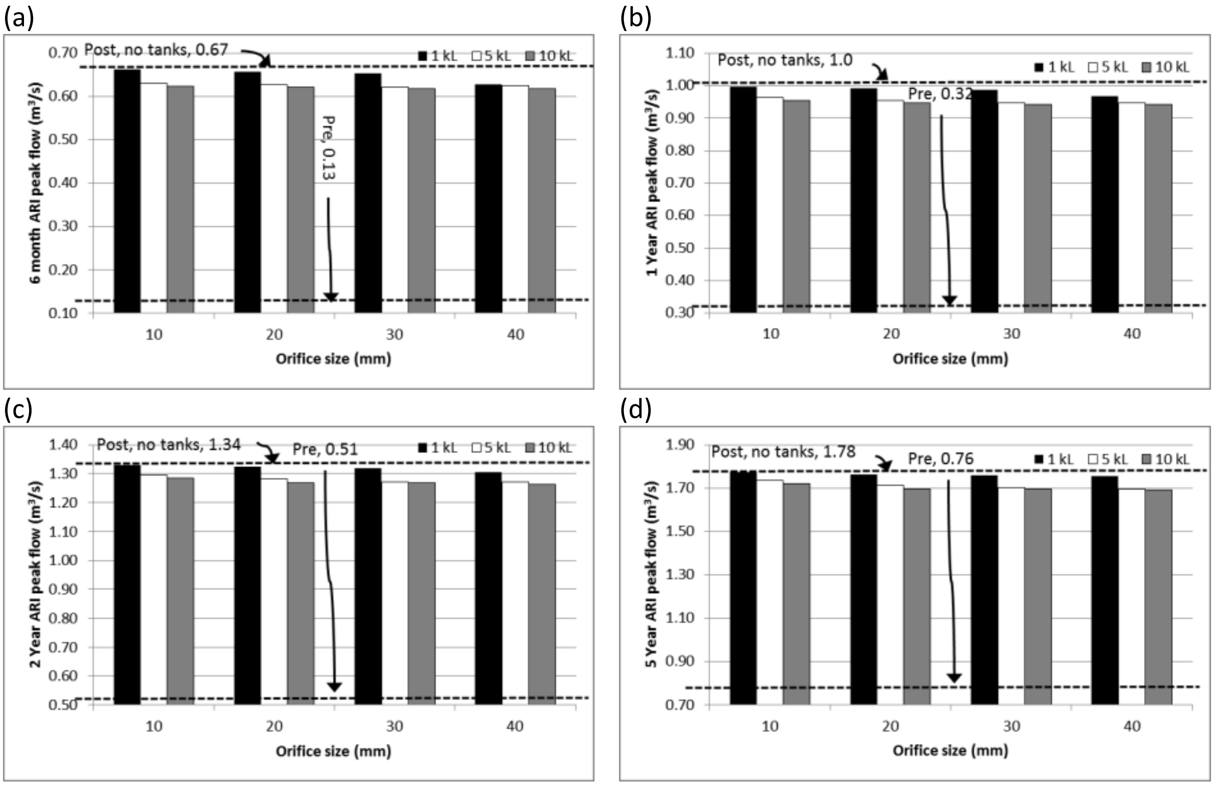


Figure 8-9 – The impact of detention on the (a) 6 month ARI (b) 1 year ARI (c) 2 Year ARI and (d) 5 Year ARI of peak flows from the post development Flagstaff Pines catchment

Management of Flow Duration

The flow duration curve of the catchment pre-development, post development and with the implementation of rainwater tanks of 1 kL, 5 kL and 10 kL and a reuse rate of 100 L/day is shown in Figure 8-10. Similar results were found for other orifice sizes. Like the retention tanks scenarios, the results indicate that the post-development flow duration curve with detention tanks was similar to the post-development catchment with no WSUD in place.

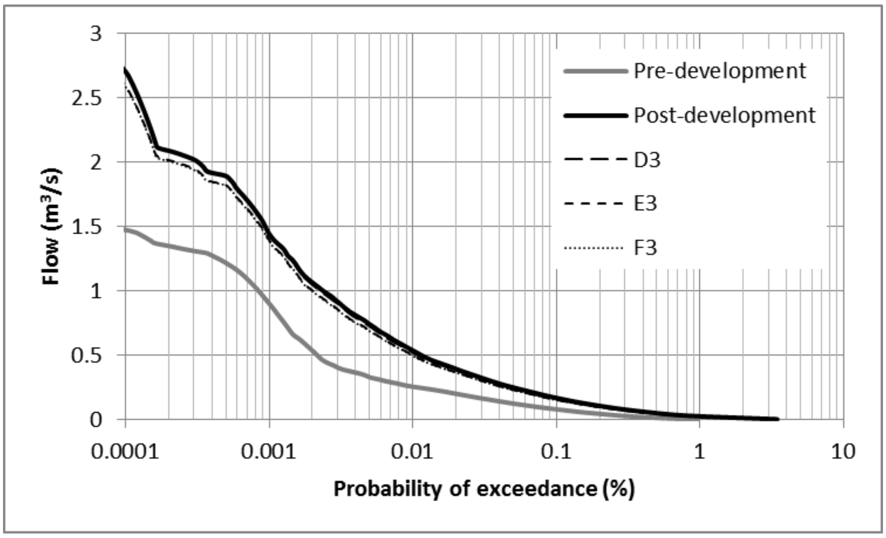


Figure 8-10 – Flow duration curve (or probability exceedance curve) for 1, 5 and 10 kL detention tanks in the Flagstaff Pines catchment (30 mm orifice)

8.4.4 Effect of Street scale Bioretention on Stormwater Runoff

Management of Runoff Volume

The management of runoff volume by street scale bioretention was explored using a scenario where each bioretention system was disconnected from the stormwater network and infiltration was enabled through the base of the system at 3.6 mm/hr. This layout produced a 9.7% reduction in post development rainfall runoff volume from approximately 37 ML/annum to 33.5 ML/annum.

Management of Peak Flow Rate

The effectiveness of street scale bioretention systems for the management of peak flow rates across the Flagstaff Pines catchment are shown in Figure 8-11. It shows the results for the assumption of bioretention which was lined and connected to stormwater drains, or bioretention in the same locations which was not lined and allowed infiltration to occur.

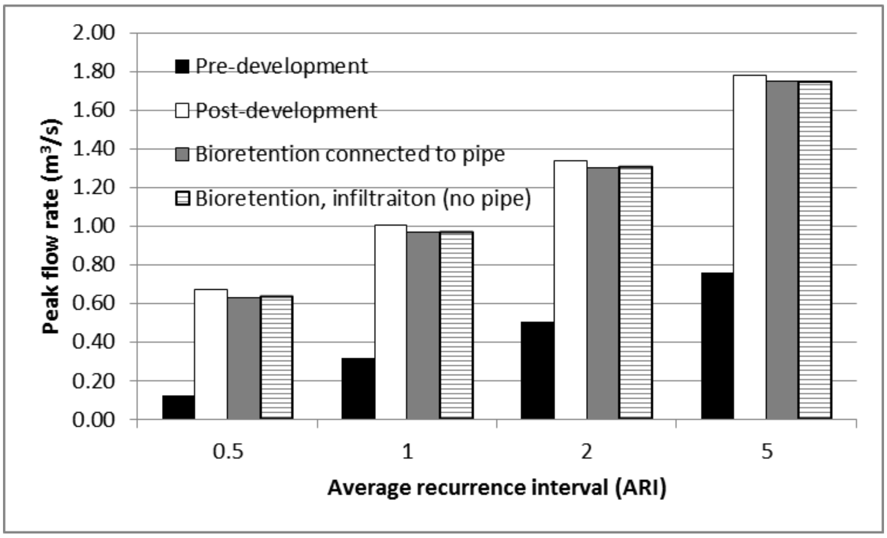


Figure 8-11 - The impact of street scale bioretention on the 6 month ARI, 1 year ARI, 2 Year ARI and 5 Year ARI peak flows from the Flagstaff Pines catchment with bioretention

The bioretention systems were unable to reproduce the pre-development peak flow rates, with very little relative difference between the developed catchment with and without WSUD under the assumed scenario. Furthermore, there was very little difference between bioretention systems connected to street drainage and disconnected from street drainage under the assumed infiltration rate of 3.6 mm/hr (through the base of the system only).

8.5 Discussion

A comparison of the pre-development and post development runoff volume indicated that the volume of flow through the creek would increase from 10.5 ML/annum to 37 ML/annum, a factor of approximately 3.5. Peak flows would also increase; for example the 5 year ARI peak flow increased by a factor of 2.3, from 0.76 m^3/s to 1.78 m^3/s. The flow duration curve also showed a significant change, with flow in the creek present more often and at higher flow rates.

Retention in the form of rainwater tanks or infiltration systems on each property was not able to restore peak development runoff volume, peak flow rate or flow characteristics. Implementing retention tanks of 1 kL, 5 kL and 10 kL reduced post development annual runoff volume by 6.5%, 8.9% and 9.9% respectively with modest reuse rates (100 L/day). With high demand/infiltration rates (1000 L/day), the mean annual runoff reduction was higher, at 10.9%, 13.2% and 13.8% respectively. The 5 year ARI peak flow rate for these tanks sizes was reduced by 1.4%, 4.7% and 5% when a generous demand/infiltration of 1000 L/day was assumed. However, a reduction of 57% is required to restore the pre-development 5 year ARI peak flow. The post development flow duration curve with retention was generally similar to the post development case without any onsite WSUD. Additional benefit was achieved with larger tanks sizes and larger demand/infiltration disposal, however there was little benefit in adopting a tank size greater than 5 kL which may be a function of connected roof area.

Detention systems were not able to restore the pre-development peak flows when they were fitted to 100 m^2 of roof on each new allotment in the Flagstaff Pines catchment. For example, the 2 year ARI peak flow rates were reduced by 2.7%, 5% and 5.6% for 1 kL, 2 kL and 5 kL detention tanks with a 40 mm orifice. The flow duration curve was also not very different in appearance to the flow duration curve for retention tanks for the best performing scenarios. The best performing system was a detention tank of 5 kL with a 30 mm to 40 mm orifice. There was little benefit achieved by adopting larger tanks, but this may be a function of connected impervious area.

Street scale bioretention was also not able to restore the pre-development runoff volume or peak flow rates in the Flagstaff Pines catchment under the assumed layout. There was a 9.7% reduction in runoff volume when systems enabled infiltration at a modest flow rate. Regardless of whether the systems were connected to the stormwater drainage system, peak flow rates were reduced by 2.6% and 1.7% for the 2 year and 5 year ARI, respectively, compared to the 62 and 57% required.

It should be noted that the rainwater tanks, detention tanks and street scale measures were only tested in isolation in the analysis undertaken here. The limited effectiveness in adopting these measures may be improved by a combination of these measures. It should also be noted that larger scale downstream measures, such as distributed or end-of-pipe detention basins, wetlands or larger biofilters were not considered in this analysis for their contribution to flow management. Such technologies may be suitable in a greenfield development. The performance of rainwater tanks should also be considered in light of their contribution to reduction in mains water demand.

8.6 Summary

The impact of greenfield residential development on a previously rural catchment with an ephemeral creek was explored using a case study catchment in Flagstaff Hill, SA. The mean annual runoff, peak flow rate and duration of creek flows from the 16 Ha catchment was determined based on simulating and comparing the effect of 19 years of rainfall over the catchment in a pre- and post-development condition. Development was shown to cause changes to the pre-development flow regime in the creek with increases in the total annual flow and peak flow rates, and alterations in the flow duration curve. The inclusion of retention or detention on all homes constructed was ineffective at maintaining the pre-development flow regime in the creek. Street scale bioretention was also unable to restore pre-development flows. This indicates that the potential for either on-site or street scale WSUD to maintain the pre-development flow regime of a greenfield catchment at this gradient was limited. It should be noted that on site and street scale measures were only tested in isolation, and that the limited effectiveness may be improved by combined measures. Larger scale downstream measures, such as detention basins, wetlands or larger biofilters were not considered in this analysis for their contribution to flow management.

9 Application of Optimisation Tools to select WSUD

9.1 Introduction

In recent years there have been several investigations into the optimisation of WSUD feature design and placement. This has resulted in software tools being developed or adapted for use by catchment managers, however very few of these tools are widely available to the profession. In 2009, the USEPA produced an open source platform called the System for Urban Stormwater Treatment and Analysis IntegratioN Model (SUSTAIN). The software has been cited in several reports for investigating the potential impact of WSUD features on managing flows (Shoemaker et al, 2011; Lee et al. 2012; Shamsi et al, 2014). SUSTAIN includes (Lee et al., 2012):

- a GIS interface
- Suitability analysis tool for WSUD measure[4] siting
- A wide range of stormwater quantity and quality simulation algorithms for both watershed and BMP modelling
- Multiple optimisation techniques to find a least-cost solution and develop cost-effectiveness curve
- A variety of structural WSUD options for evaluation and design optimisation

Since it has been developed as an open source modelling platform by an industry leader in the field of water runoff modelling, SUSTAIN was considered to be the best candidate for a potential modelling platform for optimising urban runoff management using sustainable techniques in South Australia. To investigate how applicable SUSTAIN might be for use in South Australia, the project team explore the application of SUSTAIN to optimise the management of runoff in a small urban catchment. The trial application is detailed in the following sections.

9.2 Aims

The objective of this study was to find the optimum combination of WSUD features (type, number and location) with least cost to reduce the two and five year ARI peak flow rates from a re-developed (higher impervious area) version of a catchment to meet current, pre-infill development peak flow rates in that catchment.

9.3 Methodology

The optimum WSUD framework required to ameliorate the impact of infill development was evaluated using SUSTAIN. Evaluation of peak flows pre- and post-infill development was similar to the eight steps in Section 3.4. Site selection (Step 1) involved selecting a site which was manageable

[4] It should be noted that WSUD measure is used in this text in lieu of references to best management practices, or 'BMPs' in the US based literature regarding SUSTAIN. A BMP in the context of stormwater management practice in the US generally refers to a structural WSUD measure. See Fletcher et al. (2014) for more information.

in the SUSTAIN software. Further information on the site selection and its characteristics are provided in Section 9.3.1. The computer software for the analysis (Step 2) was SUSTAIN, selected as it was the widely applied platform available for the optimisation of WSUD layout and design. A model of the catchment was assembled based on data and assumptions outlined in Section 9.3.2. This includes information on the pre-and post-development scenario. SUSTAIN was then used to produce a 19 year hydrograph of the pre-infill development scenario to determine the target 2 year ARI and 5 year ARI peak flow rates prior to infill development using the procedures in Section 3.4.7 and the rainfall data described in Section 3.4.6. The occurrence of events with peak outflow similar to the 2 year ARI and 5 year ARI were then identified on the outflow hydrograph, and the corresponding rainfall event was identified. These events were then used as characteristic rain events for optimisation to be undertaken. Note that SUSTAIN is only able to restore the peak flow rate (or pollutant load generation) from some maximum value in a time series to the target value. In this case, SUSTAIN was used to optimise a WSUD arrangement that would restore the post-infill development peak flow value to the pre-infill development peak flow rate. If the entire 19 year time series was used, SUSTAIN would attempt to minimise the peak flow of any events larger than the 2 year ARI or 5 year ARI peak flows to the specified flow rate. It would also take a long time to undertake optimisation with a 19 year time series. However, it should be noted that the event selected for optimisation was not a single event; following the identification of the single event which caused outflow similar to the 2 year and 5 year ARI flow, a lead in time of approximately one year months was selected to produce a realistic 'warm up' period for the model run. This period was cross checked for events in excess of the 2 year or 5 year ARI.

Details of the assumed nature of potential WSUD solutions were identical to those in Frederick Street, but are briefly described in Section 9.3.3. The cost functions adopted to simulate WSUD scenarios in SUSTAIN are described in Section 9.3.4. The analysis of results was conducted in accordance with Section 9.3.5. The results of the analysis are presented in Section 9.4 with a discussion of these results and other commentary regarding the application of SUSTAIN in Section 9.5.

9.3.1 Site Selection

The catchment selected for this study was a subcatchment of the Paddocks catchment, detailed previously in Section 7. The 6.5 Ha subcatchment represents five subcatchments from the original Paddocks model detailed in Section 7 and is illustrated in Figure 9-1.

Figure 9-1 – An aerial view of the study area and its sub-catchments

9.3.2 Catchment Properties

Existing development data was acquired from the analysis of the catchment and calibration of the model of the entire Paddocks catchment in Section 7. A total of 46 houses existed in the five subcatchments of the study area in the current development state. For the infill condition, it was assumed that 1 in 2 single home allotments were demolished and replaced with 2 new homes on the same allotment, producing 69 homes in the infill development case. The properties of new allotments were assumed based on an analysis of infill development characteristics in Section 6.3.7, and are summarised in Table 9-1. The resulting pervious and impervious areas of each sub-catchment are shown in Table 9-2 prior to and following assumed levels of infill development.

Table 9-1 – Properties of new allotments in the infill development scenario

Houses	2
Roof (m^2)	400
Paving (m^2)	70
New connected imp (m^2)	470
new indirect imp (m^2)	30
Total impervious (m^2)	500
Connected impervious per allot (%)	54.20
Total (%)	0.57

Table 9-2 – Impervious and pervious percentage in the catchment

Sub catchment	**Area (hectare)**	**Pre development**		**Post development**	
		Impervious (%)	Routed (%)	Impervious (%)	Routed (%)
C1	0.53	44.7	55	62.6	32.3
C2	2.15	47.4	39	70.5	18.8
C3	0.17	44.7	24	66.1	1.8
C4	2.12	44.7	41	63.7	19.2
C5	1.49	43	55	56.5	32.3

The hydrologic parameters of the model were based on the calibrated model described in Section 7. The values of these hydrologic parameters are listed in Table 9-3.

Table 9-3 – Parameters of the SWMM model used for each subcatchment

Parameter	**Value**
N-Impervious	0.01
D-Store Impervious	0.0196
N-Pervious	0.02
D-Store Pervious	0.098
% Zero Impervious	0
Maximum infiltration rate (in/hr)	4.1713
Minimum infiltration rate (in/hr)	0.00787
Decay constant (1/hr)	1
Drying time (days)	14
Maximum infiltration volume (in)	0

Additional information required to run SUSTAIN, including spatial data (GIS maps, catchment properties, land use) and climate data (6-minutes precipitation and average monthly evaporation from the BOM gauge at Parafield Airport, 023013) was collected from the data sources used for model development in Section 7, including local government (City of Salisbury) and state government (Department for Environment, Water and natural Resources) sources.

9.3.3 Water Sensitive Urban Design Measures

Two types of WSUD scenario were examined using the SUSTAIN optimisation tool. These included retention tanks (the 'cistern' node in SUSTAIN) and detention tanks (using an adapted 'bioretention' node in SUSTAIN). It was assumed that each of the 46 new homes was constructed with one WSUD system – a retention tank, detention tank or rain garden. In all cases, the connected roof area was assumed to be 200 m^2. The roof connection was higher in this case than in the previous scenarios presented in Sections 6, 7 and 8 because in the previous cases, 100 m^2 was considered a more achievable outcome. However, when a 100 m^2 roof connection was assumed in SUSTAIN, it was difficult to find an adequate solution, and the connected impervious area was doubled to achieve a reasonable solution from the SUSTAIN model.

Rainwater tanks were simulated using the 'cistern' node in SUSTAIN, while detention tanks were studied by applying the bioretention node with different properties to suit their function. A sample layout of the catchment in SUSTAIN is shown in Figure 9-2. The bioretention node was adapted for detention systems because in trial runs of simulation detention using a cistern node, it was apparent that the inclusion of an orifice on the cistern, though available, was not having any effect on tank emptying. As such, the surface storage of a bioretention was used to represent a detention volume.

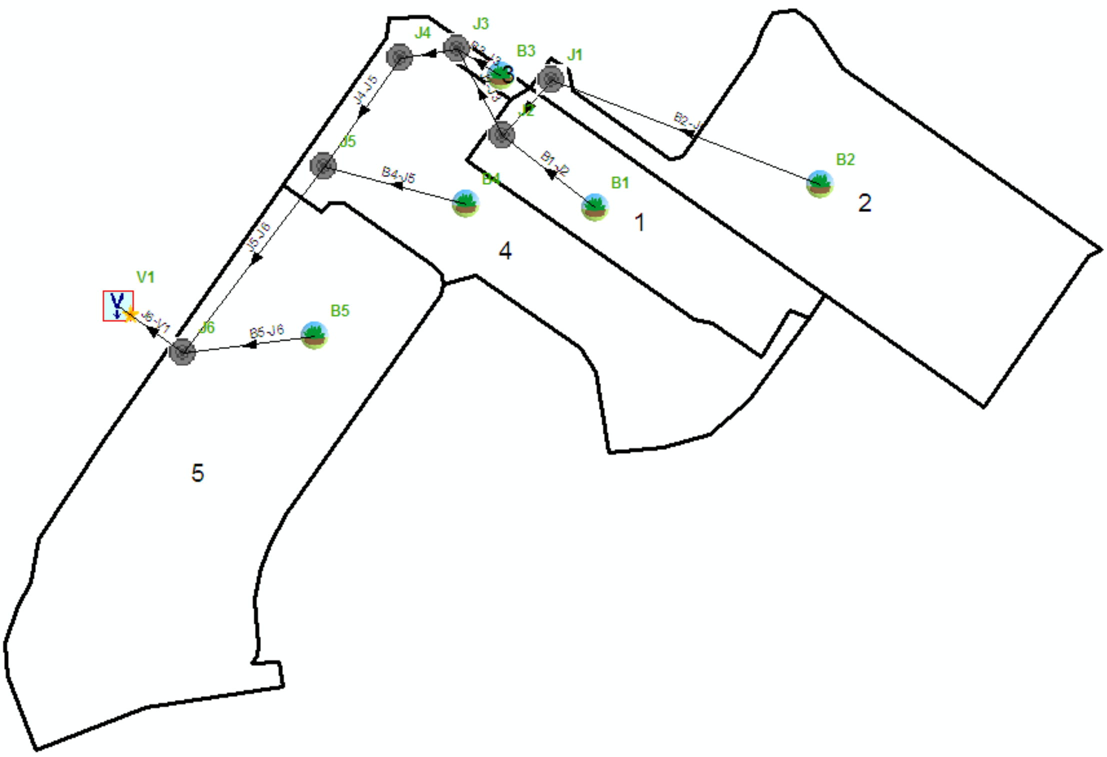


Figure 9-2 – layout of the case study catchment in SUSTAIN

Retention tanks were assumed to be attached to each new home in the model. There were 46 tanks spread evenly across the catchment. In all scenarios, SUSTAIN was used to optimise the size of tanks from 0 kL to 10 kL (based on optimising tank diameter from 0 to 11.7 ft.). All tanks were assumed to be 1 m high (represented by a weir height of 3.3 ft.) and connected to 200 m^2 of impervious area (roof). The optimal outcomes for a maintaining 2 and 5 year ARI peak flows with a tank demand of

100 L/day, 200 L/day, 500 L/day, 1000 L/day, 2000 L/day and 5000 L/day were determined. The influence of tank demand was analysed with repeated optimisation runs at for both the 2 year ARI and 5 year ARI. The fixed properties of on-site rainwater tanks (simulated using the cistern node) are shown in Table 9-4.

Detention tanks were simulated using the bioretention node by adjusting bioretention system node surface storage and outflow properties, and disabling the infiltration and underground storage properties. The number of tanks in each sub catchment was fixed at the number of redeveloped lots. In all scenarios, SUSTAIN was used to optimise the size of tanks from 0 kL to 10 kL (based on optimising the length of the system from 0 to 16.41 ft.). All tanks were assumed to be 1 m high (represented by a weir height of 3.28 ft.) and connected to 200 m^2 of impervious area (roof area). The optimal outcomes for tank orifice size values of 12 mm and 25 mm were examined by iterative optimisation runs with different orifice sizes for preserving the 2 Year and 5 Year ARI (SUSTAIN was not able to optimise this property). The properties of the assumed on site detention tanks, simulated using an adapted bioretention node, are shown in Table 9-5.

Table 9-4 – Properties of retention tanks using the SUSTAIN cistern node

Property	**Value**
Diameter	0 to 11.7
Drainage area (m^2)	200
Orifice diameter (ft.)	0
Orifice height (ft.)	0
Weir height (ft.)	3.3

Table 9-5 – Parameters of the detention and rain garden units applied in SUSTAIN using the rain garden node

Property	Detention Tank
Length (feet)	0 to 10.764
Width (feet)	10
Drainage area (m^2)	200
Orifice Diameter (in)	0
Weir Height (feet)	0.5
Orifice Height (feet)	0
Rectangular Weir (feet)	1
Depth of soil (feet)	3
Soil porosity	0.4
Soil field capacity	0.25
Soil wilting point	0.15
Initial surface water depth (feet)	0
Initial moisture content	0.15
Saturated soil infiltration (in/hr)	1
ET multiplier	1
Storage depth (feet)	0.5
Media void fraction	0.5
Background infiltration (in/hr)	0.5
Infiltration method	Horton
Suction head (in)	3
Initial deficit (fraction)	0.3
Maximum Infiltration (in/hr)	3
Decay Constant (1/h)	4
Drying times (day)	7
Maximum Volume (in)	0

After designing the study area, defining data layers, specifying potential WSUD units and their properties, specifying the routing network and setting parameters of each sub catchment, SUSTAIN was run for approximately one year leading up to the selected 2 year and 5 year ARI event (two separate model runs). To do this, the 'internal simulation' option was used to generate the pre- and post-infill development runoff time series for each sub catchment. The optimisation process was then initiated by defining the assessment point (the point where the flow threshold was set and the benefit of WSUD scenarios was assessed by SUSTAIN). Following this optimisation commenced. In this study the final drainage point (effectively the catchment outlet in this model) was selected as the assessment point for all of the scenarios. The assessment process was based on minimising cost and the evaluation factor was setting a threshold value on the peak discharge at the assessment point. The evaluation factor was reducing the peak discharge of the post infill development scenario to be equal to the 2 year or 5 year ARI of the pre-infill development scenario. The number of near optimal solutions was set to 1, and the model was instructed to stop searching when it could not produce a cost effective solution which saves less than $2000 expenditure. Depending on the catchment area, the number of sub catchments, the complexity of WSUD and the assessment parameters, several iterations were calculated and compared by SUSTAIN to find the optimum WSUD scenario.

9.3.4 Cost Function

The main consideration of the optimisation routine is the identification of an optimum successful arrangement of WSUD devices at minimum cost. As such, the assumed cost of each solution is one of the main components in optimisation of BMPs.

The assumed cost for rainwater tanks in this study included a fixed cost of AUD$2546 for the purchase of each rainwater tank as well as AUD$3.34 for each cubic foot of tank storage. This was based on rainwater tank costing conducted by Marsden Jacob and Associates (2007). The assumed costs for detention tanks included AUD$1907 as the fixed cost and AUD$3.34 for each cubic foot of surface area. This was determined based on the data for rainwater tanks provided by Marsden Jacob and Associates (2007), excluding the cost of pumps and pipes to reuse the water.

It should be noted that the cost difference should not affect the outcomes of this analysis. As the model was applied to take advantage of the hydraulic routing properties of the underlying SWMM model in SUSTAIN, we were unable to run optimisation scenarios in SUSTAIN that would select an optimum technology. The scenarios reported here were limited to selecting a blanket application of a either retention or detention across the catchment (in this case, attached to new homes with every redeveloped allotment) and the size of the system was optimised from zero (no tank required) up to 10 kL volume.

9.3.5 Interpretation of Results

The results produced by SUSTAIN which were extracted in this analysis included a series of text output files and a bar chart, each of which indicated optimisation results for four different conditions. These included the following scenarios:

1. Pre-development – an estimate of runoff from an equivalent greenfield catchment
2. Post-development – either the current pre-infill scenario or the post-infill scenario, which were modelled separately. This data was only used to determine the 2 and 5 year ARI of the pre-development outflow.
3. Existing - either the current pre-infill scenario or the post-infill scenario including any existing WSUD. In our case, this was always identical to the above (no existing WSUD assumed) and this data was not used.
4. Best – this result represents the optimum results of the optimisation process. In this study, it was used to refer to the extent to which WSUD could reproduce the post-infill development scenario to the previously identified pre-infill development peak flow rates for the 2 year ARI and 5 year ARI.

An example of this output is shown in Figure 9-3 which corresponds to the results of a detention tank scenario. While there was a 'results post-processor' bundled with the SUSTAIN software, it was not functional in our hardware configuration (see Section 9.5) and the interpretation of results was conducted by consulting text file output. In Figure 9-3, fixed terms are used on the x-axis which cannot be changed in SUSTAIN. The data is for a post-infill development scenario for which optimisation has been completed. As such, 'Pre-dev' refers to the peak flow rate from the Pre-development catchment; 'PostDev' and 'Existing' both refer to the peak flow rate from the post infill development catchment and Best1 refers to the peak flow rate resulting from having the optimum arrangement of WSUD in place. The target value refers to the specified flow threshold, which is the peak flow rate resulting at the catchment outlet from the 2 year ARI storm event over the catchment in the pre-infill development scenario. It should be noted that the solution for Best1 in this case was achieving the target threshold, but this may not always be possible. The solution Best1 may be above the target value if the WSUD options considered cannot be arranged to achieve the target flow rate. Also note that the model can only accept and provide results in US customary units. In this report, results have been converted manually to SI units.

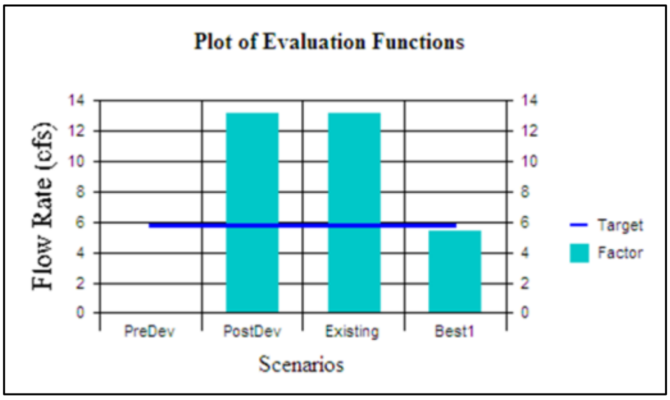


Figure 9-3 – Graphical output from the SUSTAIN optimisation procedure

9.4 Results

9.4.1 Increase in Runoff due to Infill Development in the Case Study Area

The increase in runoff flow rates due to the assumed level of infill development in the catchment is shown in Table 9-6.

Table 9-6 –Peak flow rate increase due to infill development in the catchment

	1 Year ARI	2 Year ARI	5 Year ARI
Pre-infill	0.16	0.22	0.31
Post-infill	0.29	0.39	0.50

9.4.2 Selection of Characteristic Peak Flow Events for Optimisation

The recorded peak discharge events which occurred on 26 March 2004 and 15 January 1997 were selected as the 2 year and 5 year ARI peak flow values, respectively, because these values have the closest value to estimated 2 year and 5 year ARI events in the pre-development flow time series at the end of the catchment, and there is no larger event in the preceding year or so. These values were selected to represent the threshold flows for optimisation in SUSTAIN, including the preceding 'warm up' period of approximately 12 months. Table 9-7 shows the selected 2 year and 5 year ARI storm events and the total period optimised.

Table 9-7 – Events selected for optimising the WSUD arrangement to preserve the 2 year and 5 Year ARI post development peak flow

	2 Year ARI Event	5 Year ARI Event
Flow threshold	0.22	0.31
Date/time of occurrence	26th March 1984, 12:12	15th January 1977, 17:30
Rainfall period	1st April 1983 to 28th March 1984	15th January 1976 to 16th January 1977

Figure 9-4 shows the hyetograph, current hydrograph and post-infill development hydrograph for the selected 2 year ARI event. This data is shown for the 5 year ARI event in Figure 9-5.

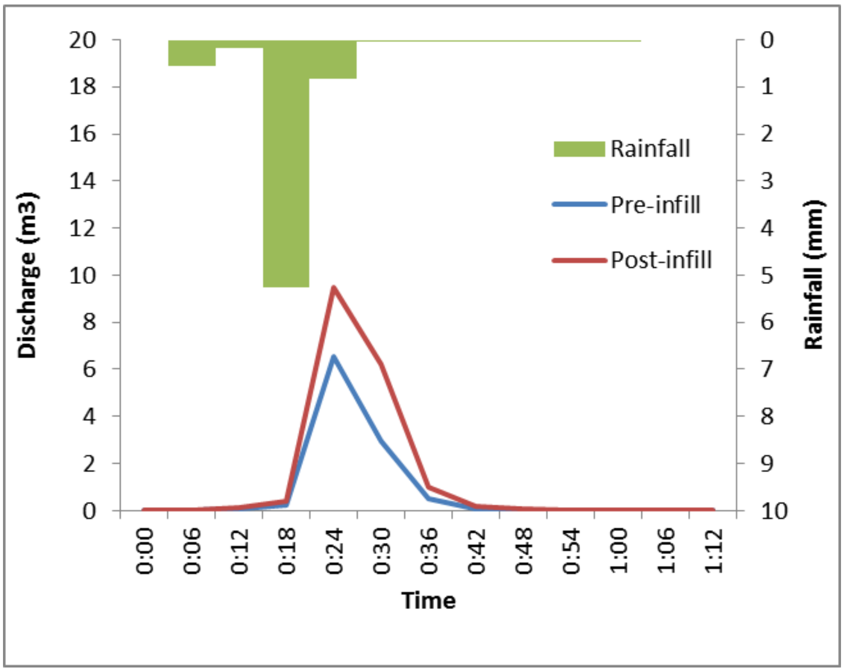


Figure 9-4 – The hyetograph and the pre- and post-infill development hydrograph for runoff during the selected 2 year ARI flow event

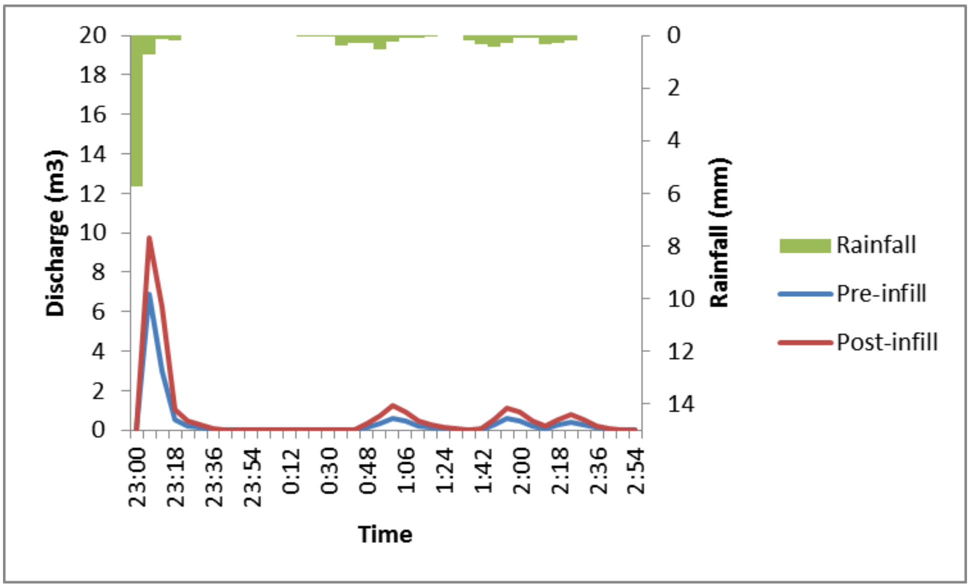


Figure 9-5 – The hyetograph and the pre- and post-infill development hydrograph for runoff during the selected 5 year ARI flow event

9.4.3 Optimisation of Retention Tanks

Rainwater tanks were able to achieve the pre-infill development flow threshold in a limited number of scenarios. To preserve the 2 year ARI and 5 year ARI peak flows from the catchment following infill development, a minimum demand (or other disposal) of approximately 450 L/day was required to reproduce the peak flow rates for the 2 year ARI storm event. Much higher demand was of approximately 5000L/day was required to preserve the 5 year ARI peak flow rates. To preserve the peak flow rate in either case, SUSTAIN placed tanks centrally in the catchment as a priority (catchment 3).

Results were unusual however because when SUSTAIN was used to produce an optimum solution to preserve the 2 year ARI peak flow with a demand of 400 L/day (just below the minimum requirement), a sub-optimal solution was produced which indicated that the 2 year ARI could not be preserved, but the tank sizes within each catchment were not maximised.

9.4.4 Optimisation of Detention Tanks

Detention tanks were able to achieve the pre-development flow threshold in a limited number of scenarios. Successful scenarios tended to provide more detention at the top of the catchment (subcatchments 1 and 2) than at the lower end of the catchment. This differed from the retention tank scenarios, where optimal placement tended to be centrally located.

The orifice size assumption was checked by undertaking multiple runs (this parameter could not be optimised in SUSTAIN). Orifice sizes between 12 to 25 mm tended to be successful, with 25 mm successful at both the 2 year ARI and 5 year ARI standard.

Of the unsuccessful runs, the optimisation procedure tended to produce tank size estimates that were maximised in almost all catchments. As noted above, this was not the case for the retention systems (using the cistern node). This may suggest that the apparent problem with the cistern node was not present for the bioretention node (adapted to simulate detention tanks).

9.5 Discussion

The US EPA SUSTAIN software tool was used to optimise the placement and design of both a retention and detention scenario in a collection of subcatchments of the Paddocks catchment in Para Hills. SUSTAIN successfully produced results for preserving peak flow rate within the catchment, producing data regarding the least cost solution for the size and placement of retention or detention storages. In the retention scenario, levels of demand (or disposal) greater than 450 L/day of retained water were required to produce an optimum result for the 2 year ARI storm event, and greater than 5000 L/day to preserve the existing peak flow of the 5 year ARI storm event. For detention systems, an orifice 25 mm was successfully used for preserving both the 2 year ARI and 5 year ARI. However, there did not appear to be any clear pattern in the placement of retention or detention tanks, with results varying each time a successful scenario was rerun. Further optimisation with multiple outcomes is recommended to produce a pattern of optimal tank arrangement.

Several shortcomings were identified which limit the useability of the tool as an everyday choice as a decision making tool. The following issues were encountered using the software:

- This model is not a self-executed software tool, it is an application run inside the ESRI ArcGIS application, Version 9.1. As such, while the software is available as freeware, it requires the user to have the ESRI ArcGIS software installed.
- The software is only capable of running on a combination of Windows XP Service Pack 2 and ArcGIS Version 9.1. This unique and currently out of date software combination requires some effort to assemble and inhibits wider use
- SUSTAIN does not appear to support the use of historic rainfall data prior to 1992. To use rainfall records prior to 1992, the user must manually change the date of the rainfall data records for the model to run. Once run, an accurate representation of time for recording purposes requires the user to manually re-convert model output to the original time stamp.
- Input to and output from SUSTAIN is restricted to imperial units, despite the fact that the underlying software of SUSTAIN, US EPA SWMM, is capable of producing results in imperial or SI units.
- The software was generally not a stable platform in the experience of this project. It was subject to several errors during model setup and operations which were defined with dialogue boxes making references to lines of code. Such error responses are not easily interpreted by the user, and the user manual provided limited guidance with respect to model input procedures and errors.
- The software does not appear to be able to handle significant change to input data well. For example, if a catchment model is setup to optimise rainwater tanks, the user cannot simply adjust the ‘cistern’ nodes of tanks in the menu provided to produce a scenario for rain gardens as an alternate scenario. While it is technically possible to make such a change using the menus provided, doing so throughout this project led to optimisation runs which produced no results. Unfortunately, there was no obvious error, and the problem was only

apparent when the parameters were subject to reanalysis on a freshly produced simulation scenario (i.e. starting with a blank map in ArcGIS and rebuilding the model). Therefore, to ensure that appropriate results are obtained in any given optimisation scenario, the user must begin a new model from a new ArcGIS file, leading to a lot of repetitive model setup. The cause of this problem was unknown and there was no guidance on this issue in the user manual.

- Results produced by SUSTAIN were generally found to be not repeatable – multiple runs of the same scenario produced results which were not equal or in some cases, not similar.
- The output processor which is bundled with SUSTAIN was not functional due to compatibility issues, despite the fact that the computer platform adopted was specifically designed to satisfy the requirements of SUSTAIN. Manual interpretation of data based on text files in the model output was required to interpret the results from this study.
- The full suite of files generated by SUSTAIN was not documented fully in the supporting documentation for SUSTAIN (Shoemaker et al., 2009). The content/structure of these files was also not fully described. Some files are similar to SWMM output, and it was beneficial that these data files are well described by Rossman (2010) and familiar to experienced users of the SWMM model. However, in many cases significant changes in both structure and terminology have taken place without explanation in the literature available for SUSTAIN.
- The terminology in SUSTAIN simulation files was inconsistent. For example, the SUSTAIN solutions for 'Existing' in a chart of results (see Figure 9-3) is referred to as Init_Eval.out and in the output files, making identification confusing for post processing of flow data.
- The software is very sensitive to the validity of input data; in the experience of this project, 'typo' errors during the repeated process of scenario development caused the software to crash with no error message
- While catchment layout data for a scenario may be saved, much of the model input data cannot be saved. This is a particularly large drawback because entering much of the model input data is a tedious and time consuming task and must be done precisely in a repeated fashion to ensure the model does not 'crash'. For example, data on the model time step and desired location for storing reference and output files must be input for each run.
- SUSTAIN requires input for features that are often not required. For example, the user must input data on maximum and minimum temperature and wind speed when these features are only required for the simulation of runoff due to snow melt in the underlying SWMM model. Without this data, the model will not run, but it is entirely unnecessary to the model outcome in many applications, particularly in South Australia.
- The model was found to be limited in its potential for optimisation scenarios. For example, if a rainwater tank 'policy' scenario is examined, it would be beneficial if the model could provide one optimal solution for the volume of multiple tank nodes across subcatchments (e.g. to explore a uniform policy rollout with respect to tanks on new homes across multiple subcatchments). However, the characteristics of WSUD features in each subcatchment are unique, and optimised in isolation. While this has benefits in identifying spatial importance of WSUD placement, the model does not allow for optimisation of WSUD characteristics uniformly across all subcatchments (e.g. what size tanks should be adopted across every subcatchment to reduce flow rates?). Where optimisation is undertaken on multiple nodes,

it would be beneficial if the user could 'tie' or group the optimisation parameter such that regardless of where a WSUD node is placed, there is a single result across the catchment. The result may be technically sub-optimal, but it may provide guidance on an optimal policy setting which is socially equitable across a catchment.

Despite these shortcomings, SUSTAIN was found to be effective at producing an optimum result to reduce flow rates. It is recommended that SUSTAIN be applied to larger catchment areas to determine whether it can produce consistent advice on the optimal placement of retention or detention based WSUD. For example, application to the total Paddocks catchment (76 Ha) and application to a broader watershed, such as the Dry Creek, or Torrens catchment, is recommended to further explore the potential application of SUSTAIN for optimising WSUD strategies for flow management. The capability of SUSTAIN for water quality improvement modelling (for example, by specifying a reduction in the total annual load of a particular pollutant) may also produce effective results at the macro-level for projects associated with the Adelaide Coastal Water Quality Improvement Plan (McDowell & Pfennig, 2013).

9.6 Summary

The application of optimisation tools to explore WSUD alternatives in a catchment was explored using the USEPA SUSTAIN optimisation software. Five subcatchments of the Paddocks catchment in Para Hills were selected for the case study. The tool was used to identify the most cost effective arrangement of on-site retention or detention to maintain pre-infill development peak flow rates. SUSTAIN successfully produced a runoff time series from the urbanised catchment pre- and post-infill development, and provided optimal solutions for the distribution of retention and detention based scenarios in some circumstances. Recommendations were not consistent however, with retention or detention recommended in various arrangements when identical optimisation runs were repeated. This may be because the case study catchment was too small. There were several difficulties encountered in the application of SUSTAIN. The most significant included a generally unstable operating environment and the requirement for an out of date operating system and ArcGIS software which inhibit recommendations for wider application at this stage. For research purposes, it is recommended that SUSTAIN be applied to larger catchment areas to determine whether it can produce consistent advice on the optimal placement of retention or detention based WSUD.

The US EPA SUSTAIN software tool was used to optimise the placement and design of both a retention and detention scenario in a collection of subcatchments of the Paddocks catchment in Para Hills. The objective of the exercise was to produce a least-cost implementation of WSUD to preserve the current 2 year ARI and 5 year ARI peak flow rates of a residential catchment following infill development.

10 Application of the MUSIC model to Urban Catchments in South Australia

10.1 Introduction

The model for urban stormwater improvement conceptualisation (MUSIC) is a hydrological tool developed by eWater which has been widely used in Australia to help urban stormwater professionals visualise strategies to tackle problems with urban stormwater hydrology, harvesting and pollution. The MUSIC model estimates stormwater flow and pollutant generation and simulates the performance of stormwater treatment devices individually and as part of a treatment chain to provide information on whether a proposed system can achieve flow and water quality targets.

Like any other hydrological model, the reliability of predictions depends on how well the model structure is defined and how well the model is parameterised. However, estimation of model parameters is difficult due to the uncertainties involved in determining parameter values, which cannot be directly measured in the field. Therefore model calibration where observed data is available is necessary to improve model performance. The calibrated model parameters can then be adopted with care in other areas with similar characteristics where there is no observed data for calibration.

To develop a MUSIC model, access to a series of guidelines which provide adequate information regarding modelling steps and standard model parameters is of great benefit for model users and those who use the model output. Such guidelines exist in other jurisdictions, such as Melbourne (Melbourne Water, 2010), South East Queensland (WaterbyDesign, 2010). Some local government have also developed their own guidelines. Examples include Mackay, QLD (DesignFlow, 2008) and Strathfield, NSW (Equatica, 2011). It is understood that there is currently no such guideline for MUSIC modelling in South Australia to ensure a minimum standard and consistent approach. This study was therefore an attempt toward preparing MUSIC guidelines suitable for regions in South Australia.

10.2 Aims

This project aims apply MUSIC to estimate the runoff hydrograph of a catchment in South Australia. In accomplishing this, this study aims to provide some recommendations regarding the estimation of rainfall-runoff model parameters used in MUSIC to help South Australian practitioners choose adequate parameters where there is little or no observed data for calibration.

10.3 Methodology

10.3.1 Rainfall-Runoff Model in MUSIC

The algorithm adopted to generate urban runoff in MUSIC is based on a simplified rainfall-runoff model developed by Chiew et al. (1997) and is shown in Figure 10-1. The model was initially developed using a daily time step, but has been modified for incorporation into MUSIC to allow disaggregation of the generated daily runoff into sub-daily temporal patterns at a minimum of six minutes.

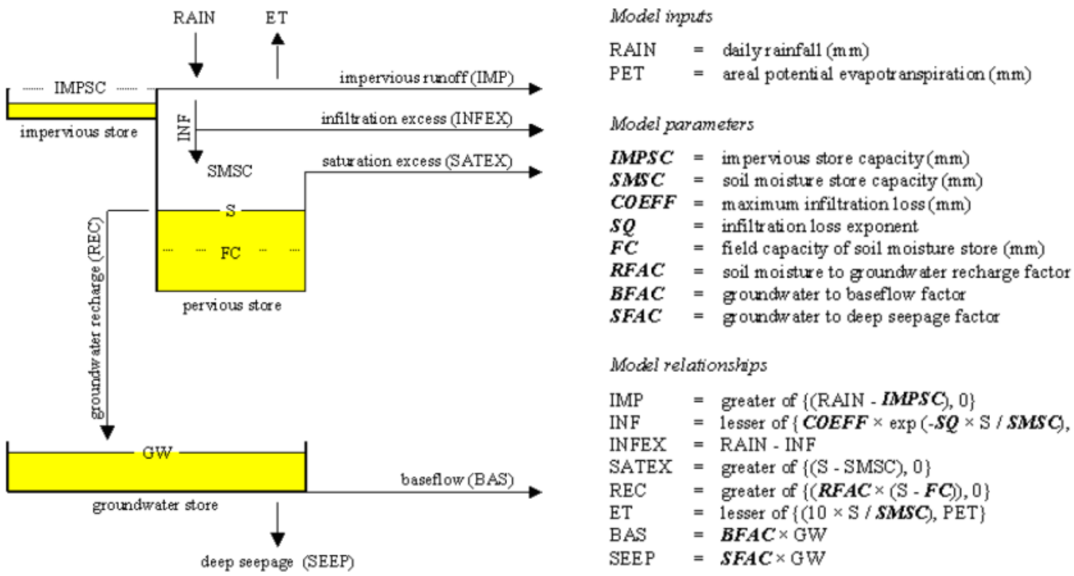


Figure 10-1 – Conceptual daily rainfall-runoff model adopted for the MUSIC model (eWater, 2012)

Flows from the directly connected impervious area (often called effective impervious area) and the pervious area (which includes indirectly connected impervious area and pervious areas) are modelled separately, with impervious area runoff being a function of the proportion of catchment imperviousness with an initial loss term. The imperviousness in MUSIC is referred as the effective impervious area which is a percentage of the total impervious area and it is a measure of the area of land that is directly connected to the stormwater drainage system. It should be noted that unlike other popular models used in Australia such as DRAINS and SWMM, the MUSIC model does not account separately for indirectly connected impervious area. The extent of pervious area runoff depends on soil properties and antecedent conditions. Generally, runoff from pervious areas will only occur during large or intense storm events, when the pervious soil storage reaches saturation or where rainfall intensity is too high to be intercepted.

In the MUSIC model, each node requires the total sub-catchment area and the proportion of effective impervious area to be defined. These values together with the rainfall data and soil properties define the runoff generated from the modelled catchment area. The MUSIC rainfall-runoff model parameters are shown in Figure 10-2.

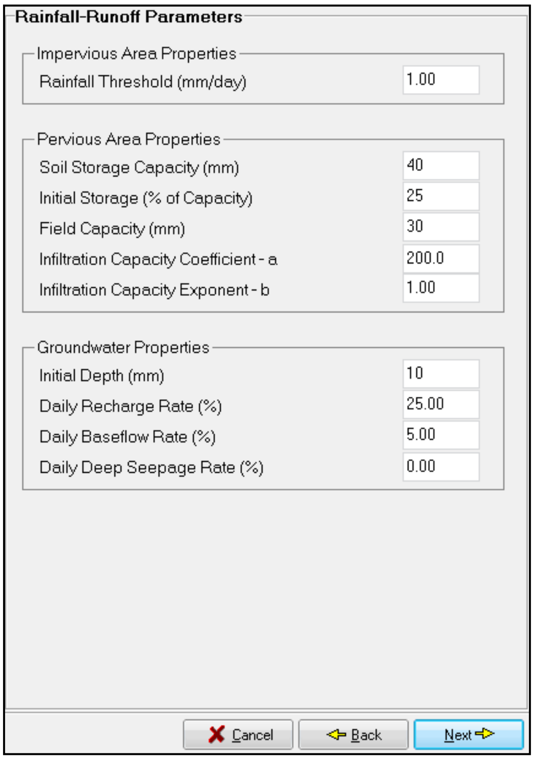


Figure 10-2 – MUSIC's Rainfall Runoff Parameters

Based on sensitivity analysis conducted by Dotto et al. (2011) the most sensitive parameter in the MUSIC rainfall runoff model was found to be the effective impervious area percentage and it was recommended that satellite images be used to determine the total impervious area and the impervious areas that are directly connected to the drainage network. The Muskingum Cunge translation factor (*k*) is another very significant parameter for estimating peak flows and runoff timing, as it reflects the travel time of the flood wave throughout the drainage system. This becomes important when simulating larger catchments where the influence of flood routing becomes important.

Dotto et al. (2011) also indicated that once the effective impervious area percentage is greater than 30%, the adjustment of pervious area parameters has little significance in improving runoff prediction. When the level of urbanisation and consequently effective impervious area is lower than 30% two key pervious area parameters (i.e. soil storage capacity and field capacity) become more important. It was suggested that these parameters may be safely applied within the recommended lower and upper limits or simply fixed to their default values when the model is applied to urban catchments with effective impervious areas greater than 30%.

The MUSIC model manual (eWater, 2012) provides recommendations for the simulation of runoff in urban catchments across Australia. For Adelaide catchments, it is recommended that a soil store capacity of 40 mm and a field capacity of 30 mm provides be adopted as a starting point for calibrating gauged catchments, and has been used as an estimate for the properties of ungauged catchments in previous works (GIWR, 2011). There is no information provided on what analysis was undertaken to produce these recommendations, but they are used as a starting point in the calibration undertaken in this report.

10.3.2 Case Study catchments

Figure 10-3 and Figure 10-4 show the MUSIC model layout of two fully urbanised case study catchments within the Adelaide metropolitan area where observed flow data was available. These catchments were the Frederick Street catchment and the Paddocks catchment, which were previously described in Sections 6 and 7 of this report, respectively. These catchments and their properties have been considered for calibration in MUSIC and the results of the modelling have been compared with the results from existing models of these catchments prepared in the SWMM model applied previously.

The data required to set up the MUSIC models such as climate data, flow data and the characteristics of the proposed catchments were obtained from Section 6 (Frederick Street) and 7 (Paddocks). On both catchments there was data from two rainfall gauges for rainfall input and one flow gauge for calibration at the catchment outlet. However, unlike SWMM which allows the modeller to use multiple climate data sources across a catchment, MUSIC uses a single rainfall data input to generate runoff time series at all nodes in a model. It was therefore necessary to prepare an average climate data file based on data from the two rain gauges in each catchment.

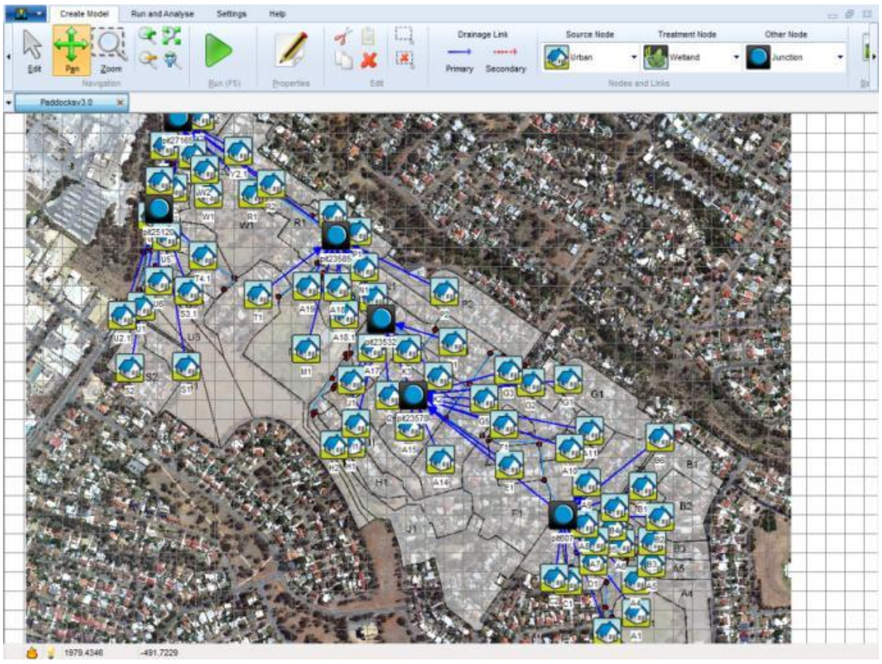


Figure 10-3 – The Paddocks catchment model layout in MUSIC

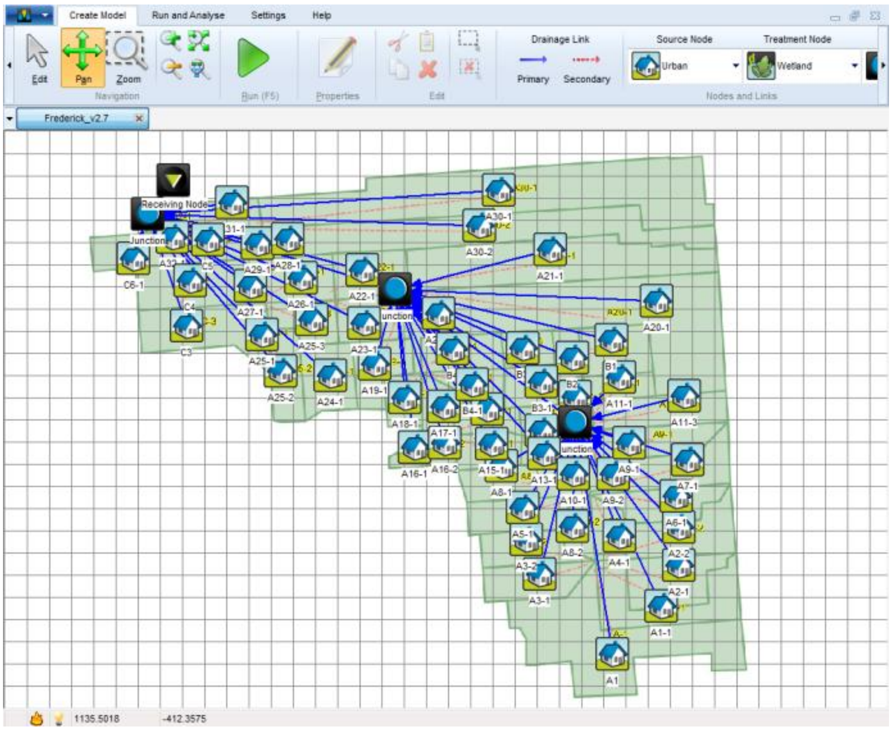


Figure 10-4 – The Frederick Street catchment model layout in MUSIC

From existing data, the average effective impervious area in both catchments in 1993 (where flow data was available) was around 30%. Based on the information provided by Dotto et al. (2010), the pervious area parameters may play an important role in generation of runoff in the MUSIC model. Therefore calibration was performed manually on both catchments using flow data collected over a period of almost 3 years (1992-1995) to optimise key rainfall runoff parameters. A number of events were considered for calibration analysis during this period and the modelled flow time series were compared with observed flow data. For comparison with previous calibration runs, the calibration data was also compared to the modelled flow data from SWMM modelling for Frederick Street (Section 6.3.3) and the Paddocks (Section 7.3.3).

10.4 Results

As shown in Table 10-1, for the Frederick Street catchment, the default values provided in the MUSIC manual for soil storage capacity (40 mm) and field capacity (30 mm) in Adelaide were found be to be the most effective. It was found that these values, in addition to the Muskingum Cunge factors (k = 18 minutes, theta = 0.49) had the most impact on acquiring the correct peak flow rate from the catchment.

However, this was not the case for the Paddocks catchment, with the results shown in Table 10-2. In the Paddocks, the calibrated values for soil storage capacity and field capacity were 250 mm and 230 mm, which is different from the MUSIC default values for Adelaide but still within the bounds of the recommendations of the MUSIC manual (in fact equal to the values recommended for Perth). The Muksingum Cunge routing values adopted were k = 11.2 minutes and theta = 0. Figure 10-5 and

Figure 10-6 show a comparison of modelled data and observed data for the particular event for both catchments. A summary of all calibration statistics is provided in Appendix C.

Table 10-1 – Frederick Street catchment information

Name	Fredericks Catchment
Study Period	1992-1995
Study Area	45 Ha
k, min	1080
theta	0.48
Calibrated soil storage capacity	**40 mm (default = 40 mm)**
Calibrated Field Capacity	**30 mm (default = 30 mm)**
Average Imperviousness	31%
Total Modelled Volume	171,912 m^3
Total Observed Volume	177,015 m^3

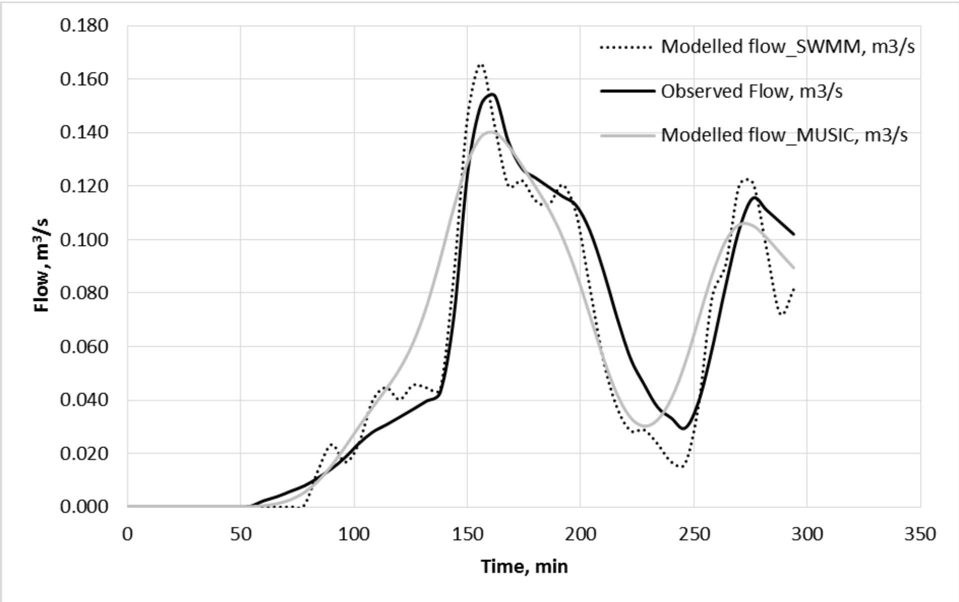


Figure 10-5 – Comparison of Observed data and Modelled flow data, 11/07/1992, Frederick Catchment (Soil Storage Capacity= 40mm – Field Capacity= 30mm)

Table 10-2 – The Paddocks catchment information

Name	Paddocks Catchment
Study Period	1992-1995
Study Area	69 Ha
k, min	672 min
theta	0
Calibrated soil storage capacity	**250 mm (default = 40 mm)**
Calibrated Field Capacity	**230 mm (default = 30 mm)**
Average Imperviousness	28%
Total Modelled Volume	260,290 m^3
Total Modelled Volume using default values	445,730 m^3
Total Observed Volume	237,956 m^3

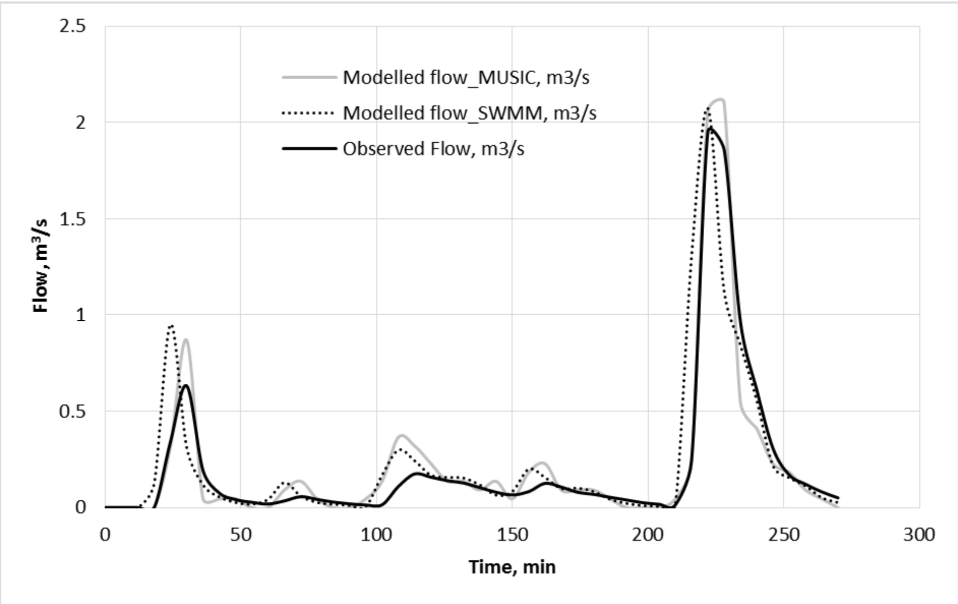


Figure 10-6 – Comparison of Observed data and Modelled flow data, 17/11/1992, Paddocks Catchment (Soil Storage Capacity= 250mm, Field Capacity = 230mm)

To make sure the calibrated parameters were reasonable, the pervious area parameters which were dependent on the soil texture were compared with the suggested ranges provided by Macleod (2008) in Table 10-3. To achieve this, the dominant soil type of both catchments was first identified using soil data from the South Australian government (Government of SA, 2010) and the corresponding values from Table 10-3 were used for validation. Based on the Adelaide soil data, the dominant soil type in the Paddocks and Frederick Street catchments were “Sandy clay Soil” and “Silty clay Soil”, respectively.

It should also be noted that the selection of appropriate values for the Muskingum Cunge flow routing was important to produce a good estimate of peak flow and representative timing of the overall hydrograph with respect to the measured data.

Table 10-3 – Pervious Area Soil Storage Capacity and Field Capacity (Macleod, 2008)

	Soil Storage Capacity		Field Capacity	
Dominant Soil Description	0.5m root zone	1.0m root zone	0.5m root zone	1.0m root zone
Loamy sand	139	279	69	134
Clayey sand	107	214	75	145
Sandy loam	98	195	70	135
Loam	97	194	79	154
Silty clay loam	100	200	87	167
Sandy clay loam	108	217	73	138
Clay loam	119	238	99	189
Clay loam (sandy)	133	267	89	169
Silty clay loam	88	175	70	133
Sandy clay	142	283	94	179
Silty clay	54	108	51	96
Clays	93	187	68	127

Paddocks catchment, Soil storage capacity = 250 mm, field capacity = 230 mm

Frederick Street catchment, Soil storage capacity = 40 mm, field capacity = 30 mm

10.5 Discussion

From these analyses it is believed that the proposed calibrated parameters can be adopted for other urbanised catchments within Adelaide metropolitan area with similar characteristics where there is no data available for calibration. However there appears to be some variation in the soil storage capacity and field capacity values between the two soil types, which should be further explored before implementing recommendations in a guideline for applying MUSIC for Adelaide catchments, particularly where the connected impervious area is less than 30%. The importance of applying the correct parameters is emphasised by the finding that applying default parameters from the MUSIC manual for Adelaide (eWater 2012) to the Paddocks catchment, produced too much runoff from the catchment, with approximately 70% excess runoff produced in the model compared to the observed. Production of excess runoff can lead to over estimation of pollutant export, and an over-sizing of treatment systems. Oversizing of treatment systems can lead to increased costs of construction and an unnecessarily large footprint, which may discourage implementation of WSUD by practioners. It should be noted that the opposite effect may also occur if flow volume is underestimated. Under estimation can lead to the underestimation of pollutant loads, an underestimation of the size required to adequately treat stormwater runoff from a catchment and the implementation of a WSUD system that is not providing the anticipated service.

It is stressed that MUSIC only accounts for directly connected impervious area, and the reported pervious area is made up of indirectly connected impervious area and actual pervious areas. Other models (DRAINS, SWMM) have three separate areas, adding the rainfall depth from the indirectly connected impervious area to the pervious area, and applying the loss model to the pervious area alone. When calibrating the loss model in MUSIC, it is subsequently applied to the total of indirectly connected and pervious area, so in urban areas with substantial indirectly connected impervious areas the calibrated loss will not be like that from a pervious area in isolation. This feature will make it difficult to recommend the MUSIC model for use in ungauged urban catchments due to variations in the nature of connected and indirectly connected impervious area and the impact this can have on runoff. However, as discussed, once the effective impervious area percentage is greater than 30%, the adjustment of pervious area parameters has been shown to have little significance in improving runoff prediction, therefore the MUSIC model can be used to model runoff volumes across time. However, due to the dependence on Muskingum Cunge routing factors for which there are no recommendations in ungauged urban catchments provided in the MUSIC model, MUSIC should not be used for the prediction of peak flows.

Within highly pervious catchments the hydrology is more complex than urbanised areas with high proportions of impervious surfaces. Factors including rainfall interception, rainfall intensity, catchment slopes, soil field capacity, soil drainage, interflow rates, groundwater recharge, evapotranspiration rates and infiltration rates may each have a significant influence on the hydrologic cycle and may vary between sites. Modelling of highly pervious catchments in MUSIC should be undertaken with care, with model results checked against available gauged data.

10.6 Summary

The suitability of the MUSIC model as a tool for stormwater quantity (volume) assessment in South Australia was investigated to examine its application for estimating runoff volumes and flow rates, which is an important preliminary step in examining the effectiveness of WSUD strategies. The study applied the model to identify suitable parameters for South Australian urban catchments based on the known parameters of the Frederick Street and Paddocks catchments. The results indicated that MUSIC provided a good estimate of the flow volume and peak flows with input parameters derived from calibration. Using the default parameters provided in MUSIC revealed an error in flow volume estimation in the order of 70%. The analysis found that different input values were required for the two catchments and a further investigation is necessary to assess the use of MUSIC. This will be necessary to develop guidelines for practitioners to apply the model with more confidence to assess pre- and post-development flow conditions in ungauged catchments with different properties.

11 Conclusions and Recommendations

This section draws together conclusions and recommendations that were produced based on the reported results. It includes conclusions and recommendations from the current study, and recommendations for future research to support policy and planning in South Australia.

11.1 The role of various WSUD Approaches

In Section 2, the role of WSUD at different development scales was investigated based on literature review and a comparison of published recommendations. A summary table was produced to cross match the recommendations of the Australian guidelines. It was found that existing guidelines generally agree on the role played by WSUD tools with respect to development type. Disagreement may be attributable to the definitions of development type and scale used by guidelines, as these terms were not well defined in existing guidelines.

Recommendations

- The study recommended that future updates to WSUD guidelines consider the findings of this review when providing advice on selecting WSUD tools with respect to scale
- The study recommended that future guidelines express the definition of development type and scale clearly

Future Research Opportunities

- It was considered beneficial to supplement this review with information on the effectiveness of WSUD at different development scale with respect to the design goal. For example, if stormwater runoff is a priority and quality is important, a rain garden may be expected to provide more benefit than a detention system. Similarly, if water demand reduction is important, then the application of rainwater retention systems is of greater benefit than infiltration or detention measures (Section 2.4).

11.2 Identification of a Methodology to Assess the Impact of WSUD on Catchment Flow Characteristics

Section 3 of this task report included the development of approaches to examine the effectiveness of WSUD systems to restore flow rate and volume in urban catchments. Based on a critical review of previous studies, a new approach was developed based on long term simulation and partial series analysis of the resulting flow time series. Continuous simulation avoids the need to make assumptions regarding catchment soil characteristics and the WSUD system storages prior to the start of a design storm. The new approach was used throughout the report for assessing the impact of greenfield development, infill development and WSUD systems on flow rate, volume and/or flooding in case study catchments.

Recommendations

- The developed approach was applied successfully throughout this task report, and it is recommended that this approach be considered as a tool for assessing the impact of greenfield development, infill development and WSUD strategies on the runoff volume and peak flows.

Future Research Opportunities

- The partial series analysis approach to investigating flow was applied to simulated flow data to examine the effects of greenfield development, infill development and application of WSUD to these development cases. It is recommended that verification is undertaken to ensure that the simulated data for post-infill and WSUD cases corresponds with simulated outcomes.

11.3 Case Study 1: Performance Assessment of B-Pods in Union Street, Dulwich

Section 4 provided an assessment of the impact of B-pods in the City of Burnside streetscape. Based on the methodology developed in Section 3, the study simulated a flow regime prior to and following the implementation of B-Pods in a suburban street. It was found that the B-Pods had minor impacts on peak flow rates and flow volumes from the streetscape. Better results were found when the clay subsoil was assumed to be sand (due to higher infiltration rates) but there was little benefit provided by increasing the size of the B-Pods.

Recommendations

- The impact of B-Pods on peak flow rates and volume was not large, but it is not recommended that the B-pod systems are considered unsuccessful. A truly fair assessment should consider the total cost and benefit of such systems, including the cost of construction (as part of scheduled kerb and gutter works) and their potential savings in terms of street tree irrigation.

Future Research Opportunities

- B-Pods are an example of kerbside irrigation. Their installation presents an opportunity for investigating the impact of kerbside irrigation on the adjacent road, kerb and gutter over an extended period. Such a study will provide information to practioners on the impact of passive irrigation and infiltration on the verge and in the median strip of road infrastructure.

11.4 Case Study 2 – Rain Gardens in Tarragon Street, Mile End

Section 5 provided an assessment of the impact of street scale rain gardens in the City of West Torrens streetscape. Based on the methodology developed in Section 3, the study simulated a flow regime prior to and following the implementation of rain gardens in a suburban street. The results indicated that the rain gardens had little impact on the volume of runoff generated from the catchment. However, the rain gardens provided benefits in the form of detention which reduced the peak flow rate of runoff from the catchment area. The peak flow reductions were higher for more frequent storm events. The removal of an impermeable liner at the base of the rain garden to allow infiltration to occur from the system produced improvements in the stormwater runoff volume. However, allowing for infiltration at the base of the system had negligible impact on the peak flow rate even when well-draining soils were assumed present.

Recommendations

- Street scale rain gardens showed greater benefits compared to B-Pods in terms of impact peak flows, effectively detaining runoff to reduce peak flow rates. It is recommended that street scale rain gardens be considered as a means of reducing peak flows in developing catchments.

Future Research

- Based on the peak flow benefits of the assumed rain garden design, it is recommended that future research explore the best design and arrangement for rain gardens in environments subject to infill development. Such research could include assessing potential improvements to rain garden placement, numbers and design features. It is however important that any changes to design do not affect public safety and garden aesthetics.
- This study did not consider external benefits provided by the rain garden systems such as the cost and benefits of streetscape aesthetics, car parking and the provision of flora and fauna habitat. There is an opportunity for a complete cost benefit analysis on the service provision of street scape rain gardens.

11.5 Case Study 3 and 4 – Frederick Street Catchment and Paddocks Catchment – Infill Development

Sections 6 and 7 explored the impact of stormwater retention and detention in two catchments with infill development scenarios. The results suggested that WSUD in the form of on-site retention or detention in association with new development was unable to completely maintain the existing peak flow rates observed in each catchment prior to infill development. This was true for both the larger catchment on a relatively high grade (The Paddocks) and for a smaller catchment with a lower grade (Frederick Street).

Recommendations

- The implementation of retention or detention tanks at infill development sites with a limited connected impervious area similar to that assumed in this research (100 m^2) showed a limited potential to improve peak flow rates. However, retention and detention based systems were not able to restore the pre-infill development flow regime under the assumed conditions. A complete retrofit of every allotment in the catchment with retention or detention, or the construction of street scale rain gardens, was effective at maintaining peak flow rates at pre-infill development levels. This indicates that any retention or detention driven policy for flow management should seek to ensure that connected impervious area is considered during policy development.

- In the development of a policy for on-site retention or detention, the catchment characteristics, extent of development and the layout of existing drainage may be different to that assumed in this study. For example, a detention system may not be able to drain to the street via gravity in all allotments. Such circumstances should be considered when assessing the potential for a catchment retrofit or a requirement for allotment redevelopment.
- There was little difference between applying retention or detention for a fixed tank size. This indicates that the potential peak flow rate and flooding benefits under the conditions simulated in this report may be discounted when selecting one option in favour of another for a developing catchment. However, these results should be explored on smaller and larger catchments (1 Ha to more than 100 Ha) to investigate the occurrence of any lagging flow issues which were not apparent in the situations examined in this report.
- Further to the point above, a cost benefit comparison between retention and detention measures should consider the additional benefits of retention (through harvesting/reuse or infiltration) and its costs (such as pump infrastructure, maintenance and potential issues where reuse demand does not meet desired levels).
- This research project has focussed strongly on flow rate. However, the WSUD systems examined in this report should be examined in a holistic manner when considering policy outcomes. For example, while the 1 kL rainwater tanks assumed in this study showed limited benefits to end of catchment flow rates when added to new homes, these tanks showed benefits in terms of their impact on mains water demand reduction. These tanks also provide other unquantified impacts in the social realm (such as improved awareness of water harvesting and reuse and broader sustainability concepts).
- The results for Fredrick Street and the Paddocks catchment each indicated that for detention systems, an orifice size between 20 mm to 40 mm was most effective at reducing peak flows. For retention tanks and detention tanks, there was generally a benefit achieved by increasing a tank size from 1 kL to 5 kL, but little additional benefit by increasing tank size beyond 5 kL.

<u>Further Research Opportunities</u>

- In this research, the connected impervious area was based on a roof area that may reasonably be assumed to be connected to an above ground tank. An underground tank may be a suitable alternative, as higher quantities of impervious area can be connected to them and fed by gravity. In light of this, further research is recommended to explore the effectiveness of the assumed tank systems in this research with greater connected impervious area.
- Using the approach in Sections 6 and 7, further research is recommended using urbanised catchments to verify that the study findings still apply at a smaller and larger scale. This would involve repeating this study for catchments approximately 1 Ha, and more than 100 Ha in size.
- The current approach to provide acceptable drainage in existing urban catchments where the stormwater system is under stress is to design and construct upgraded drainage systems. However, this research demonstrated that a complete catchment retrofit with

retention, detention, or rain gardens can be effective at preserving peak flow rates at pre-infill development levels. According to the DEWNR (2013), costs are an important consideration in the planning stage of a WSUD project. Based on this, it is recommended that the economic costs and benefits of effective retention, detention and rain garden strategies are assessed and compared with an equivalent stormwater drainage design and upgrade scenario to determine which option provides the most cost effective means of preserving peak flows in catchments where infill development is occurring. Such a study could include an economic assessment based on tangible costs only, as well as one which includes intangible costs (such as water quality and ecosystem health).

- There is a possibility that combined retention and detention systems may provide a more effective outcome than either type of on-site measure in isolation, and the potential to implement such systems could be explored further using the approach in this report.
- The results for Fredrick Street and the Paddocks catchment each indicated that for detention systems, an orifice size between 20 mm to 40 mm was most effective at reducing peak flows. However, this may vary depending on the size of the catchment. It is recommended that the methodology in this study is repeated for a very large catchment to explore whether the most effective orifice size remains in this range. It may be possible to vary detention tank orifice size across very large catchments to achieve a fixed goal at the end point, or in key points across the catchment. Such locations may include points with limited drainage capacity, high ecological value or points where stream bank stability may be compromised.
- It is recommended that further research explore the impact of retrofitting street or precinct scale detention and retention tanks using the continuous modelling techniques developed as part of this research. Implementation of street scale or precinct scale systems may be a cheaper alternative to retrofitting systems to every allotment. It is acknowledged however that such systems would require space which may not be readily available in existing urban areas.
- According to the continuous simulation of runoff from the current state of both Frederick Street and the Paddocks catchments, each was exhibiting flooding for events lower than the 5 year ARI, a level generally adopted for minor system capacity. The fact that these systems were already under-capacity may have impacted the study results, and the investigation of a system closer to the design capacity should be considered to verify the outcomes.
- The current research has not considered the impact of infill development on runoff quality, which is an area of interest to local and state authorities. It is recommended that further work is undertaken to consider the impact of retention and detention pollutant loads downstream, and any improvement provided by on-site and street scale measures.
- The scope of this project did not consider the impact of sites designated as 'transit oriented development' which may produce localised areas of very high connected impervious area in Adelaide. The *30 Year Plan for Greater Adelaide* indicated areas where such development may occur in future. It is recommended that further research quantify the impact of these localised impervious areas on downstream drainage capacity, and explore options for WSUD techniques which can reduce or, if possible, ameliorate the impact. For example, large site based or precinct scale rainwater tanks, infiltration measures or detention tanks may

provide benefits which mitigate flow rates in a manner which does not compromise downstream drainage system capacity.

- The current project used modelling to simulate the impact of increased development in areas subject to infill development. It is recommended that the findings of this project be verified where possible with observed flow data. A verification of the modelling results may be possible using newly available flow data from the Frederick Street catchment for which monitoring was reinstated in 2013/2014.

11.6 Case Study 6 – Flagstaff Pines Catchment – Greenfield Development

Section 8 considered the impact of greenfield development on the flow regime in a natural stream, and the potential for WSUD to restore flows to pre-infill conditions. The study was undertaken by assessing the flow regime in the stream prior to development, following development, and by assuming WSUD was included as part of the greenfield development. Development was shown to cause changes to the pre-development flow regime in the creek with increases in the total annual flow and peak flow rates, and alterations in the flow duration curve. The inclusion of retention or detention on all homes constructed was ineffective at maintaining the pre-development flow regime in the creek. Street scale bioretention was also unable to restore pre-development flows.

<u>Recommendations</u>

- The implemented of retention, detention or rain gardens were not effective at maintaining the flow regime of the natural stream immediately downstream of the development. However, these on site and street scale measures were only tested in isolation, and the limited effectiveness may be improved by combined measures.

<u>Future research</u>

- Large scale downstream measures, such as detention basins, wetlands or larger biofilters were not considered in this analysis for their contribution to flow management. The ability of these measures to improve downstream flow regimes (as opposed to peak flow rates at specific intervals) should be assessed.
- In the planning of this analysis, it became apparent that there was little knowledge available on the nature of flows in small undeveloped catchments producing intermittent streams in the Adelaide region. Since current stormwater drainage design for green field development hinges on the preservation of pre-development peak flow rates, is it considered important that monitoring is undertaken to explore the pre-development flow regime in small undeveloped catchments in Adelaide. Such information will be valuable to verify that design tools are being used appropriately to estimate with some confidence the baseline pre-development flow regime for which a stormwater design must refer.

11.7 Application of Optimisation Tools to select WSUD

Section 9 explored the use of the optimisation tools to select WSUD strategies. Five subcatchments of the Paddocks catchment in Para Hills were selected for a case study site to which the current and a future infill development scenario were compared. The USEPA SUSTAIN model was used to identify the most cost effective arrangement of on-site retention or detention to maintain pre-infill development peak flow rates. SUSTAIN successfully produced a runoff time series from the urbanised catchment pre- and post-infill development, and provided optimal solutions for the distribution of retention and detention based scenarios in some circumstances. Recommendations were not consistent however, with retention or detention recommended in various arrangements when identical optimisation runs were repeated.

Recommendations

- SUSTAIN was found to produce outcomes which achieved the desired goal indicating it has some potential for optimising WSUD strategies to preserve peak flow rates.
- Users should be aware of issues identified with the SUSTAIN model, detailed in Section 9.5.

Future Research

- The model did not provide a consistent pattern in the placement of detention and retention tanks. This may be considered a natural outcome from the optimisation algorithm, but the absence of a general recommendation for tank placement may be a result of selecting a catchment that was too small for flow routing to begin to take effect. It is therefore recommended that the SUSTAIN model is trialled on larger catchment areas to explore the placement of retention and detention measures.
- This study did not consider the water quality prediction and WSUD placement optimisation tools included in the SUSTAIN software. There is therefore an opportunity to further explore the applicability of SUSTAIN for water quality management in South Australia.

11.8 Application of the MUSIC model to Urban Catchments in South Australia

In Section 10, the ability of MUSIC to estimate peak flow rates and runoff volumes of urban areas in Adelaide was explored. The study applied the model to identify suitable parameters for South Australian urban catchments based on the known parameters of the Frederick Street and Paddocks catchments. The results indicated that MUSIC provided a good estimate of the flow volume and peak flows with input parameters derived from calibration. Using the default parameters provided in MUSIC revealed an error in flow volume estimation in the order of 70%.

Recommendations

- MUSIC was able to produce accurate estimates of runoff volume and peak flows when calibration of model parameters was undertaken.

- There was variation in the parameters required to produce a calibrated MUSIC model for the Frederick Street and Paddocks catchments. It may therefore be necessary to produce guidance specific to zones across Adelaide and South Australia.

Future Research

- The lack of gauged catchments in urban and rural SA produces difficulty for practioners to produce calibrated models and estimate parameters in ungauged catchments. As well as DRAINS and SWMM based models, MUSIC and SOURCE are becoming popular tools for hydrological assessment and conceptual design of WSUD systems. However, there is little guidance on the application of these software tools for ungauged systems in South Australia. It is recommended that further research is undertaken to produce guidance for the selection of appropriate modelling parameters in ungauged catchments.

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13 List of Appendices

Appendix A	Fit of the observed and simulated data to calibration and verification events in Frederick Street
Appendix B	Fit of the observed and simulated data to calibration and verification events in the Paddocks
Appendix C	Calibration statistics for events used in the calibration of MUSIC

Appendix A

The following charts show the fit of the observed to the simulated data in the calibration and verification of the Frederick Street runoff model.

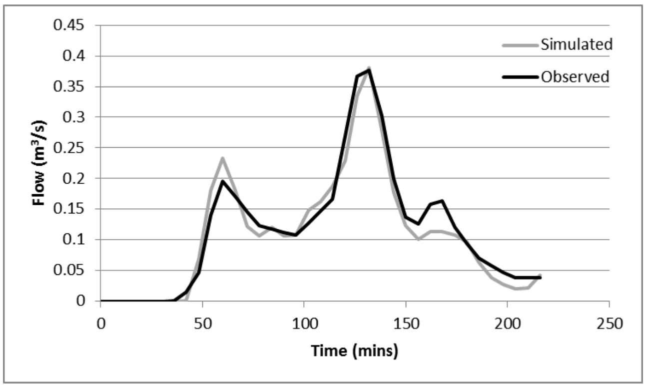


Figure A 1 – Comparison of the observed and simulated data for Frederick Street calibration event Cal1

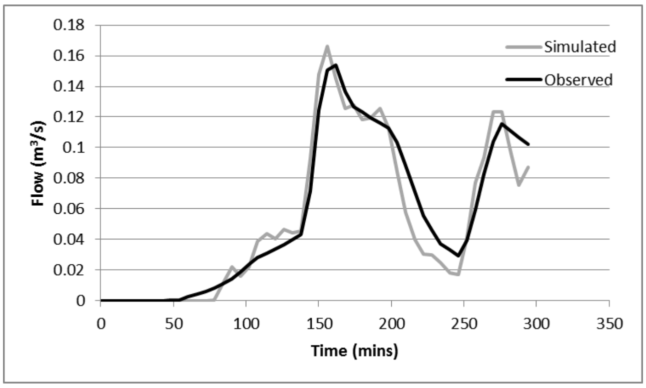


Figure A 2 – Comparison of the observed and simulated data for Frederick Street calibration event Cal2

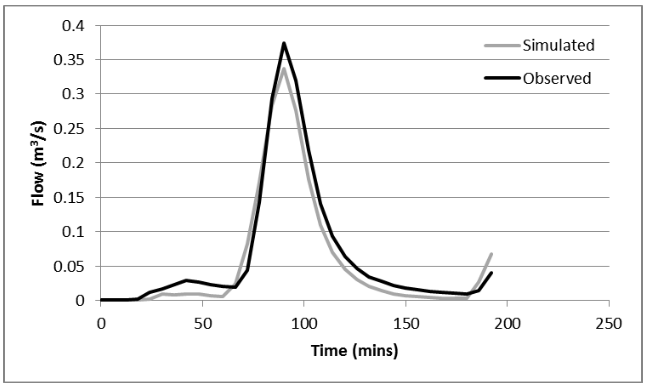


Figure A 3 – Comparison of the observed and simulated data for Frederick Street calibration event Cal3

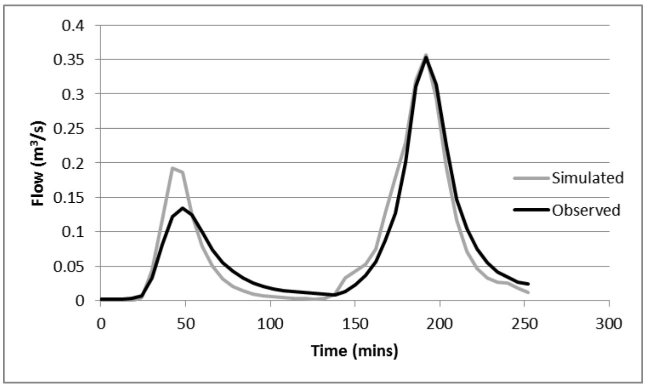


Figure A 4 – Comparison of the observed and simulated data for Frederick Street calibration event Cal4

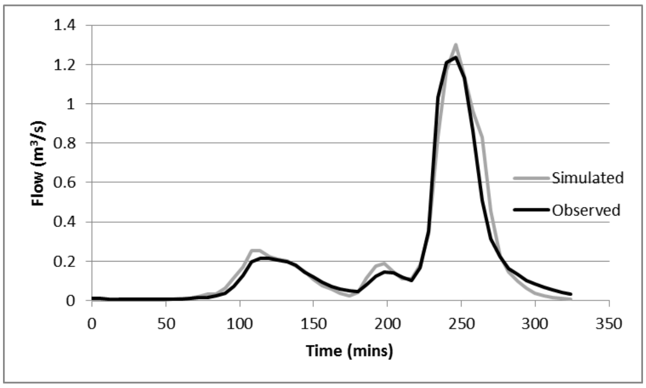


Figure A 5 – Comparison of the observed and simulated data for Frederick Street calibration event Cal5

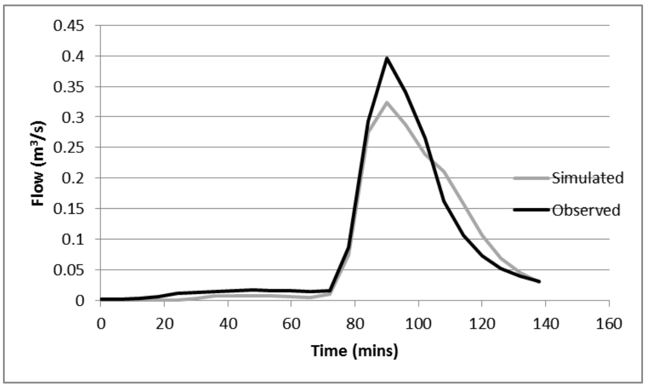


Figure A 6 – Comparison of the observed and simulated data for Frederick Street calibration event Cal6

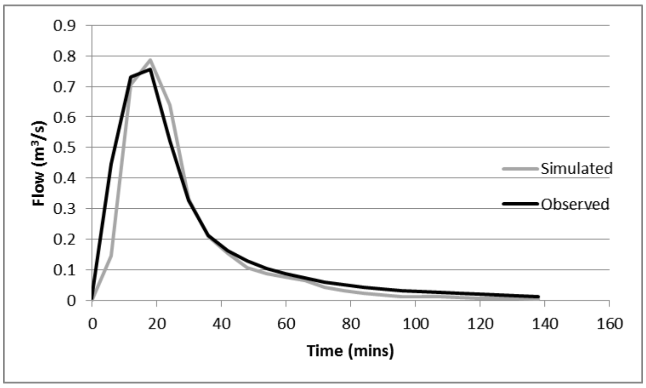


Figure A 7 – Comparison of the observed and simulated data for Frederick Street calibration event Cal7

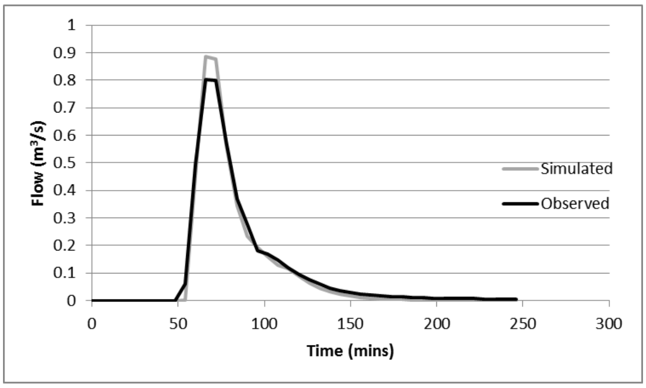


Figure A 8 – Comparison of the observed and simulated data for Frederick Street verification event V1

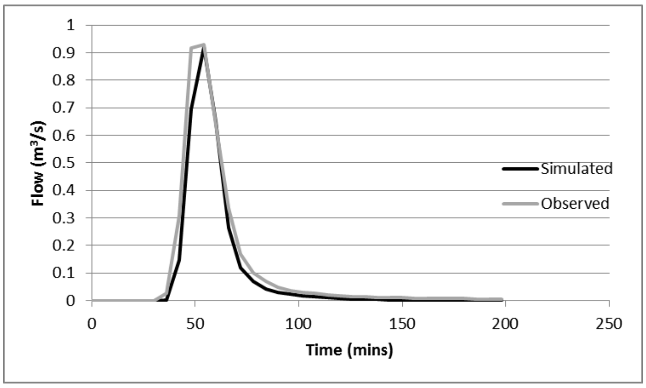


Figure A 9 – Comparison of the observed and simulated data for Frederick Street verification event V2

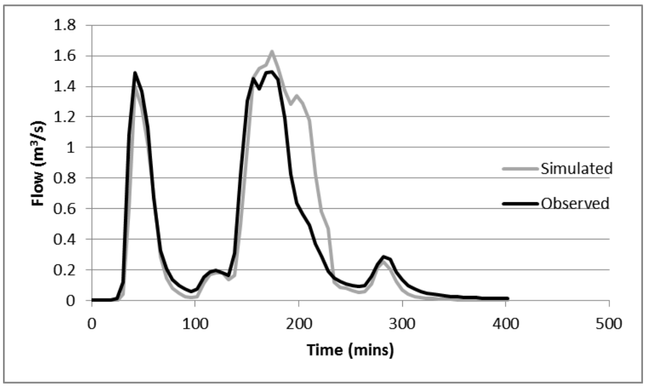


Figure A 10 – Comparison of the observed and simulated data for Frederick Street verification event V3

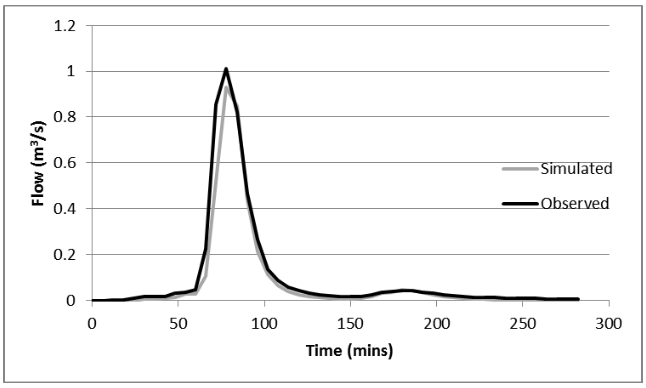


Figure A 11 – Comparison of the observed and simulated data for Frederick Street verification event V4

Appendix B

The following charts show the fit of the observed to the simulated data in the calibration and verification of the Paddocks runoff model.

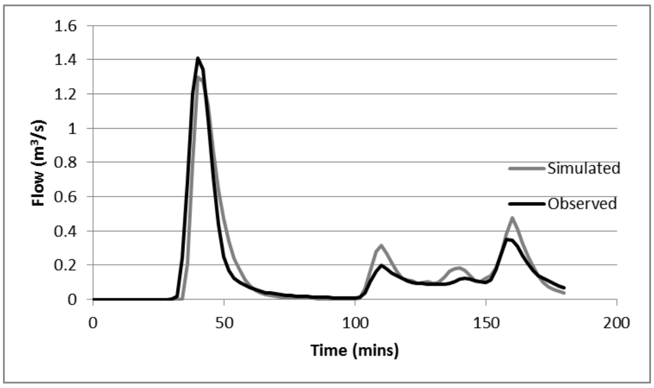


Figure B 1 – Comparison of the observed and simulated data for Paddocks calibration event Cal1

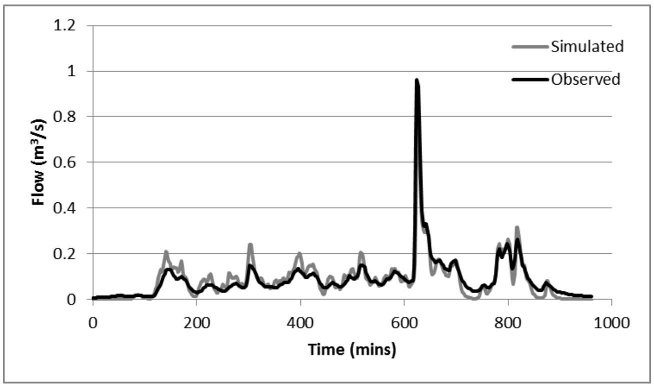


Figure B 2 – Comparison of the observed and simulated data for Paddocks calibration event Cal2

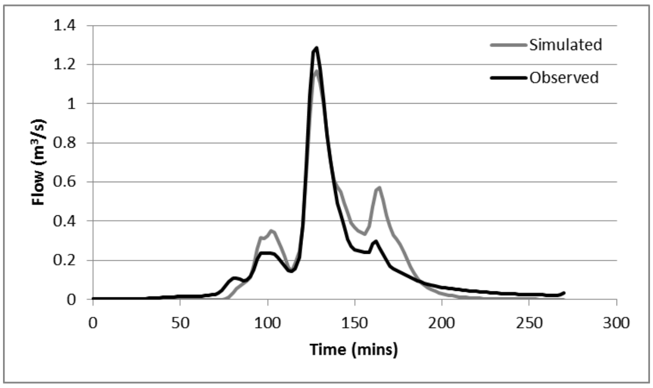


Figure B 3 – Comparison of the observed and simulated data for Paddocks calibration event Cal3

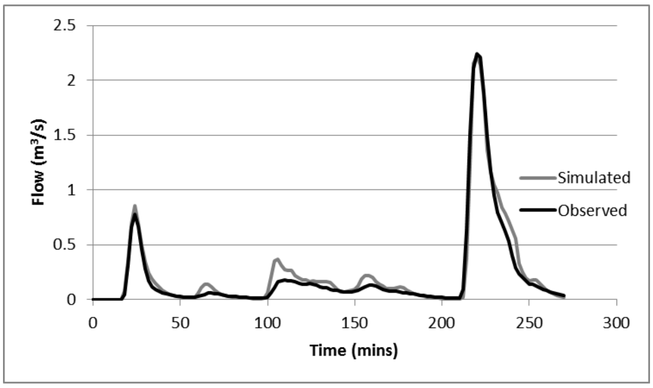


Figure B 4 – Comparison of the observed and simulated data for Paddocks calibration event Cal4

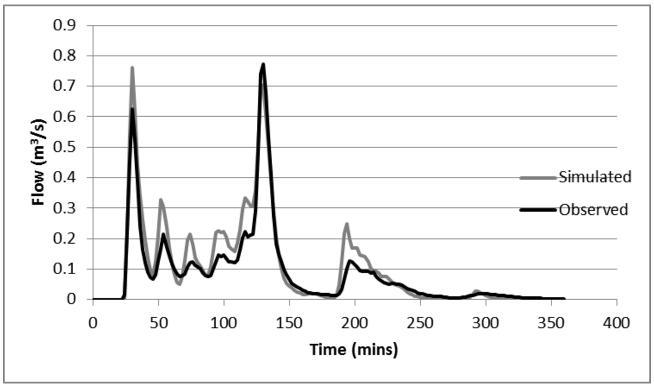


Figure B 5 – Comparison of the observed and simulated data for Paddocks calibration event Cal5

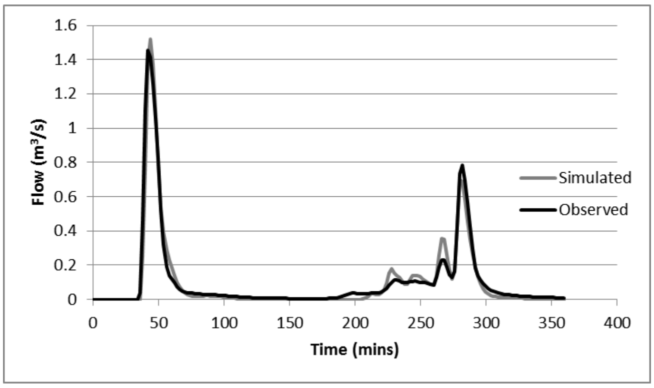


Figure B 6 – Comparison of the observed and simulated data for Paddocks calibration event Cal6

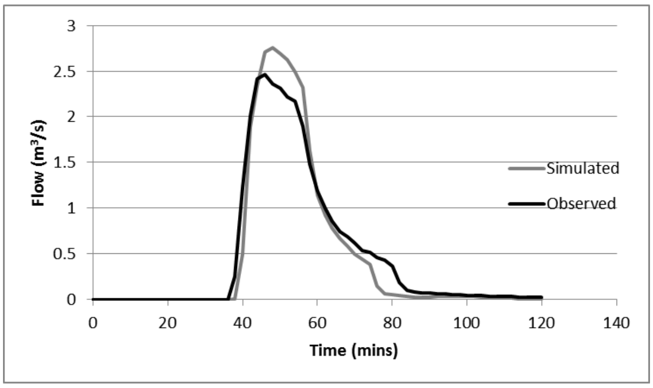


Figure B 7 – Comparison of the observed and simulated data for Paddocks calibration event Cal7

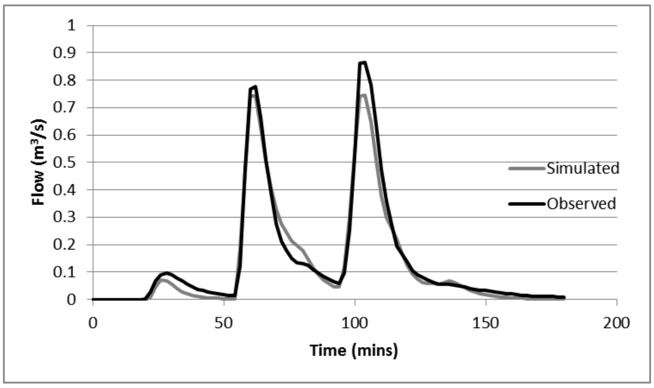


Figure B 8 – Comparison of the observed and simulated data for Paddocks calibration event Cal8

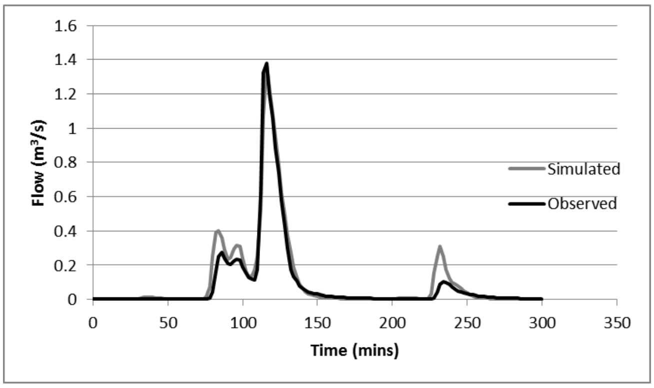


Figure B 9 – Comparison of the observed and simulated data for Paddocks calibration event Cal9

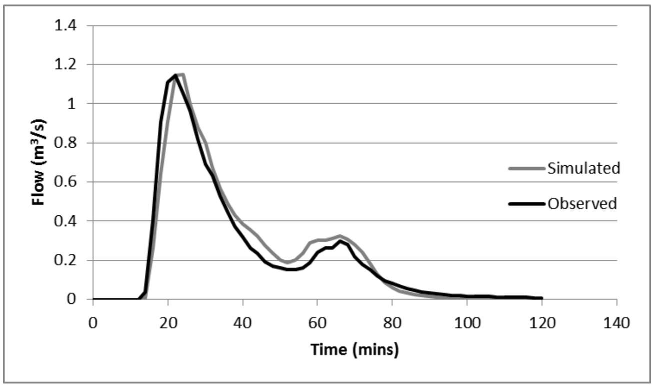


Figure B 10 – Comparison of the observed and simulated data for Paddocks calibration event Cal10

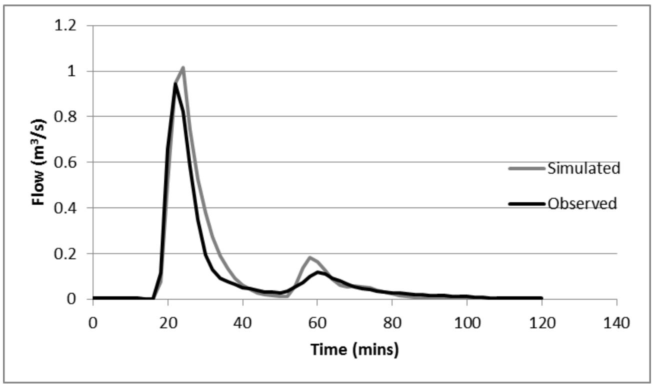


Figure B 11 – Comparison of the observed and simulated data for Paddocks calibration event Cal11

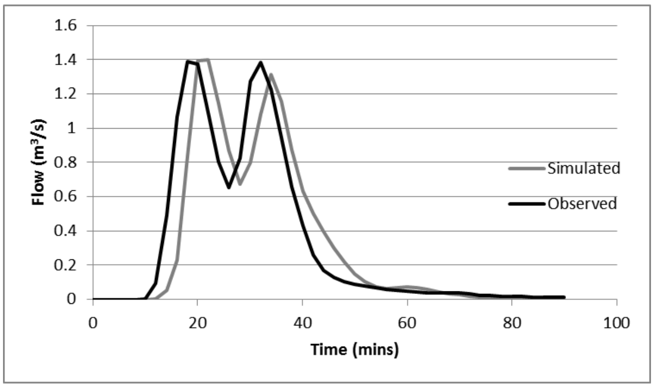


Figure B 12 – Comparison of the observed and simulated data for Paddocks calibration event Cal12

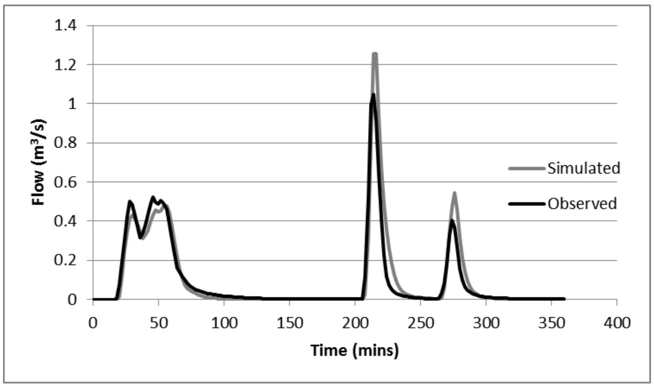


Figure B 13 – Comparison of the observed and simulated data for Paddocks calibration event Cal13

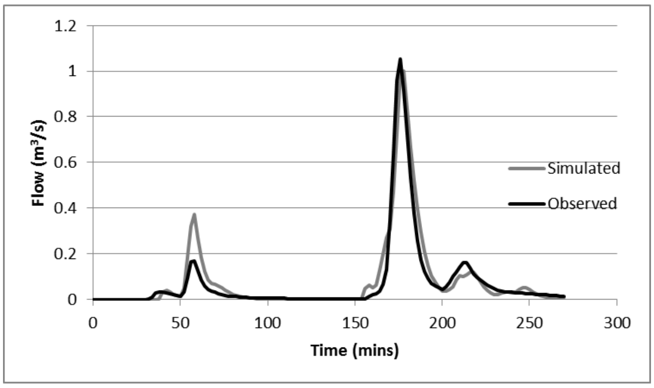


Figure B 14 – Comparison of the observed and simulated data for Paddocks calibration event Cal14

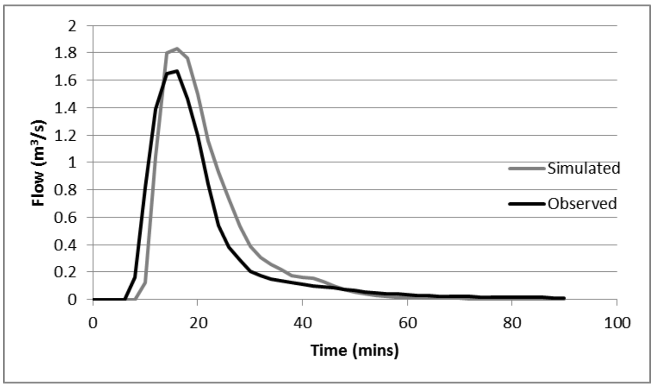


Figure B 15 – Comparison of the observed and simulated data for Paddocks calibration event Cal15

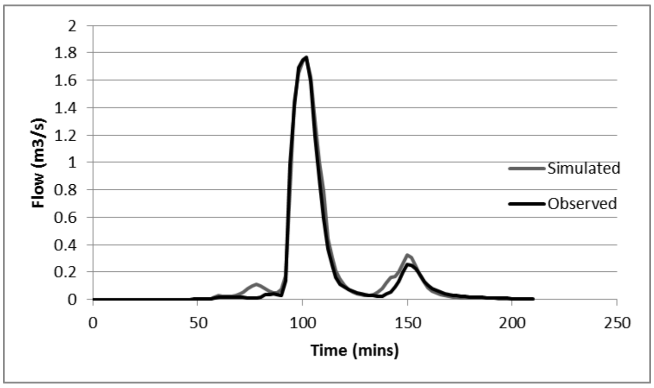


Figure B 16 – Comparison of the observed and simulated data for Paddocks verification event V1

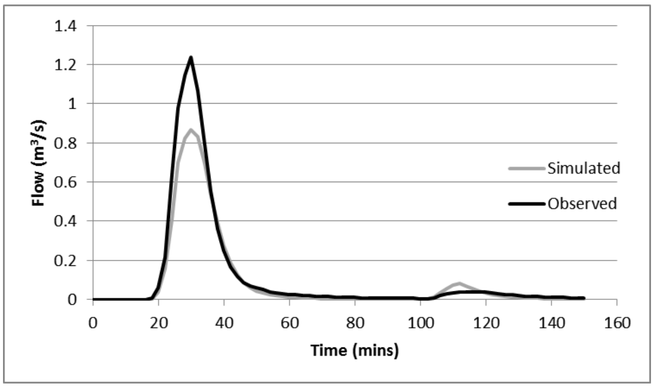


Figure B 17 – Comparison of the observed and simulated data for Paddocks verification event V2

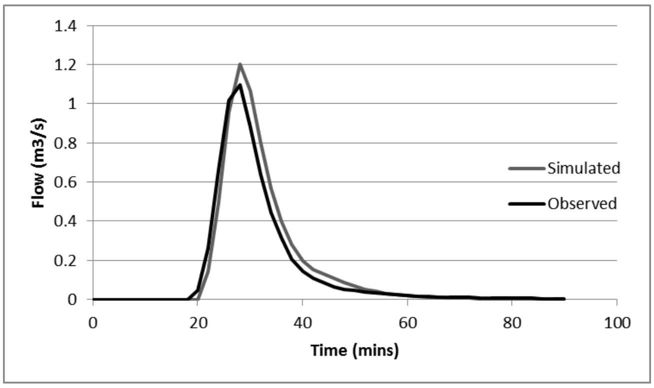


Figure B 18 – Comparison of the observed and simulated data for Paddocks verification event V3

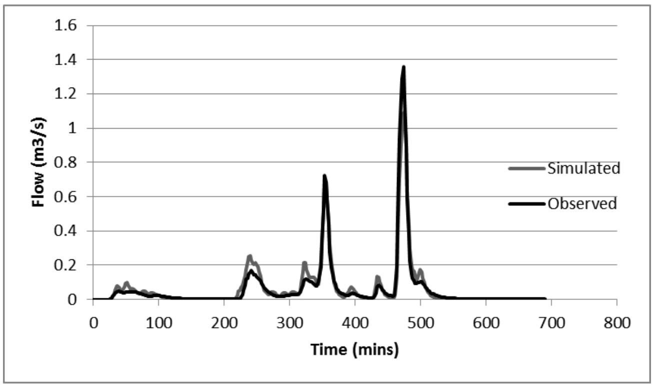


Figure B 19 – Comparison of the observed and simulated data for Paddocks verification event V4

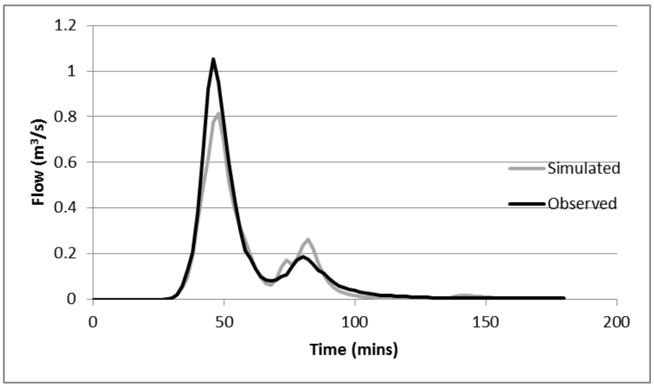


Figure B 20 – Comparison of the observed and simulated data for Paddocks verification event V5

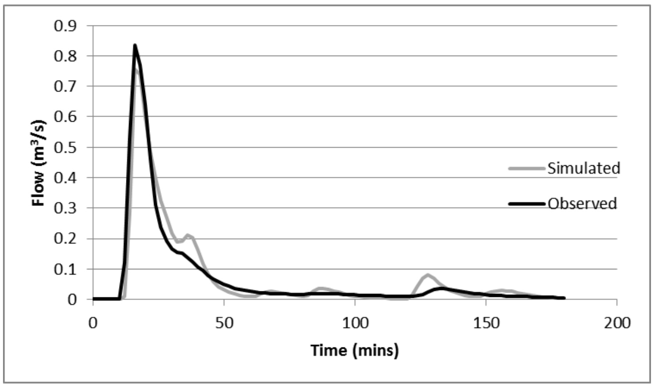


Figure B 21 – Comparison of the observed and simulated data for Paddocks verification event V6

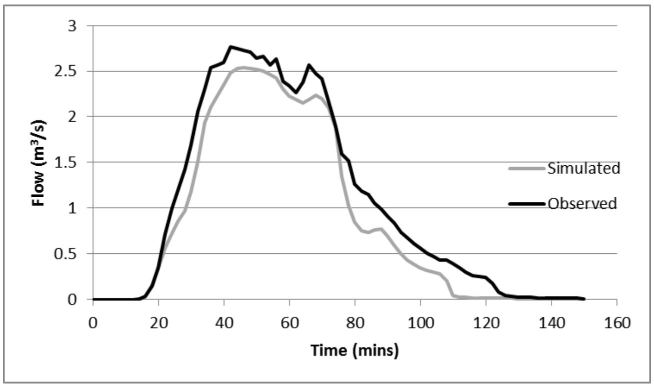


Figure B 22 – Comparison of the observed and simulated data for Paddocks verification event V7

Appendix C

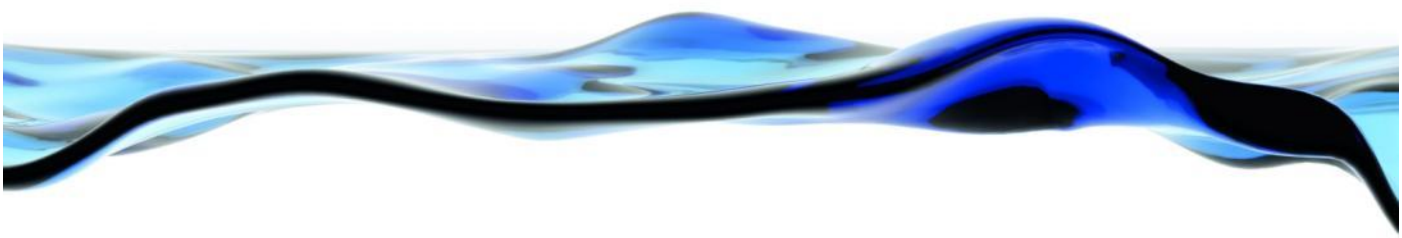
The calibration statistics in Tables C 1 and C 2.

Table C 1 – Summary of calibration statistics for the Paddocks catchment

Events	Date	Time	Modelled peak	Observed peak	PEP	PEV	R^2
1	3/10/1992	1600 to 2100	1.42	1.11	28.1	-3.9	0.85
2	8/10/1992	0200 to 1800	0.64	0.90	-28.8	-7.6	0.72
3	8/10/1992	1930 to 0000	0.96	1.24	-22.8	-30.6	0.82
4	17/11/1992	1130 to 1600	2.10	1.95	7.9	4.6	0.93
5	20/11/1992	2200 to 0400	0.69	0.67	3.0	20.4	0.67
6	18/12/1992	1600 to 2200	2.01	1.37	46.5	-1.0	0.73
7	19/12/1992	1300 to 1500	3.84	2.30	67.3	-14.3	0.52
8	27/02/1993	2200 to 0100	0.78	0.84	-7.1	-2.4	0.79
9	21/05/1993	1200 to 1700	1.14	1.30	-12.0	5.5	0.87
10	3/06/1993	1630 to 1830	1.28	1.05	21.7	-3.4	0.87
11	11/06/1993	1400 to 1600	0.87	0.59	48.6	-23.6	-1.38
12	30/08/1993	1700 to 1830	1.19	1.29	-8.1	-13.1	0.80
13	17/10/1993	0800 to 1400	1.05	0.85	23.5	-4.9	0.86
14	18/10/1993	0600 to 1100	0.96	0.98	-1.5	41.7	-0.41
15	13/12/1993	2230 to 0000	2.15	1.57	37.0	7.8	0.83
16	14/12/1993	0000 to 0400	2.38	1.76	35.5	-5.9	0.87

Table C 2 – Summary of calibration statistics for the Frederick Street catchment

Events	Date	Time	Modelled peak	Observed peak	PEP	PEV	R^2
1	3/07/1992	2340 to 0300	0.25	0.38	-33.1	-7.5	0.79
2	11/07/1992	0324 to 0818	0.14	0.15	-9.1	1.7	0.88
3	19/07/1992	0418 to 0724	0.21	0.37	-43.2	-14.1	0.72
4	7/08/1992	1542 to 1946	0.24	0.35	-30.9	-10.2	0.82
5	30/08/1992	0106 to 0630	2.36	1.24	90.8	145.3	-1.28
6	31/08/1992	1248 to 1442	0.77	0.40	94.3	217.2	-2.89
7	18/12/1992	1642 to 0012	1.58	1.34	18.6	36.7	0.67
8	24/05/1993	0200 to 0430	0.23	0.39	-40.5	-14.7	0.78
9	30/08/1993	1640 to 1900	0.51	0.80	-36.7	-9.9	0.77
10	19/09/1993	1100 to 1406	0.48	0.93	-48.0	-14.4	0.71
11	30/09/1993	1930 to 2218	0.19	0.38	-49.6	-16.2	0.68
12	17/10/1993	0748 to 1048	0.41	0.66	-38.0	-11.5	0.83



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