University of South Australia

Division of IT, Engineering and the Environment

Educating Professionals • Creating and Applying Knowledge • Serving the Community

The impact of infill development and WSUD measures on minor drainage system performance

Australian Flow Management Group, School of Natural and Built Environments

University of South Australia

Baden Myers

David Pezzaniti

David Kemp

Important Notice

Version History

Version	Description	Date	Author(s)	Approved by
Draft 1	Internal draft	20/11/2017	BM, DP, DK	
FinalRev	Final report for SMA comment	18/12/2017	BM, DP, DK	DP
Final	Final report to SMA	15/02/2018	BM, DP, DK	DP

Acknowledgements

The authors acknowledge the financial support of the following organisations to undertake this research:

- The South Australian Government via the Stormwater Management Authority and the Adelaide and Mount Lofty Ranges Natural Resource Management Board
- City of West Torrens
- City of Marion
- City of Salisbury
- City of Port Adelaide Enfield
- City of Charles Sturt
- City of Holdfast Bay
- City of Onkaparinga

The authors would also like to thank Professor John Argue and Mr Christopher Wright for their assistance during the project development, including assistance with refining methodology and assumptions and through peer review of the final draft.

Citation

It is suggested that this report be cited as follows:

Myers, B, Pezzaniti, D & Kemp, D 2017, The impact of infill development and WSUD solutions on minor drainage system performance - Australian Flow Management Group, University of South Australia, Adelaide, SA, Australia.

Summary

Key Findings

A study has been undertaken to investigate the role of WSUD in maintaining minor system drainage capacity and standard in the case where infill development is being experienced in a residential catchment. The study focussed on a residential catchment in the western suburbs of Adelaide, South Australia where significant infill development is occurring and extensive hydrological field monitoring data exists. Existing research indicated that between 1993 and 2013, the capacity of the minor system reduced from a 2.5 to a 2.2 year ARI due to infill development in this catchment. Using the monitoring data a calibrated hydrological model was developed and used to assess the hydrological effect of several WSUD scenarios including retention and detention based measures. Analysis was also undertaken to assess the appropriateness of the design storm approach for assessing the hydrological performance of WSUD measures. A basic cost comparison was carried out to compare a conventional drainage system upgrade and alternative WSUD options.

The main findings of the study were:

- Between 1993 and 2013 the catchment studied experienced a 12.5% increase in total impervious area due to allotment subdivision and building extensions on existing allotments. Considering allotment areas only (excluding road area), impervious area increased by 17%. A total of 69% of this increase was attributed to infill development and the remaining 31% was attributed to home improvement activity.
- Analysis of observed flow data indicated a 10% increase in runoff volume due to this level of infill. Use of a calibrated simulation tool indicated that peak flow rates increased by approximately 16% for the events considered.
- By projecting infill to a point where 1 in 2 allotments were subdivided in the 1993 baseline case, it was demonstrated that mean annual runoff volume increased by 54.5%. The peak flow for events up to a 50% AEP (or 2 year ARI) increased by 35%.
- The ability of WSUD in the form of retention or detention storages (between 1 to 10 kL per redeveloped lot) to mitigate increased runoff and peak flows rates varied. Performance was influenced by storage size, emptying rate and connectivity to upstream impervious area.
- As an example, in the retention case, a 5 kL storage, with an emptying rate of 1000 L/day (5 day emptying time) connected to allotment roofs can reduce mean annual runoff by 60% of that required to restore the original 1993 mean annual runoff volume – however connecting the storages to all impervious area increases this performance to 83%. This scenario reduced the peak flow rate of a 50% AEP (2 year ARI) event by 25% of that required to restore the equivalent peak flow rate in 1993.
- In an equivalent detention case using 5 kL tanks with a 20 mm orifice connected to roofs only, (approximately 3 hours emptying time) the peak flow rate of a 50% AEP (2 year ARI) event was reduced by 72% of the value required to restore the equivalent peak flow rate in 1993. There was however no impact on mean annual runoff using detention storage.
- Retention system performance required rapid emptying times (less than one day) to have strong impact on peak flow rates. Detention systems typically performed best with an orifice of 20 to 30 mm for this case study catchment.

- There was no notable impact of catchment slope on WSUD storage effectiveness in the case study catchment.
- WSUD storages have greater impact on the runoff volume, peak flow rate and flooding for more frequent storms (less than 50% AEP) compared to less frequent events (20% AEP or greater).
- The suitability of using the design storm approach for assessing the hydrological performance of storage based WSUD measures is highly influenced by the pre-event storage volume and emptying rate. Examination of typical rainwater tank storage using a 100 year time series of continuous data indicated that the average available storage volume prior to 20% AEP events varied from 10% (for storages emptying at 100 L/day) to 70% (for storages emptying at 500 L/day) of the total volume.
- Restricting the size of the connected impervious area to a WSUD storage (i.e. to house roofs only) limited the performance for volume and peak flow mitigation in the catchment studied.
- The cost estimate of allotment scale WSUD options were generally comparable to or cheaper than a ‘conventional’ pipe duplication project for this case study catchment.

Introduction and Background

Councils across the greater Adelaide metropolitan region are experiencing residential infill development. In most cases the infill results in additional load on existing drainage infrastructure. This lowers the standard of flood protection provided by existing drainage infrastructure, in particular the ‘minor’ system which represents a significant proportion drainage infrastructure expenditure. Traditionally, the approach to dealing with infill development has been the use of detention or retention (including WSUD) measures, or a drainage system upgrade. However, there is little information available regarding the effectiveness of these WSUD measures to ameliorate the impact of infill development on drainage system capacity. There is also limited information on the cost of implementing detention or retention based WSUD measures in comparison to the cost of a conventional drainage system upgrade. Furthermore, the technical justification of detention and retention based WSUD measures and conventional drainage system upgrades is commonly assessed using design storm approaches. A shortcoming of the design storm approach is that it is based on a storm ‘burst’ and no consideration is given to rainfall preceding the design storm. Based on these identified problems, three objectives were proposed for this project:

1. Collect evidence to quantify the impact of infill development on runoff flow rates and volumes in an urban catchment.
2. Identify and present a robust approach to compare the technical justification and cost of maintaining drainage system capacity using either a technically justified WSUD based policy measure or a conventional drainage system upgrade.
3. Compare the effectiveness of the ‘design storm’ approach for assessing WSUD based systems with storage with a continuous simulation approach.

Quantifying the Impact of Infill Development

The impact of infill development was examined using gauged rainfall and flow records for the Frederick Street catchment, City of Marion, in 1992/1993 and 2013/2014. The analysis also used examination of aerial photography from 1966, 1993 and 2013. Infill development was shown to have significant impact on the catchment and the flow regime. Between 1966 and 2013 (47 years), the total pervious area decreased from 61.5% to 39.9% of the area, a 35% reduction attributable to the progress of infill and additions to existing homes. Over the same period, the total catchment area covered by home roofing increased from 13.6% to 24.9%, an 83% increase between 1966 and 2013.

Analysis of observed flow data collected in 1992/1993 and 2013/2014 indicated that over this 20 year period, the volumetric runoff coefficient of the catchment increased from 24.4% to 31.2%, producing a 10% increase in runoff volume per event. This equates closely to the 12.3% increase in imperviousness between 1993 and 2013. An event runoff model, ILSAX, was then used to explore the impact of the increased imperviousness on peak flow rates, and it was found that the directly connected impervious area calibration parameter had to be increased by +15% to meet the 2013/14 peak flow rates.

The progress of infill development is expected to continue in light of the state government shift in focus from developing the urban fringe to denser urban living. The evidence presented in this report, in addition to the impacts of existing development policy in urban areas, emphasise the need to consider the adoption of tools or policy to reduce the impact of infill development on peak flow rate and runoff volume. Alternately, catchment managers and the broader community must accept the consequences of increasing runoff volumes to receiving waters, and the ongoing reduced capacity and standard of the minor drainage system where infill occurs.

A technical and economic assessment of the effectiveness of WSUD

The projected impact of ongoing urban infill and WSUD on runoff volumes, peak flow and flooding in in the Adelaide metropolitan area was investigated using a calibrated hydraulic modelling tool capable of continuous simulation. Continuous simulation scenarios were run for several catchment scenarios, including calibrated catchment models for Frederick Street, City of Marion and Paddocks, City of Salisbury. Conceptual models of a home allotment and residential street were also employed. Modelling was conducted to explore six problems associated with urban infill and WSUD performance as follows:

- Ongoing infill in a residential catchment
- Implementation of WSUD with infill development, in the form of retention or detention systems at the allotment and street (lumped) scale
- Implementation of this same WSUD, but comparing a flat and sloped catchment response
- Implementation of this same WSUD, but comparing higher and lower intensity storm events
- Implementation of this same WSUD, but comparing catchments with higher and lower infiltration rate
- Implementation of this same WSUD, but comparing the results of assessment at site, street and catchment scale

Each question was addressed in terms of mean annual runoff volume, peak flow rate and in some cases flood (or ponding) volume. Peak flow rate and flood volume were assessed for storm events in the outflow time series which corresponded with estimates for the 86%, 63%, 50%, 20% and 11% annual exceedance potential (AEP). AEP was selected as the base terminology for storm frequency in this report, and a comparison table to compare AEP with the prior terminology average recurrence interval (ARI) has been provided (see Table 1, Page 15). A summary of this for the events considered is shown in the Table A.

Table A: Conversion of ARI and AEP for common storm frequencies

Annual Exceedance Probability (AEP)	Average Recurrence Interval (ARI)
86%	0.5 year
63%	1 year
50%	2 year
20%	5 year
11%	10 year

Effects of Infill

The effects of infill were explored by comparing the Frederick Street catchment in 1993 with a projected scenario where infill development was assumed to occur on half of the allotments greater than 500 m^2 (178 of 358 allotments) with no WSUD. The results indicated that the mean annual runoff volume increased by 54.5% due to this infill. The peak flow rate of combined surface and pipe flow at the catchment outlet increased, and the extent was greater for more frequent events. For example, the peak flow increased by approximately 35% for the 86% AEP, 63% AEP and 50% AEP storms, but by only 18% for the 20% AEP storm and by 6% for the 11% AEP storm. The volume of flooding at a selected assessment point also increased due to infill. The volume of flooding increased for all AEPs assessed by a factor of 1.5 to 3 but the response was not proportional to event magnitude. This may be due to a number of reasons including changes to the volume of surface (flood) storage during the event and impervious area and indirectly connected impervious area runoff contributions.

The Impact of WSUD on Projected Runoff Following Infill

The impact that WSUD can have on the projected runoff with infill development considered the impact of retention and detention based WSUD placed at the allotment and street scale. The installation of on-site retention (e.g. rainwater tanks or infiltration systems) could help maintain the mean annual runoff volume, but performance was a function of tank size and anticipated reuse. As an example, the installation of 5 kL retention with 100 L/d demand on every new home in the redeveloped catchment could only go 33% of the way to maintaining the mean annual runoff volume of the catchment prior to redevelopment. The installation of retention in a lumped manner at the street scale was much more effective at reducing the mean annual runoff volume compared to the allotment scale. However, daily demand or infiltration was required to be up to 2000 L/day to maintain runoff volume, with a storage size of at least 5 kL per upstream redevelopment (or 2.5 kL per home). The peak flow rate of the catchment could not be maintained to within 10% of the 1993 baseline scenario using allotment based retention. The installation of 5 kL retention on every new home in the redeveloped scenario with 100 L/day reuse could only reduce the peak flow rate of a 50% AEP storm by one quarter of that required to maintain the 1993 flow rate. When retention was installed in a lumped manner, performance was better than at the allotment scale, however to fully maintain the original peak flow rate required the highest levels of assumed reuse/infiltration capacity (50 000 L/d).

The peak flow rate of the catchment could not be maintained by allotment based detention, but the overall performance was better than retention; for example, the use of 5 kL detention on every new home draining by gravity through a 20 mm orifice plate could reduce the peak flow rate of a 50% AEP storm by 80% of that required to maintain the 1993 flow rates. When detention storage was installed in an equivalent lumped system at street scale, performance was again better than retention; 6 kL per redeveloped allotment with an emptying time equal to that of allotment based detention tanks with a 30 mm orifice were able to restore peak flow rates in this scenario.

The key finding in this aspect of the research is that detention typically out performed retention for peak flow rate management. This is attributed to the increased availability of storage in detention based systems prior to storm events (bursts) occurring. Both retention and detention performed better when lumped at the street scale compared to the allotment scale. This is because the lumped scenarios considered tanks with larger connected impervious areas than assumed at the allotment.

Comparing the Impact of WSUD on Flat and Moderate Grade Catchments

The impact of catchment slope on the effectiveness of WSUD for maintaining mean annual runoff volume and peak flow rates following infill development was examined by comparing the effectiveness of selected retention and detention scenarios on a flat catchment (Frederick Street) and on a moderately sloped catchment (the Paddocks, City of Salisbury). When equivalent amounts of infill were assumed, the increase in mean annual runoff volume was almost identical for the flat and moderately sloped case. The impact of 5 kL retention tanks on each redeveloped allotment had a similar impact on this volume, and like the case of the flat catchment, lumped retention at the street scale performed better than allotment based measures.

The increase in peak flow rates (including runoff conveyed at the surface and pipe flow) following infill development was consistently larger on the moderately sloped catchment compared to the flat catchment. For example, the 50% AEP peak flow rate increased by 34% on the flat catchment compared to 39% on the moderate grade catchment; for the 20% AEP, these values were 17.7% (flat) and 37% (moderate grade). In both cases, the percentage increase was lower as the AEP reduced (became less frequent), which was attributable to the increasing influence of pervious area under high rainfall intensity masking the impact of increased impervious area. Despite the difference in peak flow rate increase, there was no clear difference in the effectiveness of identical retention or detention layouts on either the flat or sloped catchment to maintain pre-infill flow characteristics. To further explore this, it is recommended that more catchment case studies be undertaken to identify whether slope has an impact on the effectiveness of WSUD for much larger catchment areas.

Comparing the Impact of WSUD with Lower and Higher AEP Storms

This research showed that infill development increased the mean annual runoff volume and the runoff volume produced by individual events within the simulated time series. In both cases, the percentage increase in event runoff volume gradually diminished as the AEP of the event became less frequent. Similar findings were apparent for the individual event peak flow rates. For example, the peak flow rate of runoff increased by 36% for the more frequent 86% AEP event, but by only 6 % for the less frequent 11% AEP. Results also showed that the implementation of WSUD in the form of retention or detention had a greater impact on less frequent storm events. In other words, the events most affected by infill development are those which are more frequent, and these are also the events that can be more effectively managed using WSUD measures.

These findings were attributed to catchment runoff properties. Less frequent events (20% AEP or greater) can be influenced by pervious area runoff, and when pervious areas contribute to the total runoff volume, the impact of additional connected impervious area is less apparent. This is because runoff volume and peak flows are similarly influenced by impervious area and pervious area when the capacity to infiltrate is compromised by saturated soils or a high rainfall intensity. It is recommended that future research look more closely at the relationship between WSUD effectiveness, rainfall intensity and flood volume throughout a catchment. In this project, flooding was examined using the level of ponding at one point in the catchment. A better understanding may be achieved by investigating surface flooding throughout the catchment, for example by using a 2-dimensional modelling tool.

Comparing the Impact of WSUD on catchments with Higher and Lower Soil Infiltration Rate

A comparison of the impact of implementing WSUD on catchments with higher and lower infiltration rate was relevant only to retention based measures. As the assumed infiltration rate of soil increased (reflected by demand in L/d on this study) allotment based and lumped scale retention both became more effective at reducing runoff volume. Lumped retention measures represented a more effective means of restoring runoff volumes to the levels prior to redevelopment. Generally speaking, storages which could infiltrate greater than 2000 L/d and with a 5 kL storage volume per redeveloped allotment could maintain the mean annual runoff volume. As in the case of runoff volume, allotment based retention could not maintain the catchment peak flow rates in the extended range of assumed parameters of this project, although lumped retention measures were effective. However, unlike the runoff volume case, catchment peak flow rates could only be maintained with the very highest assumed demand (50 000 L/d) and for large storage volume (minimum 8 kL per redeveloped allotment in the contributing catchment area). Increasing infiltration rates also led to reduced flood volume at the street level, but neither the allotment based nor lumped retention systems, regardless of infiltration rate or storage volume, could maintain flood volumes to those in the 1993 scenario.

Comparing the Effect of Assessing WSUD Performance in Smaller to Larger Catchments

The effect of implementing and assessing WSUD in catchments of varying size was examined by comparing the runoff volume and peak flow rate following redevelopment of an allotment, a street of allotments and a full catchment (Frederick Street). The percentage increase in mean annual runoff volume due to infill reduced as the catchment area increased, but the estimated performance of WSUD to maintain the mean annual runoff volume was broadly similar at every scale. Similarly, the results indicated that the increase in peak flow rates for each selected AEP diminished as the model scale became larger. Also like runoff volume, the ability of WSUD to maintain peak flow rates to pre-infill development levels was similar at each catchment scale. Estimation of peak flow rates following the implementation of retention and detention indicated that WSUD was not clearly more or less beneficially assessed at the allotment, street or catchment scale. Relationships for retention remained very consistent. However the required orifice size (or emptying time) of detention systems changed according to the size of the catchment being considered where peak flow rates. For example, the best orifice size for a 5 kL detention tank on an allotment was 20 mm at the allotment and street scale, and 10 mm for the catchment scenario.

Comparing the Cost of WSUD at Site and Street Scale with a Catchment Drainage Upgrade

Cost effectiveness was assessed for five WSUD implementation scenarios, with the results shown in the table below. Costing was undertaken using literature and consultation with an estimator for home construction. It should be noted that the costings are for purchase and basic installation of infrastructure only and do not include costs attributed to ongoing maintenance, electricity use, design, construction management, site supervision or inspection, traffic control and service location/survey works. Costs are also independent of any potential savings for bulk purchase of items.

Name	**Description**	**Costs ($ in 2016)**		
Option 1	Stormwater upgrade works – 'dual pipe' including 2 km pipe and 78 pits/junctions	*892,000		
WSUD Volume per redeveloped allotment		**2 kL**	**5 kL**	**10 kL**
Option 2	Allotment rainwater tanks (low use retention)	598,000	682,000	804,000
Option 3	Allotment infiltration via soakaways (low to medium use retention)	338,000	-	463,000
Option 4	Allotment detention tanks (on surface)	260,000	339,000	*469,000
Option 5	Lumped detention at pits (enlarged pipe sections fitted with an orifice)	360,000	*877,000	*1,740,000
'*' indicates that this measure was considered successful at maintaining peak flows and flooding at levels observed prior to redevelopment of the Frederick Street catchment				

Results indicated that allotment scale WSUD, which could contribute to but not fully maintain mean annual runoff, peak flow rates and catchment flooding to pre-infill development levels, was always cheaper than the cost of a conventional upgrade of the 'minor' system. The cheapest and most effective means to maintain peak flow rates was detention systems at the allotment. However it should be noted that detention makes no contribution to reducing runoff volume. The use of enlarged sections of concrete pipe at each stormwater pit to provide street scale detention storage was the most expensive means considered to minimise peak flows and flooding at the catchment scale. These results may be heavily influenced by the costing assumptions. Key among these assumptions was the exclusion of design, construction supervision and approvals costs. A further limitation on the study findings is that the requirements of detention are likely to vary based on the scale of the catchment considered.

The Use of Design Storm and Continuous Simulation to Simulate WSUD Effectiveness

The effectiveness of WSUD to maintain post-infill peak flow rates to pre-infill levels at the allotment and catchment scale was examined using a performance estimate based on a design storm simulation (where storage was assumed to be empty) and an estimate based on continuous simulation and subsequent partial series analysis of a simulated flow time series (which was the basis of all results in this report). Simulation was conducted assuming on-site measures only. The results indicated that each approach produced a significantly different performance estimate. For example, the assumption of an empty storage indicated that both retention and detention with a 4 kL volume could effectively maintain the 50% AEP peak flow rate for retention and detention cases. The continuous simulation however indicated that much larger storages were required in both cases to achieve best results and these were still not maintaining peak flow rates for the relatively frequent 50% AEP event. This is because the design storm simulation does not take into account the constant filling and emptying of storages. Based on this, further investigation was undertaken to identify is some prior capacity may be assumed as a compromise between the design and continuous simulation approaches.

The analysis was undertaken for using a long time series of rainfall recorded for Adelaide, combining records at West Terrace and Kent Town. The rainfall record was used to predict the condition of a rainwater tank of varying size fitted to 200 m^2 of impervious area with varying reuse conditions. Results showed that for all winter events the tank was near to or full prior to storm bursts with a 20% AEP (similar to the 5 year ARI). Analysis indicated that at least 12 months prior conditions were required to reach an equilibrium (where the assumed storage volume of the tank ceased to impact the ability of the tank to capture the 20% AEP storm). This observation suggests the adaption of a design pre-burst rainfall event with the design burst event is unlikely to be feasible, using design tools used by the profession (i.e. the DRAINS model). An alternative approach is to develop a relationship between storage, discharge and catchment area.

Contents

1. Introduction

The majority of councils across the greater Adelaide metropolitan region are experiencing residential infill development. In most cases, if not all, the infill results in additional load on existing drainage infrastructure. This has the effect of lowering the standard of flood protection provided by the existing drainage infrastructure, in particular the 'minor' system which represents a significant proportion of drainage infrastructure expenditure, and increasing annual discharge volumes. Opportunities to ameliorate the impacts are limited and WSUD control measures are seen as a potential option.

There are very little data available in published literature that quantifies the impact of infill development on existing drainage system capacity. With a push to increase the density of existing urban areas, a need exists to support policy (and regulation) development in this area. Should local governments need to alter the way a proposed infill development is assessed and approved with respect to stormwater management, there will be a need for appropriate evidence and supporting information, together with assessment and approval instruments. Deterministic models with dynamic hydraulic modelling capabilities provide an opportunity to understand the impact of infill development on the drainage system with a reasonable level of certainty, however observed data will provide the credible information to support future policy and/or regulations for infill development.

Traditionally, the approach to dealing with infill development has been the use of detention or retention (including WSUD) measures, or a drainage system upgrade. Throughout this report, detention is defined as any storage measure that drains to the conventional drainage system regardless of the time to empty, while retention is any measure that collects and holds/disposes of water via household reuse, infiltration or other means not including disposal to street drainage. However, there is little information available regarding the effectiveness of WSUD measures including detention or retention to ameliorate the impact of infill development on drainage system capacity, or on the cost of implementing detention or retention based WSUD measures in comparison to the cost of a conventional drainage system upgrade.

There is also a potential for inaccuracy in the design of urban drainage measures such as detention and retention. The technical justification of detention and retention based WSUD measures and conventional drainage system upgrades is commonly assessed using design storm approaches. A shortcoming of the design storm approach is that it is based on a storm 'burst' and no consideration is given to rainfall preceding the design storm. This is a significant omission when components of the drainage system incorporate storages that are integral to the drainage capacity – such as retention and detention systems. Generally the profession has dealt with this dilemma by assuming antecedent storage (less than full) conditions, but there is still uncertainty about what constitutes a reasonable or conservative approach. The significance becomes more important as continued infill leads to a greater impact on the existing urban drainage infrastructure and the level of flood protection to urban development diminishes.

2. Project Objectives

There were three objectives proposed for this project:

1. To collect evidence to quantify the impact of infill development on runoff flow rates and volumes in an urban catchment.

2. To identify and present a robust approach to compare the technical justification and cost of maintaining drainage system capacity using either a technically justified WSUD based measure or a conventional drainage system upgrade.
3. To compare the effectiveness of the 'design storm' approach for assessing WSUD based systems with storage with a continuous simulation approach.

3. The Consideration of Storm Frequency

Throughout this report, to reduce the number of terms used and in keeping with the newest iteration of the Australian rainfall and runoff guideline, the annual exceedance potential (AEP) is used to indicate storm frequency. The relationship between the AEP, the exceedance per year (EY) and other terms including the average recurrence interval (ARI) are shown in Table 1 below, acquired from the revised version of *Australian Rainfall and Runoff* Ball et al. (2016b). Note that the runoff and peak flow rate analysis that is presented in this report is generally for the 86%, 63%, 50%, 20% and 11% AEP, and therefore in the very frequent to frequent AEP range.

Table 1 – Relationship between EY, AEP and ARI (Ball et al., 2016b). The commonly used AEPs in this report are shaded grey.

Frequency description	**Exceedance per year (EY)**	**Annual Exceedance Probability (AEP) *(%)***	**Annual Exceedance Probability (AEP) (1 in *x* years)**	**Average recurrence interval (ARI)**
Very frequent	12			
	6	99.75	1.002	0.17
	4	98.17	1.02	0.25
	3	95.02	1.05	0.33
	2	86.47	1.16	0.5
	1	63.21	1.58	1
Frequent	0.69	50	2	1.44
	0.5	39.35	2.54	2
	0.22	20	5	4.48
	0.2	18.13	5.52	5
	0.11	10	10	9.49
Rare	0.05	5	20	20
	0.02	2	50	50
	0.01	1	100	100
Very rare	0.005	0.5	200	200
	0.002	0.2	500	500
	0.001	0.1	1000	1000
	0.0005	0.05	2000	2000
Extreme	0.0002	0.02	5000	5000
			…	
			Probable maximum precipitation (PMP) / probable maximum precipitation design flood (PMPDF)	

4. Methodology

4.1 Quantifying the Impact of Infill Development

The impact of infill development on urban runoff volume and peak flow rates was examined using aerial photography and field monitoring data derived from a case study catchment in Adelaide, South Australia. Approximately 20 years ago a residential catchment in the City of Marion and City of Holdfast Bay, the Frederick Street catchment (or part of 'Drain 18 of the South West Suburbs Drainage Scheme constructed in the 1970s) was extensively monitored. The monitoring program included three rainfall stations, two flow gauges and water quality sampling. The location of the catchment is presented in Figure 1 and the location of gauging is presented in Figure 1. Field assessments were also carried out to determine the level of connected impervious area. In the past 20 years the catchment has been noted to experience urban infill. The progress of this urban infill was using aerial photography from 1966, 1993 and 2013 (Section 4.1.1).

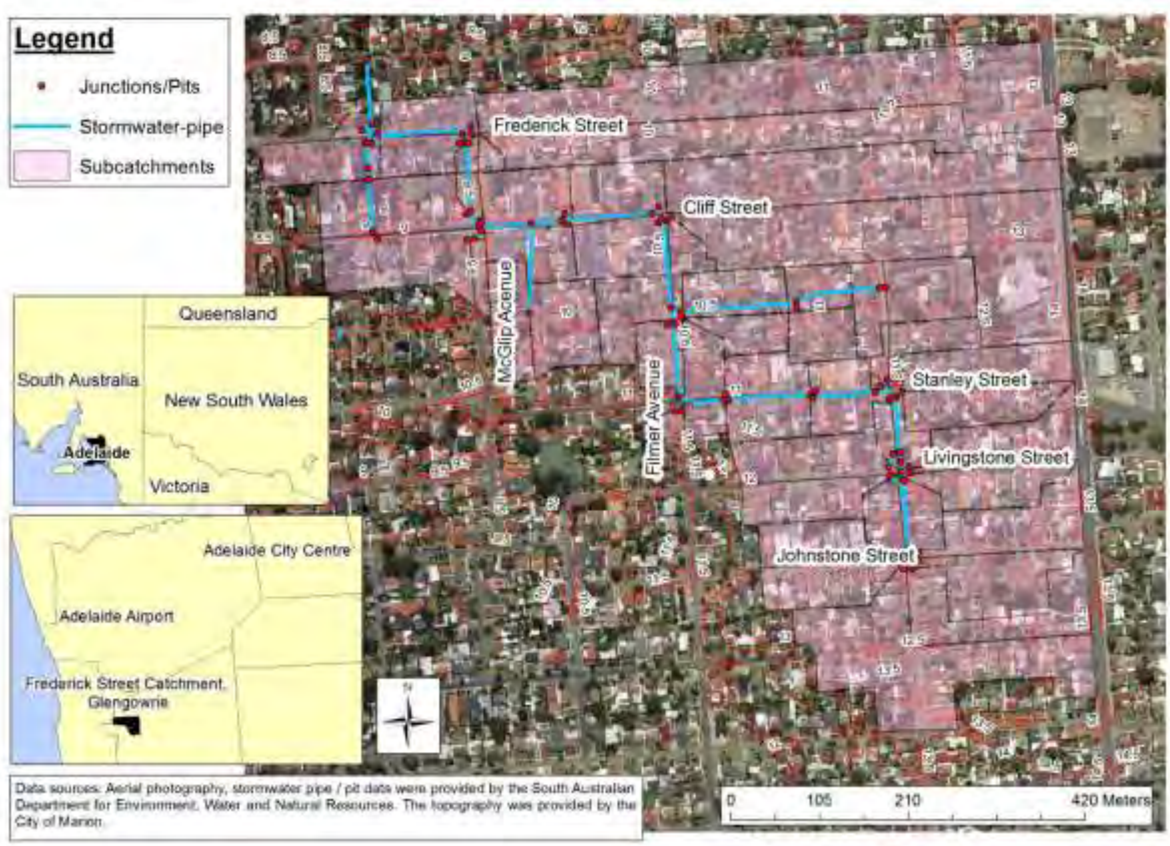


Figure 1 - Location of the Frederick Street catchment in Adelaide, South Australia indicating the catchment area, drainage system layout and roads.

Figure 2 - Location of the gauging in and near to the Frederick Street catchment in Adelaide, South Australia.

In August 2013, rainfall and flow monitoring was reinstated at the Frederick Street catchment, which forms part of the drain 18 catchment, consisting of one flow gauge and two rain gauges situated in the same location as the earlier monitoring study (Figure 2). The data collected has been used to quantify the impact of infill development on flow characteristics from the catchment using the observed record based on the procedures described in Section 4.1.2. Validation of modelling approaches determining this impact was also undertaken using the procedures described in Section 4.1.3. The results of these aspects of the study are summarised in Section 5.1.2. Appendix A consists of a full conference paper which details results comparing flows in the catchment between 1993 and 2013.

4.1.1 Aerial photography

Aerial photography of the case study catchment was available from 1966, 1993 and 2013. The aerial photography was used to determine the number of allotments and the proportion of:

- public road area;
- non-road paving (e.g. footpaths and allotment paving);
- Home roof area;
- other impervious area (which includes any clearly disconnected impervious area, such as shed roofs and
- pervious area

within the catchment boundary. Land use within the catchment boundary was analysed using ArcMap 10.5. The results of this analysis are reported in Section 5.1.1.

4.1.2 Analysis of Observed Flow Data

Previous research on the Frederick Street catchment indicated that there is little evidence of runoff occurring from the pervious portion of the Frederick Street catchment (Kemp, 2002), which is consistent with other urban catchments in Adelaide (Kemp and Lipp, 1999). Therefore, if a double mass curve of cumulative runoff volume for the catchment was plotted against cumulative rainfall volume for a defined period, a good correlation should be found with the slope of the line being close to the percentage directly connected impervious area. If infill development has an impact on runoff volume this relationship should change over time as infill development progresses. Data from two periods were therefore examined, from August 1992 until November 1996, and from August 2013 until April 2015.

4.1.3 Simulation of Change in Response to Rainfall Events

Observed changes in the response to rainfall events was further examined using conventional design storm event modelling techniques based on a model of the catchment developed in ILSAX. The approach used was to simulate a number of storms occurring in the catchment in 1992/1993 and in 2013/2014 to examine the required change in model impervious area parameters change to correctly represent runoff conditions due to infill development.

To begin, rainfall and runoff data from the seven largest storms recorded in 1992 and the five largest storms of 1993 were fitted to the ILSAX model at the Frederick Street gauging station (AW504561). Using IFD analysis techniques from a previous version of *Australian Rainfall and Runoff* (Pilgrim, 1999), it was determined that the highest rainfall intensity was between 2 and 5 years ARI for 5 minute duration.

The ILSAX model fitting procedure for 1992/1993 was as follows:

- Storms with runoff from the impervious area only were identified, by examining the percentage runoff (runoff volume/rainfall volume);
- The 1992 storms having only an impervious area runoff component were calibrated first, with model fitness improved through the use of the sensitivity adjustment available within the ILSAX model to transfer directly connected impervious area to supplementary paved area. For example a –10% sensitivity adjustment transfers 10% of the directly connected impervious area to supplementary paved area, without affecting the total catchment area.
- A paved area depression loss of 1 mm was assumed, as recommended by the ILSAX manual;
- The other storms were then modelled, using the best fit for the directly connected impervious area sensitivity adjustment. The initial loss for the impervious area was set to model the start of the rise of the gauged flow, and the initial loss for the pervious area was set to start the contribution from the pervious area where the fitted flow deviated from the gauged flow, assuming no pervious area runoff. Continuing loss on the pervious area was used to best model the total runoff volume. The apparent lag of the pervious area runoff was adjusted by altering the pervious area roughness value 'n'.

After this was completed, the process was again repeated for the 2013/2014 case as follows:

- Storms with runoff from the impervious area only were identified, by examining the percentage runoff (runoff volume/rainfall volume);
- The eight largest events were used, all of which showed no pervious area contribution
- The sensitivity adjustment available within the ILSAX model was again used to transfer directly connected impervious area to supplementary paved area.

- A paved area depression loss of 1 mm was assumed, as recommended by the ILSAX manual.

The change in the value of the sensitivity adjustment parameter in response to urban infill development over the 20 year period of observed flows are reported in Section 5.1.3.

4.2 A comparative technical and economic assessment of managing flooding using WSUD approaches and a Stormwater Infrastructure Upgrade

The capability of WSUD to manage floods in and around Adelaide was examined using simulation of case study catchments. Simulation was undertaken using calibrated catchment models of allotment to catchment scale case study areas to examine the urban runoff impacts from existing (pre-infill) catchment conditions, redeveloped (post-infill) catchment conditions and these same redeveloped catchment conditions with WSUD implemented. All simulation was conducted in the continuous simulation model PCSWMM. PCSWMM was selected because it was capable of running continuous and event simulation and includes powerful tools which aid the process of partial series analysis to estimate the AEP of peak flow and flood magnitude.

Simulation was conducted to explore a variety of catchment characteristics that might affect drainage system capacity due to infill development in Adelaide were explored including:

- Case 1: The impact of projected infill in a residential catchment, using the methods in Section 4.2.1
- Case 2: The Impact of WSUD on projected runoff conditions in a residential catchment, using the methods in Section 4.2.2
- Case 3: Comparing the impact of WSUD on flat and sloped catchments, using the methods in Section 4.2.3
- Case 4: Comparing the impact of WSUD during rainfall with lower and higher AEP, using the methods in Section 4.2.4
- Case 5: Comparing the Impact of WSUD on catchments with higher and lower infiltration rate, using the methods in Section 4.2.5
- Case 6: Comparing the effect of assessing WSUD impact in smaller to larger catchment scales, using the methods in Section 4.2.6

Due to the limitations associated with event based simulation modelling of WSUD systems (explored in detail in Section 5.3 of this report), continuous simulation modelling was employed in the study scenarios based on a 26 year rainfall time series to produce a simulated flow time series. The adopted rainfall time series has been described in Section 4.2.7. The simulated outflow time series was then assessed using the methods in Section 4.2.8 to determine the mean annual runoff volume (kL/year or ML/year), the peak flow rate at the outlet of the simulated catchment in terms of annual exceedance potential (AEP, L/s or m^3/s) and in some cases an estimate of flooding has been assessed (in kL). Broadly, the cases described above were conducted to compare the effects of infill and infill in conjunction with WSUD to determine how well the WSUD approaches maintain the total runoff volume, the peak flow rate of catchment outflow and surface ponding (flood) volume. In some of the cases, the study is exploring what other catchment influences may contribute to or hinder the ability of WSUD to maintain the pre-infill catchment conditions.

To conclude the analysis, the economic cost of implementing a number of technically feasible options was undertaken and the costs of the system upgrade, retention and detention based solutions compared. The economic assessment was limited to construction costs and has not considered indirect costs and benefits such as water quality improvement, water harvesting and/or demand reduction, maintenance costs, nor social costs and benefits. The procedures for the economic analysis are detailed in Section 4.2.14.

4.2.1 Case 1: The Impact of Projected Infill in a Residential Catchment

The impact of projected infill development in a residential catchment was examined by determining the runoff characteristics of a catchment prior to and following infill development. The catchment studied was the Frederick Street catchment located in Glengowrie, South West of the Adelaide CBD. The details of the development of this model are described in Section 4.2.11. A review of development occurring in the catchment revealed that in 1993 there were 555 residential allotments in the catchment, but by 2013 there were 632 residential allotments, indicating that 77 new allotments had been created in this 20 year period (almost 4 new allotments per year). Previous research has indicated that the change in impervious areas observed between 1993 and 2013 reduced the existing capacity of the catchment from a 33% AEP to a 36% AEP (2.5 to 2.2 year ARI). For this analysis, we have simulated a post-infill state version of the catchment where half of the allotments in the 1993 catchment were subdivided into 2 homes with a higher net impervious area on each redeveloped allotment. The nature of the assumed subdivision is identical to that outlined for the single allotment simulation scenario described in Section 4.2.9. Infill development was also restricted to lots considered to be ‘redevelopable’, on the assumption that lots with a total area *less than 500 m^2* were not suitable for infill development due to limited area for an additional home and driveway. Based on these assumptions, there were 358 allotments available for redevelopment in 1993 of which 178 were assumed to be redeveloped for the post-infill scenario. Based on the current rate of infill, this equates to a scenario which is possible in 2053, assuming that consumer demand and infill development remain constant.

The PCSWMM model of the Frederick Street catchment was used to estimate the mean annual runoff volume, the 86%, 63%, 50%, 20% and 11% AEP[1] peak flow rates and the flood volume at a selected critical point in the catchment for the pre-infill (1993) scenario and the redeveloped post-infill scenario. For convenience the AEPs are compared with corresponding ARIs in Table 2.

Table 2 –A comparison of the AEPs used in this report with equivalent ARIs.

Annual Exceedance Probability (AEP)	**Average Recurrence Interval (ARI)**
86%	0.5 year
63%	1 year
50%	2 year
20%	5 year
11%	10 year

The results of this investigation into the impact of projected infill are reported in Section 5.2.1.

[1] Note that a comparison of these AEPs and other terminology including ARIs and EYs are presented in Section 3.

4.2.2 Case 2: The Impact of WSUD on Projected Runoff Conditions in a Residential Catchment

The impact of WSUD on infill was examined using the scenarios described in Section 4.2.1. To examine the impact of WSUD, the pre-infill (1993) scenario and the post-infill scenario for Frederick Street were compared against additional scenarios where the post-infill development scenario included WSUD systems in the form of retention and detention. The design and layout of the assumed WSUD scenarios is detailed in Section 4.2.13, but included:

- Retention tanks of varying size (1 to 10 kL per allotment) and demand/infiltration capacity (50 L/day to 50 000 L/day) with 500 m^2 connected impervious area
- Detention tanks of varying size (1 to 10 kL per allotment) and orifice (5 mm to 60 mm) with 500 m^2 connected impervious area
- Street scale retention tanks at subcatchment scale, located at side entry pits and with varying size (1 to 10 kL per redeveloped allotment upstream) and demand/infiltration capacity (50 L/day to 50 000 L/day per redeveloped allotment upstream) with 100% connected impervious area
- Street scale detention tanks at subcatchment scale, located at side entry pits and with varying size (1 to 10 kL per redeveloped allotment upstream) and orifice (orifices sized to have an identical emptying time to the corresponding allotment detention tank scenarios) with 100% connected impervious area

The results were examined to assess the ability of WSUD storages at the allotment and street (or 'lumped') scale to maintain the mean annual runoff volume, peak flow rates (at selected AEPs) and flood (surface ponding) volumes in the post infill scenario to levels observed in the pre-infill (1993) scenario. The results of the investigation into WSUD performance to maintain flow characteristics with infill development are reported in Section 5.2.2.

4.2.3 Case 3: Comparing the Impact of WSUD on Flat and Moderate Slope Catchments

The impact of applying WSUD on 'flat' and 'moderate' sloped catchments was examined by comparing the impact of implementing a selection of WSUD scenarios used for Case 2 above into case study catchment areas with a flat and a moderate slope to determine if there was an appreciable change in the ability of WSUD to maintain flow characteristics where catchment slope is different. The two catchments used were the Frederick Street catchment (flat) with a moderately sloped catchment (the Paddocks, model description provided in Section 4.2.12). The flat catchment was selected to be the Frederick Street catchment. A full description of this model is provided in Section 4.2.11 and the assumed infill was that described for Case 1, Section 4.2.1. The moderate slope catchment was the Paddocks catchment situated in the City of Salisbury. A full description of this model is described in Section 4.2.12. An identical level of infill was assumed to occur in this catchment, where 50% of allotments experienced infill. In the case of the Paddocks, there were 556 allotments assumed capable of being redeveloped in the pre-infill (1993) scenario. The assumption of 50% infill lead to the redevelopment of 278 allotments. Due to the large number of model runs required, only the following WSUD solutions were compared,

- Retention tanks of 5 kL with varying demand/infiltration capacity (50 L/day to 50 000 L/day) and 500 m^2 connected impervious area
- Detention tanks of 5 kL with varying orifice size (5 mm to 60 mm) and 500 m^2 connected impervious area

- Street scale retention tanks at subcatchment scale, located at side entry pits. Tanks were assumed to be 5 kL per redeveloped allotment upstream and with a varying demand/infiltration capacity of 50 L/day to 50 000 L/day, per redeveloped allotment upstream and with 100% connected impervious area
- Street scale detention tanks at subcatchment scale, located at side entry pits. Storages were 5 kL per redeveloped allotment upstream and with carrying orifice size (orifices were sized to have an identical emptying time to the corresponding allotment detention tank scenarios) and with 100% connected impervious area

Full details of the assumed WSUD characteristics are provided in Section 4.2.13. The results of the comparison of WSUD effectiveness on a flat and moderate slope are reported in Section 5.2.3.

4.2.4 Case 4: Comparing the Impact of WSUD during Rainfall with Higher and Lower AEP

The impact of high and low intensity rainfall conditions on proposed WSUD solutions for infill was examined by looking more closely at the data developed for the pre-infill (1993), post-infill and post-infill using WSUD scenario data derived from analysis of the Frederick Street catchment simulation for Case 2, Section 4.2.2.

Firstly, to explore the effect of rainfall intensity on infill development and runoff volume, real events with varying intensity were extracted from the timeseries to determine whether the increase in runoff volume increase. Events were selected from the partial series of the pre-infill case and the post-infill case and their total runoff volume compared. Events selected were based on the catchment outflow and were estimated by the partial series analysis to be at or near to the 86%, 63%, 50%, 20% and 11% AEP[2] scenarios. The impact of intensity on peak flow rates was assessed by looking more closely at the results reported in Section 5.2.2. The results of the investigation into the effect of high and low intensity rainfall are reported in Section 5.2.4. The impact of rainfall intensity on WSUD effectiveness was then explored by examining the results reported in Section 5.2.2 in more detail, with a focus on WSUD systems with a 5 kL capacity per allotment. The results were conducted to demonstrate the change in WSUD performance with specific reference to rainfall intensity.

4.2.5 Case 5: Comparing the Impact of WSUD on catchments with Higher and Lower Infiltration Rate

The impact of implementing WSUD solutions in catchments with 'low' to 'high' soil infiltration properties was examined by looking more closely at the data developed for the pre-infill (1993), post-infill and post-infill using WSUD scenario data derived from analysis of the Frederick Street catchment simulation for Case 2, Section 4.2.2.

Note that in throughout this report, infiltration was reported as retention, and the ability of the system to infiltrate runoff is related to the 'demand' in L/day (in fact, demand is used here as a generic term for reuse demand, disposal via infiltration or both). Further details of these assumed WSUD solutions are presented in Section 4.2.13. The results of the investigation into the effect of soils with a higher and lower infiltration capacity are reported in Section 5.2.5.

4.2.6 Case 6: Comparing the Effect of Assessing WSUD Impact in Smaller to Larger Catchment Scales

To explore the impact of implementing WSUD on larger and smaller scale catchments, the impact of simulating pre-infill, post-infill and post-infill with WSUD flow characteristics of infill development at the scale of a residential allotment, a residential street and a larger catchment were comparted. The simulation of a pre-infill and post-infill residential allotment was undertaken using the assumptions provided in Section 4.2.9. Development at the allotment was represented by a simple 1 home into 2 scenario. The simulation of a pre-infill and post-infill residential street was undertaken using the assumptions provided in Section 4.2.10. In summary, the street consisted of a conceptualised sub-catchment of Frederick Street with 20 homes, of which 10 were redeveloped in accordance with the allotment case. The catchment scale case was the full Frederick Street catchment, using the model described in Section 4.2.11 and infill occurring in 50% of available allotments, as described for Case 1 in Section 4.2.1.

This comparison was considered important to identify whether the simulation of infill and WSUD at the allotment scale or catchment scale yielded variations in performance due to catchment characteristics. Such variation would need to be considered in the development of an assessment criteria for redeveloped allotments where WSUD was installed to maintain the overall catchment flow regime. The results of the investigation into the effect of simulating WSUD effectiveness at the larger and smaller catchment scales are reported in Section 5.2.6.

4.2.7 Climate Data

As noted earlier, continuous simulation was used to assess the effectiveness of WSUD in all of the cases described in Section 4.2.1 through to 4.2.6. Continuous catchment flow data for all cases was generated by catchment simulation in PCSWMM using continuous rainfall records. These records were extracted from rainfall data collected and maintained by the South Australian Department of Environment, Water and Natural Resources (DEWNR) for Happy Valley, SA (Gauge A5030532). This gauge was selected as it is located within Greater Adelaide, and comprised of a 26 year record of rainfall data which was 91.9% complete. Missing and poor quality data was filled using rainfall data derived from nearby gauges. The average monthly rainfall for the Happy Valley gauge is presented in Figure 3. Average evaporation data was also input into the PCSWMM model to enable simulation of catchment drying between events. This evaporation data was taken from gridded aerial evaporation rates purchased from the Australian Bureau of Meteorology and described by Chiew et al. (2002). The average monthly evaporation corresponding with the location of the Happy Valley gauge is presented in Figure 4.

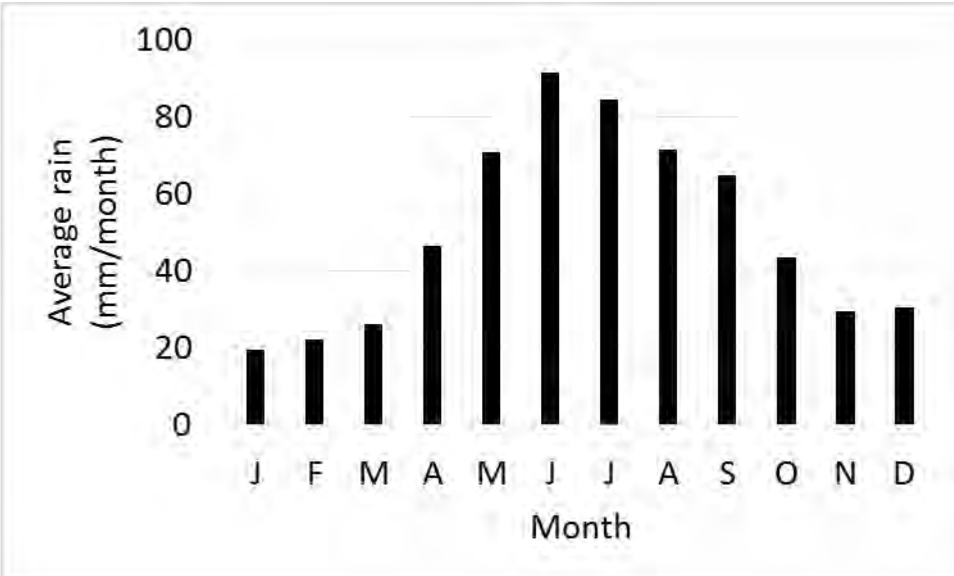


Figure 3 – Average monthly rain in the rainfall time series used from Happy Valley (A5030532)

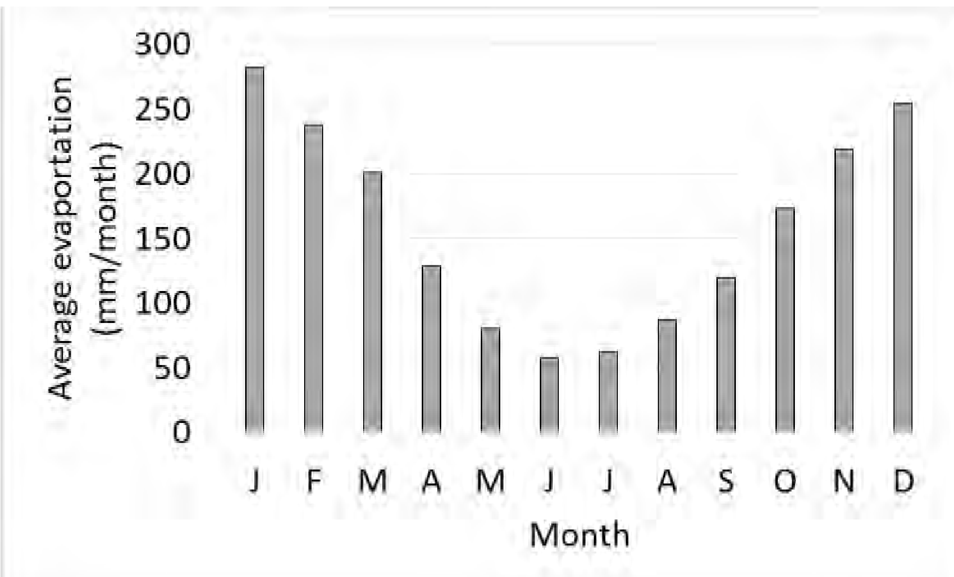


Figure 4 - Average monthly evaporation in the rainfall time series used from Happy Valley (A5030532)

4.2.8 Estimating Catchment Flow Characteristics

The flow characteristics used to assess the impact of infill and WSUD in this study included:

- Mean annual runoff volume
- Peak flow rate
- Flooding at a known 'sag' point

Mean annual runoff volume

Annual runoff volume was determined based on the cumulative volume of the outflow record at the end of the simulated catchment divided by the number of years simulated (26 years).

Peak Flow Rate

The peak flow rate was determined on the basis of known annual exceedance probabilities. The exceedance probabilities included the 86%, 63%, 50%, 20% and 11% AEP runoff event intensities. These flow rates were determined based on a partial series analysis of the outflow time series produced for the end of the simulated catchment. 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 20% AEP), 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 (1996) 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, 26 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 et al., 2014) 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 for small urban catchments. As such, a maximum of $3N$ (or 69) events was assumed to be appropriate for the partial series. In cases where more than 69 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 1. This equation was recommended by Pilgrim (1999) and is generally referred to as the 'Cunnane formula' in other literature (Maidment, 1993, Cunnane, 1978).

$$PP(m) = \frac{N + 0.2}{m - 0.4} \qquad \text{- Equation 1}$$

Where *PP(m)* refers to the plotting position, and is equivalent to the estimated AEP (also 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 the 5% AEP or 20 Year ARI). In this study, the main interest is to identify flow events up to the 20% AEP (almost a 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 20% AEP 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. An example of the resulting plot of ARI (later converted to AEP) and peak flow rate from the Frederick Street catchment is presented in Figure 5. A log trendline is fitted in the plot, however for this report due to the linear nature of the plot the resulting peak flow rate for each AEP was estimated by interpolation between points.

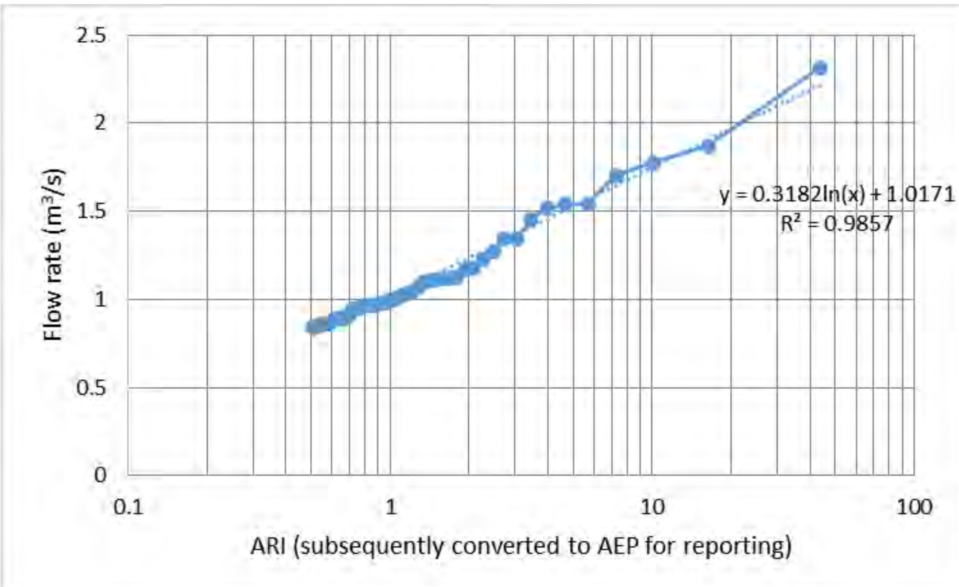


Figure 5 – Example of the partial series plot for the Frederick Street catchment (pre-infill, 1993 scenario)

Note that the peak flow rates determined for the EY/AEP values above may not match up with the flow conditions produced by storm events with an equivalent EY/AEP, because of the influence of flow routing in a catchment.

Flood Volume

In some cases, the probable flood volume was also examined at a known sag point in the Frederick Street catchment where surface overflow occurs. The location of this sag point in shown in Figure 2. The frequency of flooding was determined by recording a time series of total ponding volume at the identified sag point and applying a partial series analysis procedure (identical to that used for the flow record) to assess the extent of flooding with an 86%, 63%, 50%, 20% and 11% AEP.

4.2.9 Allotment Scale Modelling

Approach

The allotment case study site was simulated to determine the impact of infill development on peak flow at the allotment scale for Case 6 (Section 4.2.6). The allotment study was also used to compare the results of design storm based simulation and continuous simulation (Section 4.3.1), and the assumed density of infill also represents the extent of infill on each allotment assumed in the street and catchment infill scenarios.

The study determined the 86%, 63%, 50%, and 20% AEP peak flow rates resulting from a scenario of the allotment in the original (pre infill) state, a scenario of the allotment in its redeveloped (post infill) state, and multiple scenarios where WSUD has been incorporated in the redevelopment phase. The peak flow and runoff volume results of the original and redeveloped allotment were compared with the WSUD based storage scenarios to explore the effectiveness of WSUD at this scale of development to ameliorate the impact of redevelopment on stormwater flows.

Assumed Data for the Allotment Simulation

Catchment scale modelling assumed a typical '1 into 2' (or more) residence scenario. The pre-developed and redeveloped version of the allotment had the properties shown in Table 3 and illustrated in Figure 6 (note that this figure is not to scale and for illustrative purposes only). The predeveloped state of the allotment was derived by taking an average of observed allotments in the Frederick Street catchment, Glengowrie, in 1993 when very little redevelopment had occurred. The re-developed version of the allotment was derived based on consultation with representatives in the South Australian local government. The allotment scale study excluded road area at the front of the allotment.

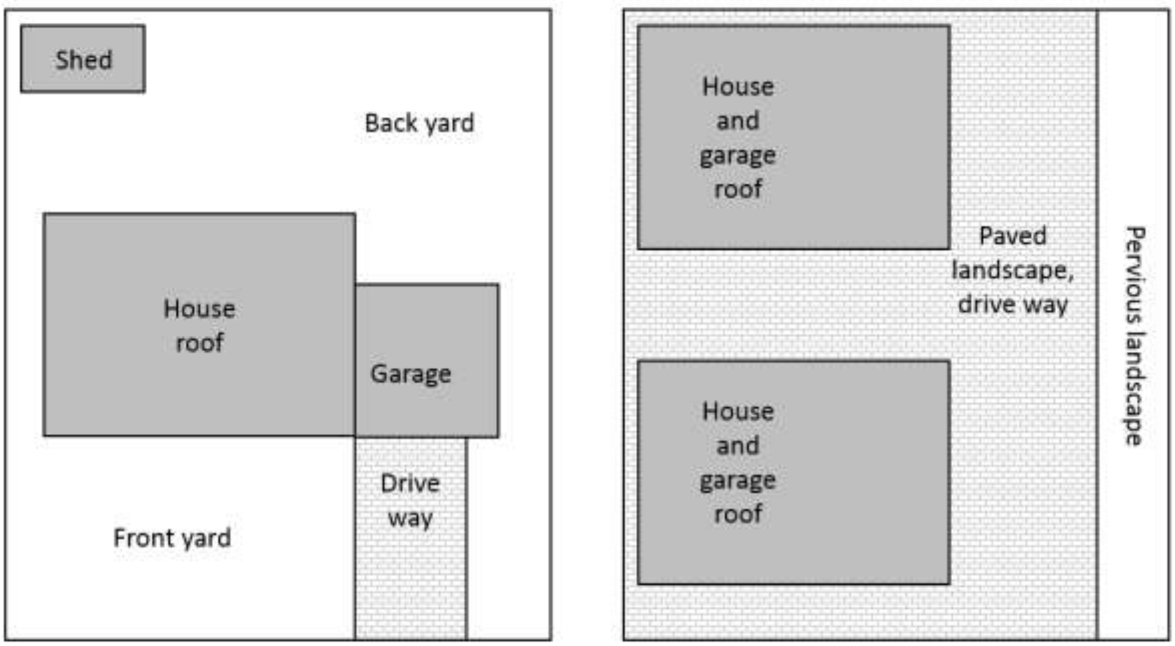


Figure 6 –Conceptual arrangement of the original (left) and redeveloped (right) allotment scenario

Table 3 – Modelling data for the conceptual arrangement of the allotment scenario

	Pre development		Post development	
Area type	Area (m^2)	%	Area (m^2)	%
Directly connected impervious	168	23	621	85
Indirectly connected impervious	138	19	37	5
Pervious areas	425	58	73	10
Total impervious	306	42	658	90
Total	**731**	-	**731**	-

Other parameters in the PCSWMM model of the allotment (all except catchment size and imperviousness data shown above) were identical to those for the PCSWMM model for Frederick Street (see Section 4.2.11). This assumption was adopted to ensure that differences with respect to scale identified for Case 6 were attributable only to catchment size.

4.2.10 Street Scale Modelling

Approach

The street study was conducted to determine the impact of infill development on peak flow at the end of a typical street of 20 homes. The study determined the peak flow rates with an 86%, 63%, 50%, 20% and 11% AEP resulting from a scenario of the street in the original (pre infill) state and a scenario of the street in its redeveloped (post infill) state, where half the homes in the street have been subdivided into 2 homes with higher net impervious area. In addition, there were multiple scenarios where WSUD has been incorporated in the redevelopment phase. The peak flow and runoff volume results of the original and redeveloped street are compared with the WSUD scenarios to explore the effectiveness of WSUD based storages at this scale of development to reduce the impact of redevelopment on stormwater flows.

Assumed Data for the Street Simulation

Street scale modelling assumed a typical 1 into 2 (or more) residence scenario. The pre-developed and redeveloped version of the allotment had the properties shown in Table 4. An illustration of the original scenario is shown in Figure 7, and the redeveloped scenario in Figure 8. Note that these figures are not to scale and for illustrative purposes only. The predeveloped state of the each allotment in the street was derived by taking an average of observed allotments in the Frederick Street catchment in 1993 when very little redevelopment had occurred. The re-developed version of each allotment was derived based on consultation with representatives in local government. While the allotment scale study excluded road area, it was included in the street scale scenario.

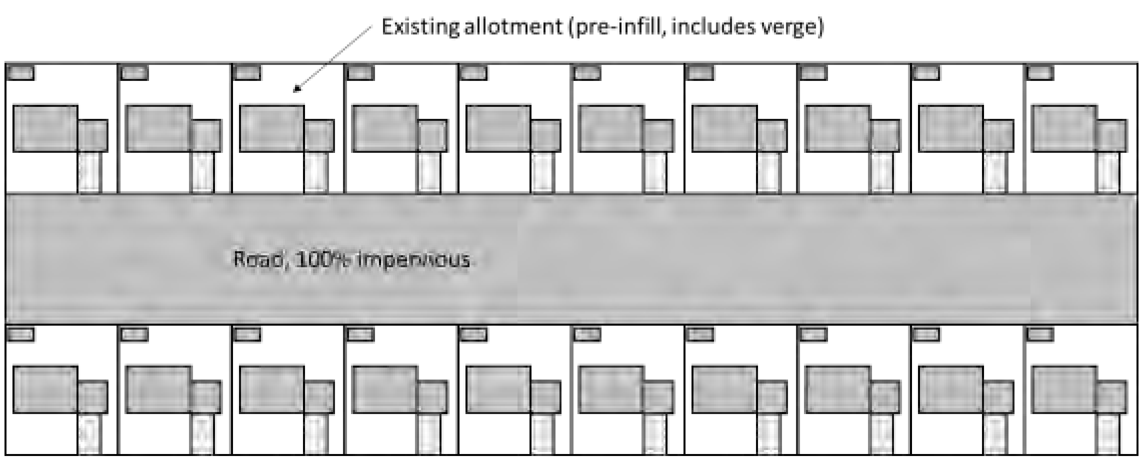


Figure 7 – Conceptual arrangement of the street scenario – existing (pre-infill)

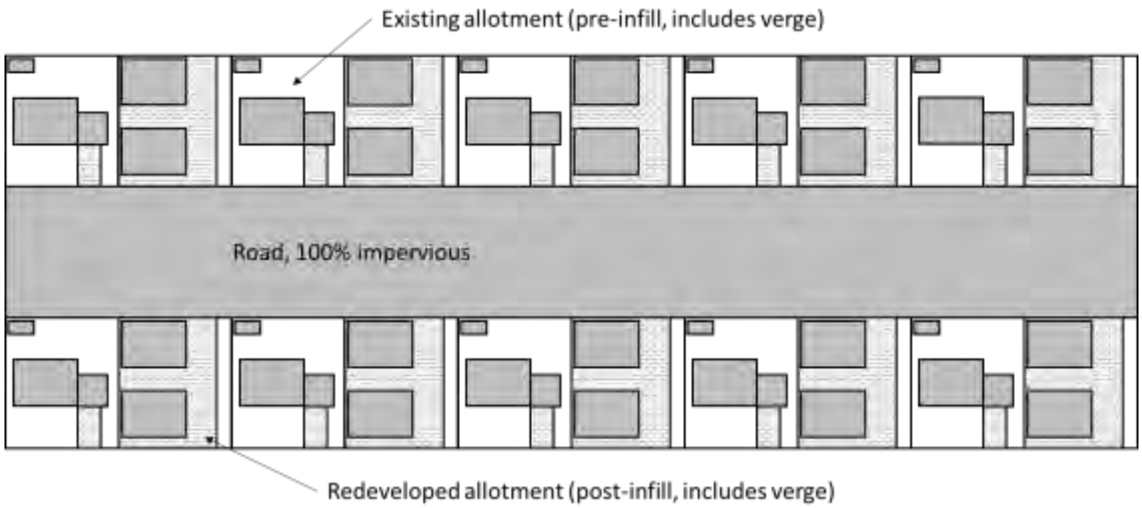


Figure 8 - Conceptual arrangement of the street scenario – redeveloped (post-infill)

Table 4 – Modelling data for the conceptual arrangement of the street scenario

	Pre development		Post development	
Area type	Area (m^2)	%	Area (m^2)	%
Directly connected impervious	4560	29	9094	57.5
Indirectly connected impervious	2760	17	1746	11.0
Pervious areas	8500	54	4981	31.5
Total impervious	7320	46	10839	68.5
Total	**15820**	-	**15820**	-

Other parameters in the PCSWMM model of the allotment (all except catchment size and imperviousness data shown above) were identical to those for the PCSWMM model for Frederick Street (see Section 4.2.11). This assumption was adopted to ensure that differences with respect to scale identified for Case 6 were attributable only to catchment size.

4.2.11 Catchment Scale Modelling - Frederick Street

Approach

The catchment scale simulation undertaken in this report used the Frederic Street catchment in the City of Marion. The 44.7 Ha Frederick Street catchment, illustrated in Figure xxx is known to be subjected to redevelopment and also benefits from current and historic flow and land use monitoring in 1992/1993 and 2013/2014 (see Section 4.1). We were therefore able to produce a calibrated model of the catchment to the 1993 case, and simulate ongoing development within the catchment boundary with and without WSUD.

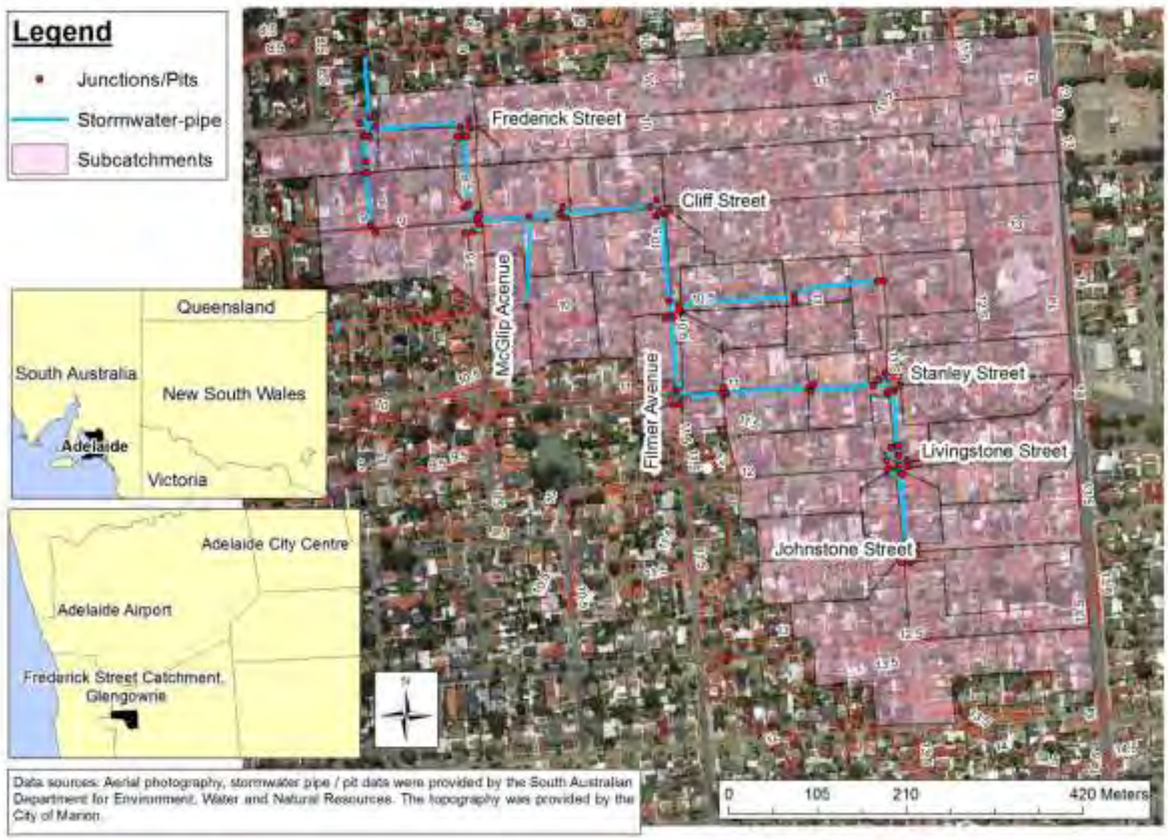


Figure 9 – Location of the Drain-18 catchment, indicating selected roads

Simulation of the Frederick Street catchment was undertaken using a model which was calibrated to reflect the catchment conditions in 1993. A detailed description of the model development has been provided previously (Myers et al., 2014). Simulation of the catchment was continuous, using a 26 year climate record (see Section 4.2.7). Detailed information on the development, calibration and verification of this model, including the model parameters, prior to running it for the pre-infill (1993), post infill and post infill with WSUD scenarios in this report is provided in Appendix B.

4.2.12 Catchment Scale Modelling – Paddocks Catchment

An additional catchment scale model was also applied to compare the impact of slope on the effectiveness of WSUD in accordance with the procedure in Section 4.2.3. The model applied for this purpose was a calibrated model of the Paddocks catchment, located in the suburb of Para Hills, and situated in the foothills area of the City of Salisbury local government area. The catchment was selected because it has an average slope of 4 to 5% and there was a body of catchment outflow data available to calibrate the model. The available rain and runoff flow data was collected between 1992 and 1995, which corresponded with the original body of flow data collected for the Frederick Street catchment. Simulation of the Frederick Street catchment was undertaken using a model which was calibrated to reflect the catchment conditions in 1993. Detailed information on the development, calibration and verification of this model, including the model parameters, prior to running it for the pre-infill (1993), post infill and post infill with WSUD scenarios in this report is provided in Appendix B. Simulation of the catchment was continuous, using a 26 year climate record (see Section 4.2.7). Figure 10 shows the location of the catchment, catchment boundary and drainage system layout.

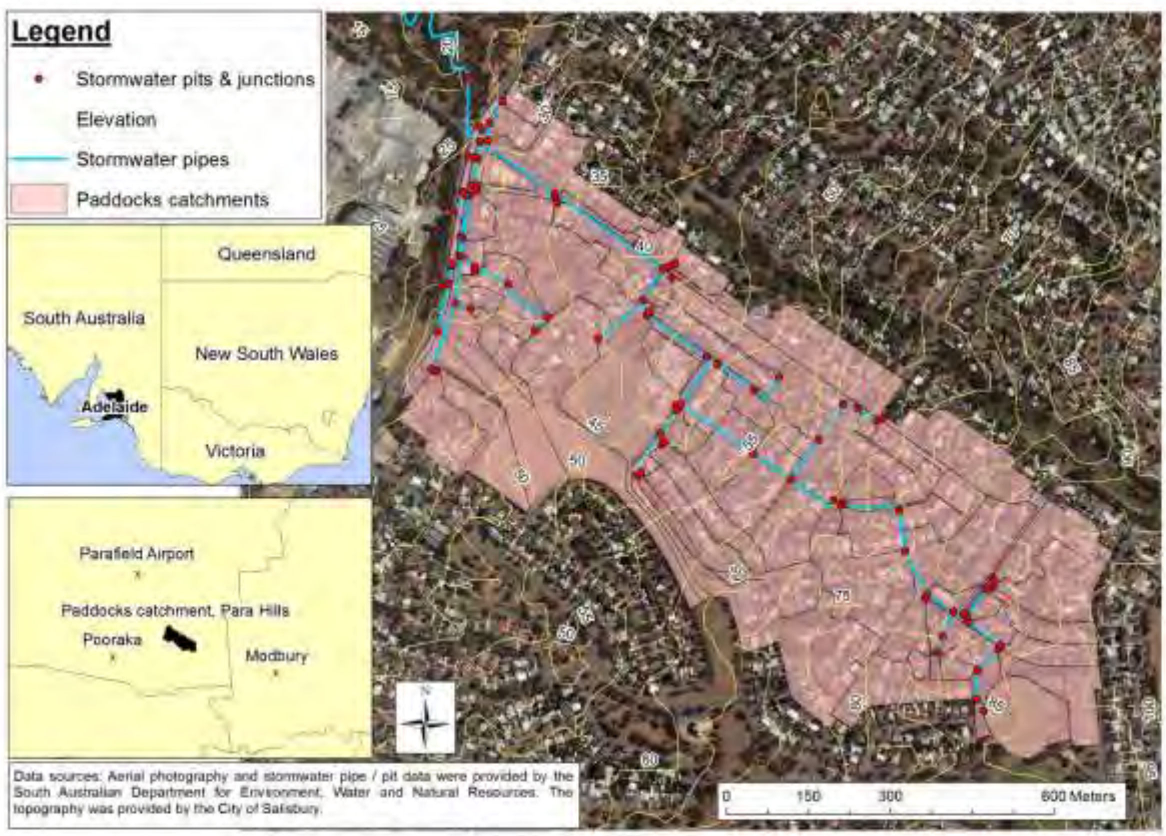


Figure 10 - Location of the Paddocks catchment model, indicating catchment boundary, drainage system layout and surrounding suburbs

4.2.13 WSUD Approaches

In simulation scenarios, WSUD was represented as either a detention or retention based storage system, and was assumed to apply at either allotment level (distributed) or at the street level (lumped). These scales were selected based on a philosophy of providing treatment which acts immediately downstream of a development. While large downstream measures may be similarly or more effective at managing catchment drainage, the gradual impact of flooding on gutter flow and stormwater pit overflow at the street level cannot be resolved at the subcatchment scale by the implementation of large, downstream storage measures. Maintaining flow prior to stormwater pits at the allotment or street scale can also prevents excessive flow entering the pipe system which may overwhelm parts of the downstream catchment drainage where infill may or may not have occurred.

Detention solutions represented solutions which temporarily hold water and release it back into the drainage system in a controlled manner. They were assumed to be represented by a detention tank fitted with an orifice, but could in reality be representative of any WSUD solution that holds a known volume and allows it to flow out to the drainage system by some means. In this research, detention tanks were assumed to be fitted with a fixed orifice size ranging from 5 mm to 60 mm at the allotment level.

A retention system is any storage that will hold water and not let it enter the drainage system. It would typically include a rainwater tank, an on-site infiltration system or rain garden/bioretention system.

These systems emptied water out of the simulation and not to the drainage system. Retention systems were assumed to be emptying at a fixed rate ranging from 50 L/day up to 50 000 L/day (a range selected to represent typical emptying rates for domestic rainwater tanks to infiltration systems with a generous foot print and high infiltration soil).

In all retention and detention tank cases at the allotment scale, there was assumed to be one tank per redeveloped allotment, and each tank was assumed to be connected to 500 m^2 of connected impervious area in the redeveloped allotment. This was a generous assumption to assess whether such connectivity would be effective, as previous studies have indicated that lower levels of connected impervious area were not effective.

As noted, in addition to simple allotment based measures, street scale or lumped measures were simulated. For the lumped detention case, a single tank in each catchment where redevelopment occurred was assumed to be installed at the stormwater collection pit. Such a tank could be physically assumed to be represented by a storage tank or biofilter device. The single tank was sized according to the number of redeveloped allotments in that catchment, and the emptying rate of the lumped detention tank was assumed to be equal to that of the individual tanks. For example, if there were 3 tanks of 5kL each with a 20 mm orifice in the allotment based detention scenario, then the equivalent street scale (lumped) scenario was assumed to consist of a single 15 kL tank at the end of the sub-catchment, with an emptying time equal to that of each 5 kL tank with a 20 mm orifice. Since the tank was located at the sub-catchment outlet (a kerbside entry pit), the connected impervious area upstream was set to be 100% of the sub-catchment impervious area.

The lumped retention case was designed similarly to the detention case and could be physically assumed to represent a large infiltration system or bioretention system. For example, if there were 3 tanks of 5kL each with a 500 L/day demand/infiltration disposal, then the equivalent street scale (lumped) retention scenario was assumed to consist of a single 15 kL tank at the end of the sub-catchment, with an emptying time equal to that of each 5 kL tank (i.e. 1500 L/day in this case). Again, the connected impervious area upstream was set to be 100% of the sub-catchment impervious area.

In all cases, overflow from the assumed WSUD storages was assumed to occur when the volume of the storage was full, and this overflow was assumed to proceed directly past the WSUD system as if it were impervious area runoff.

4.2.14 Cost Evaluation

Cost was evaluated for a conventional drainage network upgrade and for a selection of ways in which too apply the WSUD system arrangements described in Section 4.2.13. The evaluated systems included the five upgrade options:

- Option 1 – A conventional stormwater system upgrade
- Option 2 – Retention- Rainwater tanks on redeveloped allotments
- Option 3 – Retention – Infiltration soakaways on the redeveloped allotment
- Option 4 – Detention – Detention tanks on redeveloped allotments
- Option 5 – Detention – Detention storage at the street scale

The approach for each cost comparison is provided in the following sections. The results of the cost comparison are presented in Section 5.2.7.

Option 1 – A conventional stormwater system upgrade

Option 1 was evaluated based on the cost to duplicate the pit and pipe systems for the Frederick Street catchment area, including 2066 m of pipework and 78 pits or junctions. All costs for Option 1 were based on cost estimates available from Rawlinsons (2015). Costs were increased to reflect 2016 values using the Reserve Bank of Australia inflation calculator tool[2]. Note that the dual pipe system was assumed to include duplication of all pipes with a pipe of equal or approximate size via a trench 0.2 m wider than the pipe, and 1 to 2 m depth. Soil type was assumed to be clay. Reinstatement was assumed to be replacement of road surface including 200 mm road base and 25 mm bitumen. These costs were assumed to apply regardless of whether road reinstatement is necessary, to account for the increased cost of reinstating small areas of road surface compared to larger scale road replacement. Pits were assumed to consist of a 900 x 900 mm pit or a 600 x 600 mm pit, depending on required pipe diameter.

Costs considered include the cost of:

- Excavation/trenching and backfill
- Pipe and pipe laying
- Junction pits
- Reinstating disturbed road surface

Costs for Option 1 exclude additional costs due to

- any required survey works and service location
- traffic control during the construction process
- Additional inlets at kerbside
- disposal of excess fill
- any maintenance requirements over the life of the new system
- Detailed design and construction management

Option 2 – Retention- Rainwater tanks on redeveloped allotments

Option 2 considered the installation of distributed retention tanks on redeveloped allotments, and consisted of a single above ground rainwater tank of either 2 kL, 5 kL or 10 kL located at each allotment and plumbed into the home allotment. Costs for retention tanks were based on costs provided by Marsden Jacob and Associates (2007). Costs were increased to reflect 2016 values using the Reserve Bank of Australia inflation calculator tool[3]. Due to the age of the costing source (published in 2007), costs were moderated using costings based on personal communication with a home construction cost estimation professional at Rivergum Homes Group.

The costing of the retention tanks included:

- tank
- pump
- electrical connections
- plumbing connections

Retention tanks costs excluded:

[2] The Reserve Bank of Australia inflation calculator is available at http://www.rba.gov.au/calculator/
[3] The Reserve Bank of Australia inflation calculator is available at http://www.rba.gov.au/calculator/

- footings or support stands where required
- electricity use over the life of the tank
- maintenance over the life of the tank
- savings for bulk purchases (e.g. in a situation where tanks may be collectively installed by council or other authorities as a retrofit project).
- Detailed design and construction management (if required)

Option 3 – Retention – Infiltration soakaways on the redeveloped allotment

Option 3 considered cost of installing distributed retention tanks on redeveloped allotments in the form of allotment soakaways. Soakaway volumes of 1.5 kL or 11 kL were considered. Costs for soakwells were based on costs provided by Rawlinsons (2015), although it should be noted that these costs were for Western Australia only and prices may be higher in Adelaide, South Australia. Costs were increased to reflect 2016 values using the Reserve Bank of Australia inflation calculator tool[4]. The cost of plumbing to the soakwell on each allotment was acquired based on estimates made available by personal communication with a home construction cost estimation professional at Rivergum Homes Group. The resulting costing was sensitive to the assumed distance of plumbing required to transport drainage from allotment roofs to the soakwell, and a distance of 40 m was assumed (20 m per home, 2 homes per allotment)

The costing of soakaway retention included:

- soakwell
- plumbing connections

Retention tanks costs excluded:

- any overflow mechanisms fitted to the soakwell to manage flow in excess of design volume
- maintenance over the life of the soakwell
- potential savings for bulk purchases (e.g. in a situation where soakwells may be collectively installed by council or other authorities as a retrofit project).
- Detailed design and construction management (if required)

Option 4 – Detention – Detention tanks on redeveloped allotments

Option 4 considered the cost of installing distributed detention tanks on redeveloped allotments consisting of a single above ground tank of either 2 kL, 5 kL or 10 kL located at each property. The system was considered to be an above ground tank installed like a rainwater tank. An orifice fitting was assumed to be appropriate in 75% of cases, based on gravity drainage of detained flow to the road gutter. The remaining tanks were assumed to require a pump and electrical connection to reach street drainage services. Costs for the individual detention tanks were based on costs provided by Marsden Jacob and Associates (2007). Costs were increased to reflect 2016 values using the Reserve Bank of Australia inflation calculator tool[5]. Additional costs such as the detention fitting and, where required, pump and electrical works were based on personal communication with a home construction cost estimation professional at Rivergum Homes Group.

The costing of the detention tanks included:

---

[4] The Reserve Bank of Australia inflation calculator is available at http://www.rba.gov.au/calculator/

[5] The Reserve Bank of Australia inflation calculator is available at http://www.rba.gov.au/calculator/

- Tank
- Orifice discharge kit (75% of cases, suitable where a pump was not required to empty to street)
- Pump (25% of cases, to consider allotments where a pump was required to empty to the street
- Electrical connections (only where a pump is required)
- Plumbing connections (gutter to tank only, not to end use)

Costing of detention tanks excluded:

- Footings or support stands for tanks
- Electricity use and maintenance over the life of the tank (where pump was required)
- Maintenance of tank components/orifice cleanout over the life of the tank
- Potential savings due to bulk purchase (e.g. in a situation where soakwells may be collectively installed by council or other authorities as a retrofit project).
- Detailed design and construction management (if required)

Option 5 – Detention – Detention storage at the street scale

Option 5 considered the cost of installing lumped detention tanks at stormwater pits at the street scale. The storages were assumed to consist of enlarged sections of pipes immediately next to stormwater inlets. These were sized to provide a catchment based storage of 2 kL, 5 kL or 10 kL per redeveloped allotment upstream of the inlet. Costing for the lumped detention tanks were based on pipe and excavation costs from Rawlinsons (2015). Costs were increased to reflect 2016 values using the Reserve Bank of Australia inflation calculator tool[6].

The costing of the lumped detention tank option included:

- Tank (as a length of enlarged pipe to reach the desired volume)
- Excavation and trenching (assuming 1.8 m wide, up to 2 m deep trenches) and backfill
- Backfill

Retention tanks costs excluded:

- Cost of alterations and fittings to connect pipes and drain stored water appropriately.
- Any additional support required beneath detention pipework
- Maintenance over the life of the detention storages
- Disposal of excess fill
- Traffic control during the construction process
- Detailed design and construction management

4.3 Comparison of 'design' storm approach with continuous simulation to assess flood benefits by WSUD measures

In the methodology of the previous sections where WSUD effectiveness, continuous simulation was used. In this section, the effect of assuming typical design storm based design and assessment techniques and continuous simulation to assess the effectiveness of WSUD to maintain post-infill catchment outflows to pre-infill levels were directly compared using the methods in Section 4.3.1. The pre-storm burst conditions of a simple rainwater tank subjected to long periods of observed rainfall on

[6] The Reserve Bank of Australia inflation calculator is available at http://www.rba.gov.au/calculator/

an allotment was then examined in an attempt to identify the potential for a compromise between adopting continuous simulation and design storm techniques using the methods in Section 4.3.2

4.3.1 Direct Comparison of the Output from Applying a Design Storm Simulation and Continuous Simulation Approaches

A range of catchment scales were assumed to compare the predicted performance of WSUD systems using the 'design storm' approach and continuous simulation. These included:

1. A single allotment (see model assumptions in Section 4.2.9)
2. A residential street scale study (see model assumptions in Section 4.2.10)
3. A catchment scale study (Frederick Street, see model assumptions in Section 4.2.11).

Design storm event data adopted for this study was determined for the 86%, 63%, 50% and 20% AEP storms occurring in Happy Valley, SA (35.075 S, 138.577 E). This location was selected because it aligns with the rainfall record used for continuous simulation (details in Section 4.2.7). Design storm events were comprised of the 2016 intensity-frequency-duration data from Australian Rainfall and Runoff (ARR) (Ball et al., 2016a). For simplicity, the original 1987 temporal patterns were used to produce design storms which were incorporated into the rainfall runoff model. It is acknowledged that the new ARR recommends running a series of ten temporal patterns or 'ensembles' (see ARR, Section 5.5). It is recommended that a follow up project will assess the effect of the variation in the 10 ensembles for the proposed site in Happy Valley (S-SW Flatlands, East).

The continuous simulation approach was undertaken using a partial series analysis of any surface flooding and outflow data using the continuous rainfall records in Section 4.2.7, with runoff frequency estimates conducted using partial series analysis in accordance with the method described in Section 4.2.8. Results were compared for the allotment WSUD scenarios described in Section 4.2.13.

Note that the comparison of the two approaches is comparing the flow resulting from a design storm estimate, and the peak flow resulting from an analysis of the continuous flow record. As noted earlier, there are anticipated differences in the flow produced by a 50% AEP storm and the flow that occurs in a catchment with a 50% AEP frequency because of the runoff routing parameters of a given catchment.

The retention and detention scenarios were assumed to be empty at the beginning of the designs storm event simulation and at the beginning of the continuous simulation.

4.3.2 Investigating Typical 'Pre-storm burst' Conditions for WSUD Storage Design

A significant challenge in assessing the hydrological performance of volume based WSUD systems is the need to consider the available storage that will either retain or detain the runoff. The available storage is influenced by the amount of preceding rainfall and the storage discharge rate, whether it be a slow release, infiltration or a form of extraction (e.g. in-house demand). In most cases design rainfall events, derived from ARR are used to examine the hydrological performance of WSUD measures. Where a storage is associated with the WSUD system an assumption of the available storage needs to be made in order to assess its performance. Other than continuous simulation using a long rainfall time series, there is no design method that enables the designer to support and assist in determine the available storage.

Design storm events like those presented by ARR are representation of rainfall bursts that in reality may be embedded in storms of a longer duration. Pezzaniti (2003) analysed a long time series (100 years) of historical rainfall data recorded at the West Terrace, Adelaide (BOM ref 023000). The objective of the analysis was to identify events where the rainfall depth and duration were similar to design storms in former versions of ARR (Pilgrim, 1987). Results showed that the majority of short duration (e.g. 45 to 90 min) storm bursts were embedded in storms with a longer duration. Figure 11 is an example showing a number of storm bursts (equivalent to ARR design events) in a much longer duration event. In this example the water level in a rainwater tank is also shown and it can be seen that the tank is full before both the 45 minute and 60 minute bursts occur, but there is a significant amount of available storage at the commencement of the storm which commences with an equivalent 90 min storm burst.

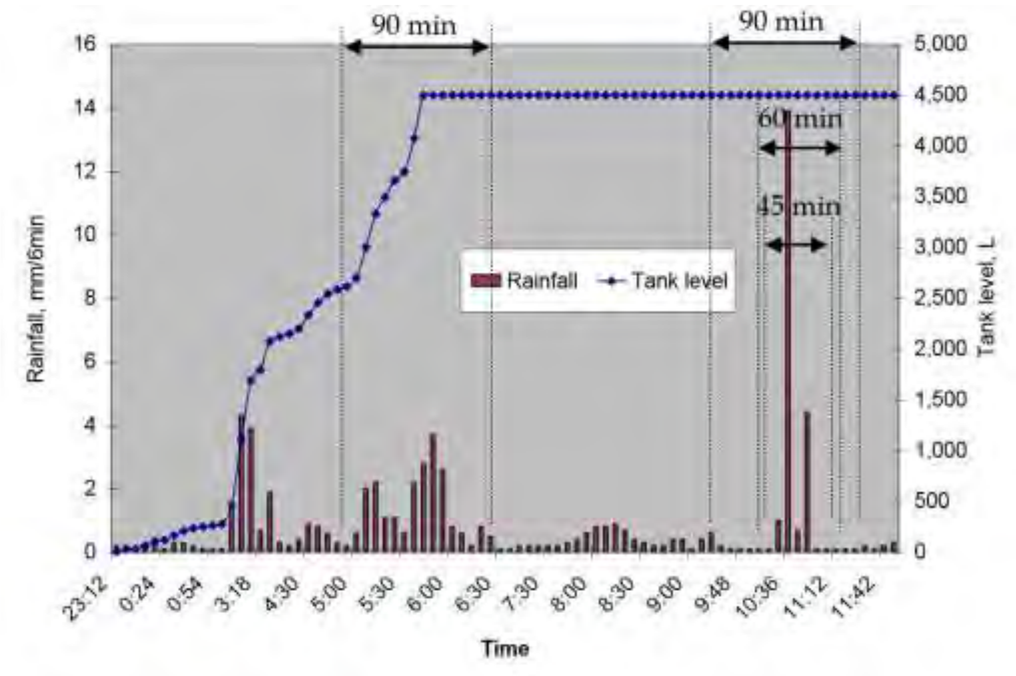


Figure 11 - Continuous simulation results highlighting the storage of a rainwater tank as two rain bursts with a 5 year ARI occurrence on 27/12/1929 Source: Pezzaniti (2003)

Similar findings have also been reported by Rigby et al. (2005) and Phillips et al. (2016), however their focus was on large catchments and the effect of the apply a storm event with and without the pre-burst rainfall. Rigby et al. (2005) found that the ARR design storm events underestimated the peak flow when the critical storm duration was less than 6 times the catchment lag duration. Phillips et al. (2016) noted that in the inclusion of pre-burst rainfall has the potential to negate adopted initial losses and partially fill detention storages prior to the rainfall burst.

ARR has recognised the need the need to consider pre-burst rainfall when assessing peak flow using design storm events. For simple urban cases ARR mentions that the application of pre-burst rainfall can be used to condition storage starting conditions. The newer release of ARR (Ball et al., 2016a) now provides advice on the application pre-burst (and post-burst) rainfall design rainfall events. However for a catchment where the influence of initial loss is significant, the antecedent condition of a catchment will be still be an important consideration. Currently, ARR does provide pre-burst rainfall depths for the preceding 24 hours of design event with a one hour or greater in duration. There is no guidance on temporal distribution of the pre-burst rainfall, and no guidance for events less than one hour duration.

To explore the impact of pre-burst rainfall on a water storage attached to an allotment roof, a water balance model was developed to assess the pre-burst status of a typical allotment storage based measure associated with WSUD systems on the lead up to storms equivalent to shorter duration storms with typical AEPs. To begin, rainfall records at Adelaide (Kent Town, 023090) and Adelaide (West Terrace, 023000) were combined and analysed to identify storms with a 20% AEP and approximately one hour duration. The model was then used to determine the filling and emptying of a storage tank attached to an impervious area for a typical allotment layout. To reduce the number of variables the assessment was based only on a contributing imperious area 200 m^2, but the storage volume and discharge (emptying) rate was varied in the analysis. The model was used to simulate 10 observed pre-burst rainfall periods, ranging from one hour through to two years. The complete array of scenarios examined are shown in Table 5. For each of the 12 events the current storage volume immediately prior to the target rain burst (20% AEP, one hour duration storm) was determined for each of the ten antecedent rainfall periods. The results of the analysis are presented in Section 5.3.2.

Table 5 –Modelling assessment scenarios for assessing pre-burst conditions

<table>
<tr><th>Connected impervious area</th><th>Number of 20% AEP events</th><th>Antecedent storage condition assumption a commencement of pre-burst period</th><th>Antecedent rainfall period modelled prior to burst</th><th>Storage sizes (kL)</th><th>Discharge rate (L/d)</th></tr>
<tr><td rowspan="3">200 m^2</td><td rowspan="3">12</td><td>100% full</td><td rowspan="3">1h, 3h, 6h, 12h, 1 day, 3 days, 30 days, 90 days 1year and 2 years</td><td>2</td><td>100</td></tr>
<tr><td>50 % full</td><td>5</td><td>200</td></tr>
<tr><td>0% full</td><td>10</td><td>500</td></tr>
</table>

5. Results and Discussion

5.1 Quantifying the Impact of Infill Development

Using the methods described in Section 4.1, the following sections detail the observed progress and impact of infill development in the Frederick Street catchment in Section 5.1.1 and the observed impact of this on runoff volume based on gauged flow data analysis in Section 5.1.2. Finally, the impact of the changes in observed flow on previous simulation of the catchment is examined with the results presented in Section 5.1.3.

5.1.1 Infill Development Progress

The progress of infill development in the Frederick Street catchment area was examined based on analysis of aerial photography in 1966, 1993 and 2013 using the methods and surface coverage types described in Section 4.1.1. The number of homes present in each aerial photo is presented in Table 6. The proportion of different land use types in the catchment is summarised in Table 7.

Table 6 - Number of homes in the Frederick Street catchment area in each aerial photograph.

	1966	1993	2013
Number of homes	357	577	655

Table 7 - Land use in the Frederick street catchment area based on an analysis of aerial photography

	1966		1993		2013	
	Ha	% total	Ha	% total	Ha	% total
Road	6.6	14.8	7.1	15.8	7.1	15.8
Paving (non-road)	2.4	5.4	4.5	10.0	4.9	10.9
Roof	6.1	13.6	8.9	19.7	11.2	24.9
Other impervious	2.1	4.7	3.6	8.1	3.8	8.6
Pervious	27.6	61.5	20.8	46.4	17.9	39.9
Total	44.9	-	44.9	-	44.9	-
Note: Road refers to public road area, paving refers to allotment paving and footpaths, roof refers to home roof area, other refers to disconnected impervious areas including shed roofs and backyard pergolas not connected to the home.						

The results indicate that the number of homes in the catchment has been steadily increasing due to infill development, and in conjunction with this, so has the impervious area contribution from paving (excluding public roads), roofing and other impervious area which is not expected to be directly connected, such as shed roof area. In fact, the pervious area of the catchment has decreased from 61.5% in 1966 down to 39.9% in 2013. In the period between 1993 and 2013 where the catchment was always fully developed, the total impervious area increased by 12.5%. The area of roofs increased by 26% and allotment paving by 9%.

Further analysis was undertaken to determine the proportion of impervious area attributable to infill development and improvements to existing allotments. Infill development was defined for allotments where a single allotment was split into two or more allotments. Existing home improvements were defined as cases where a single allotment received additional impervious area, such as a driveway, building extension or paving. Examples are shown in Figure 12. Over the 20 year period 1993 to 2013, the total allotment impervious area (excluding road) increased from 37.8% to 44.4% of the catchment area, an overall increase of 17%. These results are net increases, taking into account the impervious area that were removed and replaced during the 20 year period. The analysis examined the following impervious area components.

1. Roof area increase due to redevelopment
2. Roof area increase due to improvements
3. Paved area increase due to redevelopment
4. Paved area increase due to improvements

Figure 12 - Examples of infill development and existing site improvements

The analysis shows that similar increases in **paved area** were exhibited for both the redevelopment and improvement cases – paved area increase was roughly half attributable to existing home improvement and half attributable to infill development. In contrast, the increase in roof area was predominately due to infill development activity by a ratio of 4:1. Overall, the weighted contribution of infill and existing home improvement activity were 69% and 31%, respectively.

5.1.2 Analysis of Observed Flow Data

The impact of infill development on runoff characteristics in the Frederick Street catchment was conducted using the methods described in Section 4.1.2. The resulting mass curve plot comparing the observed rainfall and runoff between August 1992 and November 1996 with the mass curve plot for August 2013 to April 2016 is shown in Figure 13.

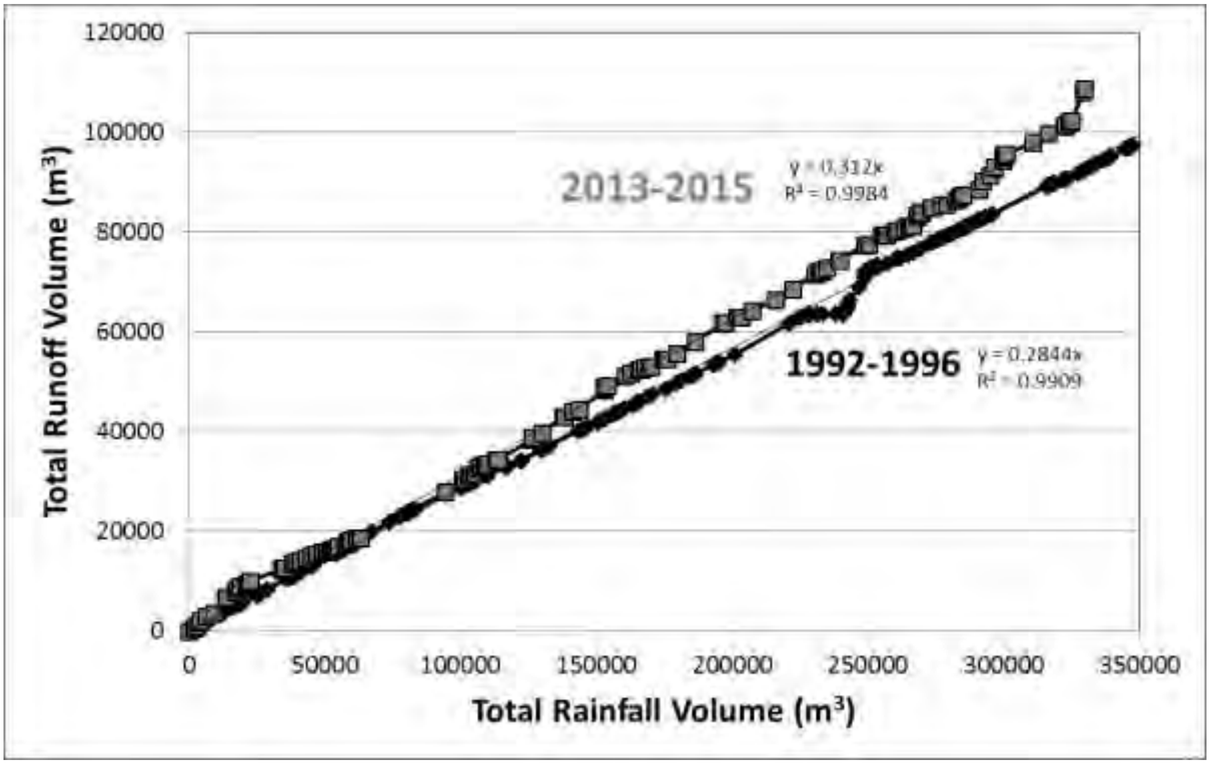


Figure 13 – Comparison of the volumetric runoff coefficient for storms occurring between August 1992 and November 1996, and August 2013 and April 2015

The mass curve plots in Figure 13 show a very strong linear relationship, except for some slight deviation in the 1992/1993 curve for events with a rainfall volume runoff between approximately 225 000 and 250 000. This deviation aside, the plots indicate that the volumetric runoff coefficient for the catchment has increased from 28.4% to 31.2%, reflecting a 10% increase in runoff volume per event which can be attributed to infill development in the Frederick Street catchment between 1993 and 2015. The total increase in impervious area over this time was 12.3% (from Table 7), which indicates that a majority of the increase impervious area was directly connected because it is directly contributing the runoff.

5.1.3 Peak Flow Analysis

The methodology used to compare the impacts of the observed increase on peak flow and runoff volume on calibrated simulation parameters in an ILSAX model of the Frederick Street catchment was presented in Section 4.1.3. In the 1992/1993 event simulations, it was found that there was no one directly connected impervious area sensitivity adjustment factor that was able to be applied to all storms to give a good match between predicted and observed flows and volumes. The effect of the constant initial loss was first investigated, but this was considered not to have a major effect. A range of sensitivity adjustment to the directly connected impervious area of 0% to -15% was examined, and an adjustment of -10% was chosen to model the storms with pervious area runoff on the basis that this adjustment was in the mid-range of the best fits for the above storms, and by inspection produced the best overall fit of the shape of the hydrographs. The result of the calibration procedure is given in Table 8.

Thus only the directly connected impervious area was used as a sensitivity adjustment, with a +5% adjustment being found to give the best overall fit to peak flow and volume. The result of the model calibration is given in Table 9.

The calibration of the same ILSAX model on the catchment from two periods twenty one years apart has shown that the required adjustment to the directly connected impervious area has changed from -10% to +5%, a significant change. Since the initial model directly connected impervious area was 30.4%, the change reflects the overall increase of 16.7% impervious area.

Table 8 – Results of the calibration procedure for events simulated in ILSAX for the Frederick Street catchment in 1992/1993

Date of storm	**Peak flow (m^3/s)**		**Ratio**	**Volume (m^3)**		**Ratio**	**Pervious runoff (%)**
	Measured	**Predicted**		**Measured**	**Predicted**		
3/07/1992	0.336	0.287	0.85	1383	1357	0.98	0
1/08//1992	0.306	0.314	1.03	909	1019	1.12	0
11/07/1992	0.128	0.142	1.11	981	971	0.99	0
19/07/1992	0.316	0.288	0.91	784	656	0.84	0
30/08/1992	1.078	1.069	0.99	3461	3158	0.91	11
31/08/1992	0.349	0.368	1.05	647	563	0.87	0
18/12/1992	1.242	1.249	1.01	5837	5801	0.99	21.6
24/05/1993	0.322	0.344	1.07	462	912	1.20	0
30/08/1993	0.534	0.654	1.23	1163	1350	1.16	0
19/09/1993	0.652	0.656	1.00	970	976	1.01	0
30/09/1993	0.312	0.255	0.82	644	617	0.96	0
17/10/1993	0.548	0.495	0.69	762	955	0.97	0
Mean			0.98			1.00	

Table 9 - Results of the calibration procedure for events simulated in ILSAX for the Frederick Street catchment in 2013/2014

Date	**Peak flow (m^3/s)**		**Ratio**	**Volume (m^3)**		**Ratio**	**Pervious runoff (%)**
	Measured	**Predicted**		**Measured**	**Predicted**		
21/08/2013	0.145	0.242	1.67	811	747	0.92	0
19/09/2013	0.334	0.462	1.38	2459	3297	1.34	0
13/02/2014	0.414	0.491	1.19	5169	6116	1.18	0
14/02/2014	0.362	0.359	0.99	6302	5641	0.90	0
29/04/2014	0.582	0.514	0.88	4314	3855	0.89	0
2/05/2014	0.505	0.402	0.80	2923	2436	0.83	0
5/05/2014	0.347	0.264	0.76	1109	867	0.78	0
9/05/2014	0.28	0.252	0.90	4113	3403	0.83	0
Mean			1.07			0.96	

All events from both periods of record were then run with the impervious area sensitivity adjustment of -10% and +5% to give an indication of the impact of the change in the catchment to overall peak flow and runoff volume from these storm events. It was found that for all events but two the ratio was very similar, indicating an increase in peak flow of 16.8% and runoff volume of 16.3% over the 21 years. The two events showing a smaller increase were the two events in 1992 that showed pervious area runoff. It would be expected for these two events that the increase would not be tied so closely to the increase in directly connected impervious area. The increase of the order of 16% in event peak flow and volume are close to those expected from the assessment of the increase in the percentage of directly connected impervious area in the catchment, outlined in Section 5.1.1.

5.1.4 Discussion

The results presented throughout Section 5.1 have clearly indicated that in a typical residential catchment experiencing infill development, the connected impervious area increases and this leads to an observed increase in runoff volume and peak flow rates. The following key outcomes of the analysis of the Frederick Street catchment demonstrate this impact:

- Over the 47 year period of observation between 1966 and 2013, the total pervious area decreased from 61.5% to 39.9%, a 35% reduction attributable to the progress of infill and improvements to existing homes.
- Over the same period, the total catchment area covered by home roofing increased from 13.6% to 24.9%, an 83% increase on the 1966 value. During this period, roofing has also been increasingly directly connected to the street via drains due to local government planning requirements.
- The area of private space paving has doubled from 5.4% in 1966 to 10.9 % in 2013.
- Analysis of observed flow data collected in 1992/1993 and 2013/2014 indicated that over the 20 year period, the runoff coefficient of the catchment increased form 24.4% to 31.2%, reflecting a 10% increase in runoff volume per event.
- The change in impervious area adjustment parameters used to accurately estimate the peak flow rate of runoff from the catchment in the ILSAX simulation tool changed from a -10% adjustment for the 1992/1993 period to +5% in the 2013/2014 period. This increase reflects an overall 15% increase in both the connected impervious area and peak flow rate of the catchment over the 20 year period.

The progress of infill development is expected to continue in light of the state government shift in focus from developing the urban fringe to denser urban living. For example, the now superseded *30 Year Plan for Greater Adelaide* (Government of South Australia, 2010) intended to shift the ratio of infill development and greenfield development from the 2010 value of 50:50 to 70:30 within the planning timeframe. This shift in development trends occurred quickly, with 76% of housing development attributed to infill by 2015, and to encourage this even further the 2017 update to the *30 Year Plan for Greater Adelaide* (Government of South Australia, 2017) intends for 85% of new housing to be built in established urban areas by 2045. The evidence presented in this report, in addition to the impacts of existing development policy in urban areas, emphasise the need to consider the adoption of tools or policy to reduce the impact of infill development on peak flow rate and runoff volume which were projected in Section 5.2.1. Alternately, catchment managers and the broader community must accept the consequences of increasing runoff volumes to receiving waters, and accept the ongoing reduced capacity of the minor drainage system in residential areas.

5.2 A comparative technical and economic assessment of managing flooding using WSUD approaches and a Stormwater Infrastructure Upgrade

5.2.1 Case 1: The Impact of Projected Infill in a Residential Catchment

The method used to predict the influence of projected future infill conditions on the Frederick Street catchment were described in Section 4.2. The impact of infill development on the mean annual runoff volume, peak flow rate and flooding in the catchment is summarised in the following sections.

Mean Annual Runoff Volume

Based on simulation of the 26 year rainfall record (Section 4.2.7), the total runoff and the mean annual runoff volume for the calibrated 1993 scenario and the re-developed scenario for Frederick Street are shown in Table 10.

Table 10 – Total and mean annual runoff volume from the Frederick Street catchment in the calibrated (1993) and redeveloped scenarios

Scenario	Runoff volume (ML)	Mean annual runoff volume (ML)
1993	1709	65.7
Redeveloped	2640	101.5

These results indicate that using the 1993 catchment as a baseline, the redevelopment of one in every 2 allotments with greater than 500 m^2 and containing only one home (i.e. 178 out of 358 allotments considered developable) caused the mean annual runoff volume to increase by 54.5%.

Peak Flow Rates

The peak flow rate of the catchment for the very frequent to frequent event probabilities in the 1993 baseline and redeveloped scenario are presented in Table 11.

Table 11 – Peak flow rate from the Frederick Street catchment in the calibrated (1993) and redeveloped scenarios for very frequent to frequent flows

Scenario	86% AEP (m^3/s)	63% AEP (m^3/s)	50% AEP (m^3/s)	20% AEP (m^3/s)	11% AEP (m^3/s)
1993	0.84	1.00	1.11	1.53	1.77
Redeveloped	1.14	1.36	1.49	1.81	1.89
Increase (%)	36	37	34	18	6

The results indicate that there was an increase in the peak flow rate for all AEPs due to infill development, however the extent of the increase gradually diminished as the event became less frequent. In other words, there was a greater increase in peak flow rates for more frequent events compared to less frequent events. This may be because pervious area runoff has started to contribute to peak flow rates in less frequent events, reducing the impact of the additional impervious area in these events.

Catchment Flooding

Flooding in the existing and redeveloped catchment scenarios was assessed at a known sag point in the Frederick Street catchment. The volume of water estimated to pond at the surface of this catchment for the very frequent to frequent events in Table 11 are presented below in Table 12. The results show that there is not a simple relationship between event frequency and the volume of flooding occurring at the selected catchment point. This may be because of the occurrence of flooding at other points in the catchment for less frequent events, masking the impact at the selected location as discussed below. Nevertheless, the results indicate an increase in flood risk as a consequence of urban infill.

Table 12 – Volume of water ponded at a known sag point of the Frederick Street catchment in the calibrated (1993) and redeveloped scenarios for a range of frequent

Scenario	**86% AEP (m^3)**	**63% AEP (m^3)**	**50% AEP (m^3)**	**20% AEP (m^3)**	**11% AEP (m^3)**
1993	0.8	2.3	5.0	19.1	35.4
Redeveloped	3.2	5.9	16.7	74.1	106.6
% increase	292	156	233	288	201

Discussion and Summary

The projected impact of ongoing infill on runoff volumes, peak flow and flooding in the catchment were provided in Section 5.2.1. The results were based on the assumption that the 1993 catchment was an acceptable baseline scenario, and that redevelopment occurred on half of the suitable allotments (178 of 358) consisting of replacing one home with two in each case. The results indicated that:

- the mean annual runoff volume increased by 54.5% following this projected infill
- the peak flow rate at the catchment outlet increased in the redeveloped scenario, and the percentage increase in peak flow was higher for more frequent events; for example, the peal flow increased by approximately 35% for the 86% AEP, 63% AEP and 50% AEP storms, but only by 18% for the 20% AEP storm and by 6% for the 11% AEP storm.
- The volume of flooding at the selected assessment point was increased due to infill development. The volume of flooding increased for all event AEPs assessed by a factor of 1.5 to 3 but did not appear to be proportional to event magnitude.

These results indicate the potential outcomes of ignoring the need for upgrades or WSUD as a means to control the increased flow rate and volume of stormwater as infill development progresses. Of some interest is the lack of a relationship between event AEP and the predicted flood volume at the selected assessment point. Further investigation indicated that flooding was beginning to occur are more locations in the catchment for less frequent events. Further research is required to determine whether the flood volume across the catchment is proportional to event magnitude, or whether the nature of individual storms (e.g. characteristics of their hyetograph) or antecedent catchment conditions (e.g. proximity to prior rainfall events) is contributing the additional flooding causing excess surface storage to occur even in less frequent events.

5.2.2 Case 2: The Impact of WSUD on Projected Runoff Conditions in a Residential Catchment

The impact of implementing WSUD in conjunction with infill development has been explored using the procedures described in Section 4.2. The full details of the assumed WSUD in the form of retention or detention were described in Section 4.2.13. The impact of installing WSUD in the form of retention or detention, and installing at the allotment or street scale, on mean annual runoff volume, peak flow rate and flood volume is summarised in the following sections.

Mean Annual Runoff Volume

The effects of implementing retention at the allotment scale are presented in Figure 14. The effects of implementing retention in lumped form at the street scale are presented in Figure 15. There are no results shown for detention system performance on mean annual runoff volume as detention does not have any impact as it temporarily holds water before letting it proceed to the drainage system.

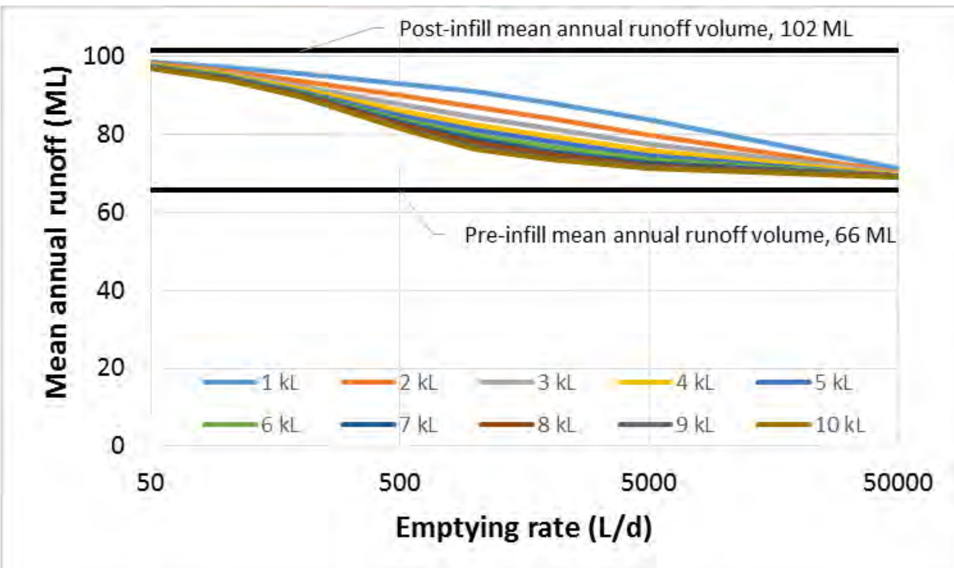


*Figure 14 – **Mean annual runoff volume** (ML) for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with a retention based storage fitted in the form of **onsite retention** systems on redeveloped allotments*

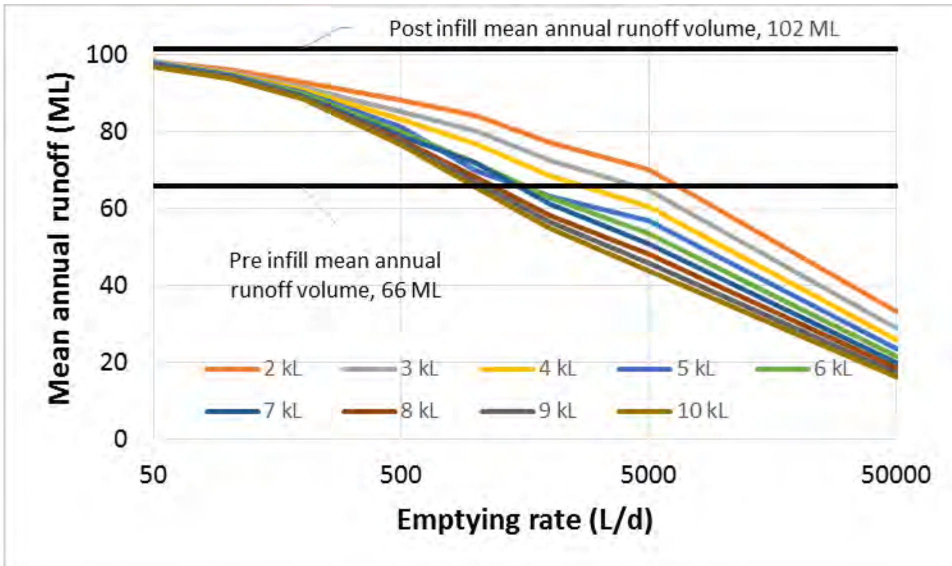


*Figure 15 - **Mean annual runoff volume** (ML) for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with a retention based storage fitted in the form of **offsite, lumped retention storages** (note volume is expressed here as a volume per redeveloped allotment in each sub-catchment).*

The results indicated that when tanks were installed onsite as part of redeveloped allotments, the mean annual runoff volume was reduced but not restored to the pre-infill development levels. However when equivalent volume 'lumped' retention systems were installed at the street scale, the mean annual runoff volume was reduced to pre-infill development runoff volumes. This is attributed to the additional impervious area connected to street scale systems – it is suggested that the higher impervious area connection leads to maximum overall effectiveness on an annual average basis. A greater runoff volume can be intercepted for small events that typically cannot fill on site systems due to limited connected area, which reduces the mean annual runoff volume.

Peak Flow rates

A summary table illustrating the ability of WSUD retention and detention systems to achieve the target peak flow rate (to reach a peak flow rate within 10% of that generated by the original 1993 catchment) and under what conditions this was possible are presented in Table 13. For brevity, only the results for the very frequent 50% AEP are then plotted in Figure 16 to Figure 19.

The results in Table 13 indicate that some retention and detention cases could maintain the peak flow rate at the end of the catchment for the frequent event magnitudes examined. For the retention cases, all cases required the very highest demand/infiltration rate of 50000 L/d. In contrast, the detention assumption has more realistic orifice size based requirements, but may be limited by the means to drain tanks to the street or drainage system.

Table 13 – Summary of simulation results indicating whether retention and detention at the allotment and street scale were successful at restoring peak flows for different event probability, also indicating the 'best' solution for that particular scenario.

WSUD Layout	Design objective (AEP)	Pre infill flow (m^3/s)	Post infill flow (m^3/s)	Target (m^3/s)	Retention success?	Best tank volume (m^3)	Minimum demand (L/day)	Resulting peak flow rate (m^3/s)	Detention success?	Best tank volume (m^3)	Orifice required (mm)	Resulting peak flow rate (m^3/s)
Lot based WSUD	86%	0.84	1.14	0.87	No	4	50000	1.07	No	7	10	0.89
	63%	1.00	1.36	1.03	No	4	50000	1.26	No	7	10	1.07
	50%	1.11	1.49	1.15	No	8	50000	1.41	No	7	10	1.18
	20%	1.53	1.81	1.56	No	8	50000	1.80	No	5	20	1.60
	11%**	1.77	1.89	1.79	No	10	50000	1.89	Yes	8	30	1.78
Lumped (street) based WSUD*	86%	0.84	1.14	0.87	Yes	6	50000	0.79	Yes	4	30 to 40	0.82
	63%	1.00	1.36	1.03	Yes	7	50000	1.02	Yes	5	30 to 40	1.06
	50%	1.11	1.49	1.15	Yes	7	50000	1.14	Yes	6	30 to 40	1.13
	20%	1.53	1.81	1.56	No	10	50000	1.65	Yes	10	40 to 60	1.53
	11%**	1.77	1.89	1.79	No	10	50000	1.88	No	10	60	1.79

* Note that street scale storage volumes must be multiplied by the number of allotments redeveloped – for example, if a 4 kL tank is required on 4 redeveloped homes at the allotment scale, the equivalent street scale scenario is assumed to be 4 redeveloped allotments × 4 kL = 1 × 16 kL tank.
** The partial series relationship for an 11% AEP is not considered reliable and any flow estimates at this magnitude should be treated with caution.

The results of the continuous simulation of the Fredrick Street catchment with WSUD fitted at the allotment site are presented for the 50% AEP in Figure 16 (retention) and Figure 17 (detention).

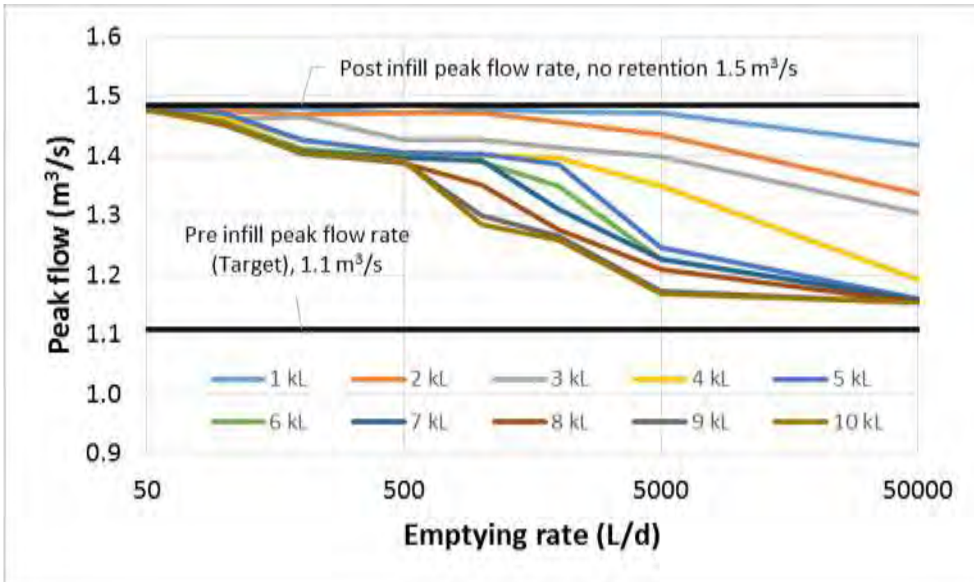


Figure 16 –50% AEP continuous simulation peak flow estimate for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with ***retention based storage fitted at the allotment level***

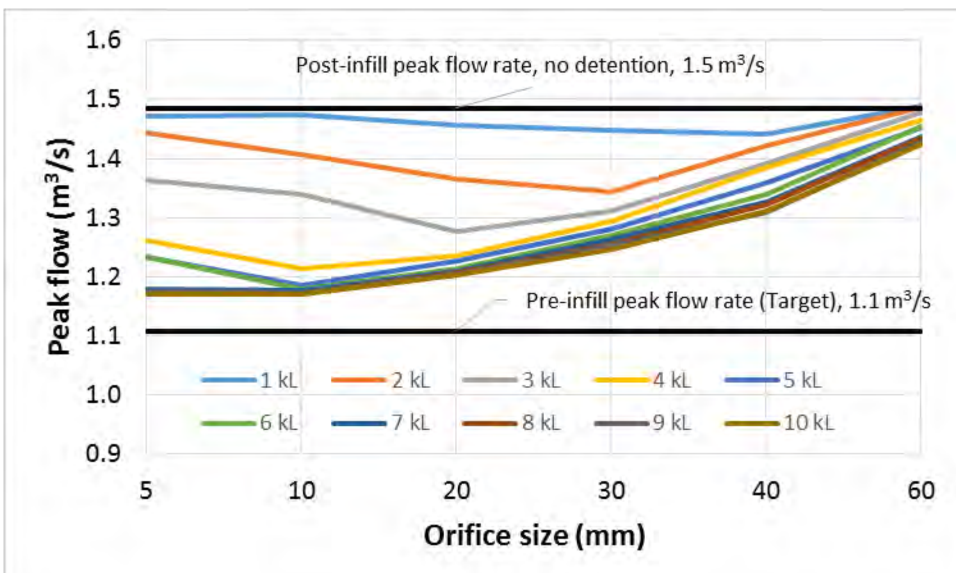


Figure 17 – 50% AEP continuous simulation peak flow estimate for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with ***detention based storage fitted at the allotment level***

These examples for the 50% AEP are broadly illustrative of all cases. A retention system can only partially limit the peak flow rate of the catchment to the pre-infill state when managing flow events with a 50% AEP. The largest retention volume (10 kL) at the largest draining rate (50 000 L/d) was almost capable of intercepting and therefore reducing peak flows to the pre-infill peak flow rate target. In the detention case, there was again no tank size that was able to limit the peak flow rates of the post-infill catchment to the pre-infill target. Storages equal to or greater than 5 kL were almost able to reach the target peak flow rate when fitted with an orifice of 5 mm to 10 mm. A larger orifice produced less effective outcomes for peak flow management on tanks of this size. For tanks 3 kL or smaller, optimal results were found with a larger orifice – 20, 30 and 40 mm for the 3, 2 and 1 kL tank sizes respectively.

The results of the continuous simulation of the Fredrick Street catchment with WSUD fitted at the street scale are presented for the 50% AEP in Figure 18 (retention) and Figure 19 (detention).

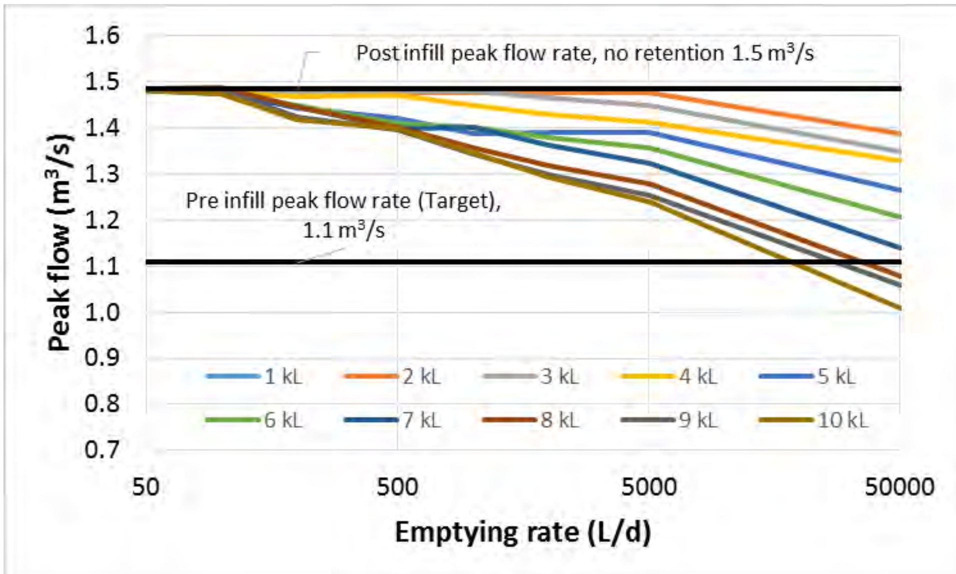


Figure 18 –50% AEP continuous simulation peak flow estimate for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with ***retention based storage fitted at the street scale***

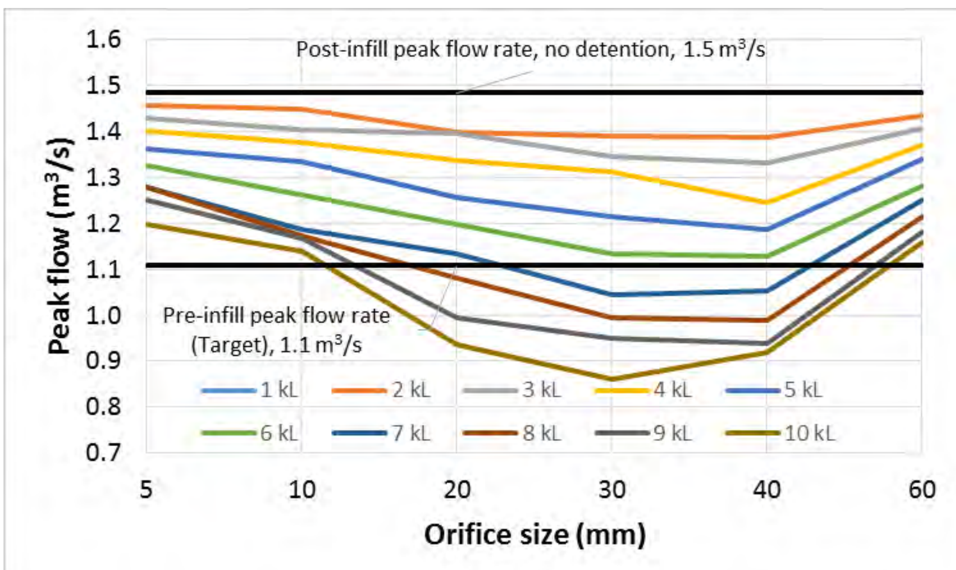


Figure 19 – 50% AEP continuous simulation peak flow estimate for the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with ***detention based storage fitted at the street scale***

The results for lumped retention and detention systems have shown that WSUD in the form of retention or detention was able to maintain the pre-infill development peak flow regime for the 86%, 63% and

50% AEP, however flow rates could not be maintained for the 20% AEP and 11% AEP. The examples given for the 50% AEP are broadly illustrative of all cases AEPs. For the 50% case, the results indicate that a retention system of approximately 7 kL volume per redeveloped allotment (or higher) could maintain the original peak flow rate when the storage had the highest level of reuse per redeveloped allotment (50 000 L/d per allotment), but retention was not effective when a more typical rate of reuse/infiltration was assumed to occur. In the detention case, storages equal to or greater than approximately 6 kL per redeveloped allotment were able to maintain peak flow rates at the end of the catchment provided the lumped tank had an orifice of 20 mm to 40 mm.

Flooding

The results of the continuous simulation of the Fredrick Street catchment with WSUD fitted at each redeveloped allotment site are presented for the 50% AEP flood estimate in Figure 20 (retention) and Figure 21 (detention).

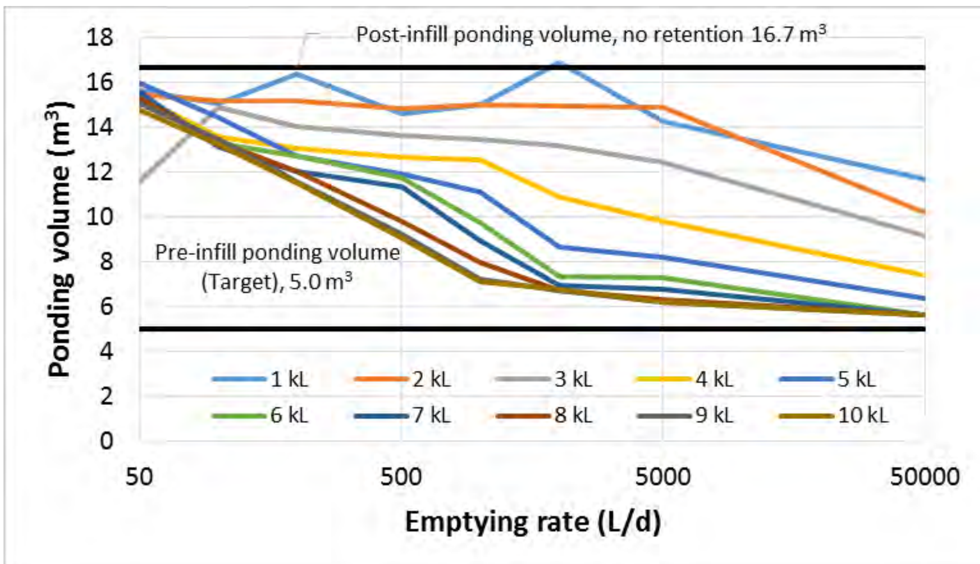


*Figure 20 - 50% AEP continuous simulation ponding volume estimate for a sag point in the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with **retention based storage fitted at the allotment level***

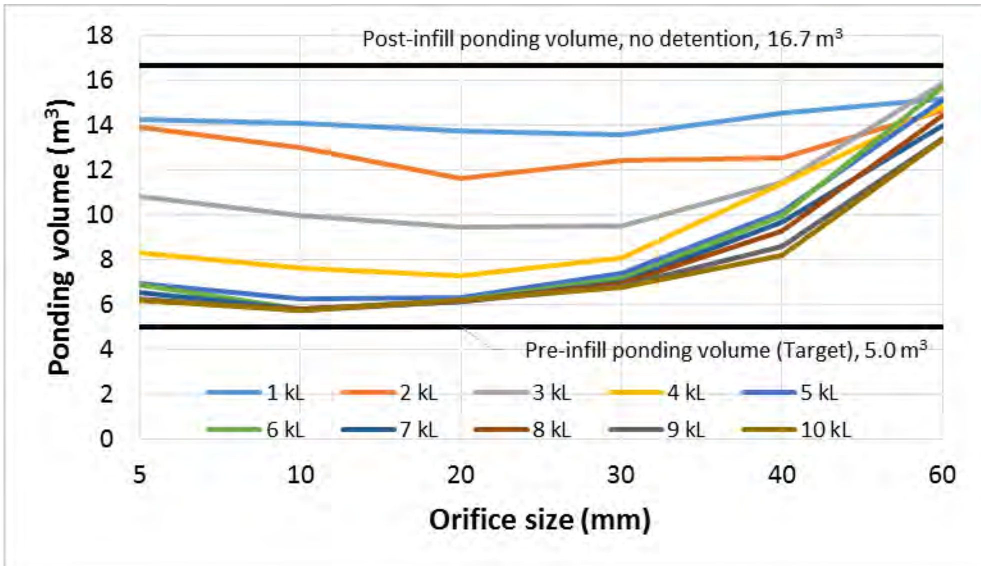


*Figure 21 - 50% AEP continuous simulation ponding volume estimate for a sag point in the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with **detention based storage fitted at the allotment level***

The results of the continuous simulation of the Fredrick Street catchment with WSUD fitted in a lumped manner at the street scale are presented for the 50% AEP ponding volume estimate in Figure 22 (retention) and Figure 23 (detention).

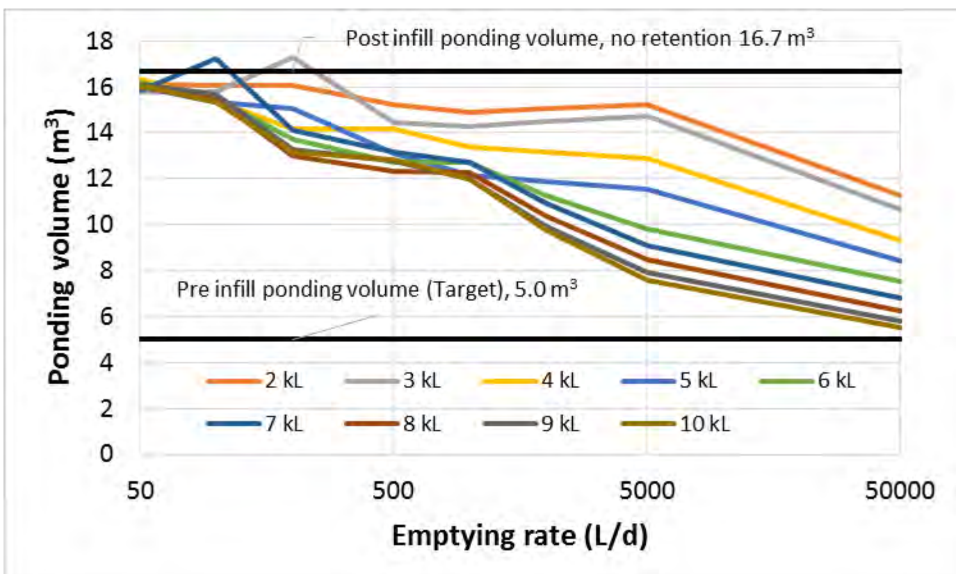


*Figure 22 - 50% AEP continuous simulation ponding volume estimate for a sag point in the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with **retention based storage fitted at the street level***

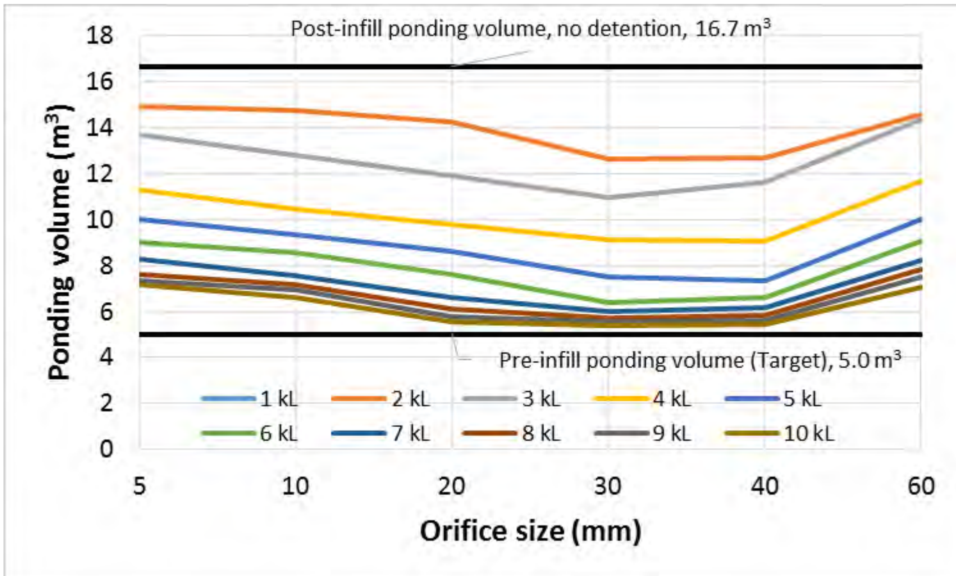


Figure 23 - 50% AEP continuous simulation ponding volume estimate for a sag point in the pre-infill and redeveloped Frederick Street catchment indicating the effect of redevelopment with ***detention based storage fitted at the street level***

The results indicate that both retention and detention have the capability to reduce the volume of ponding at the selected location in the Frederick Street catchment. Like the case for peak flow rates, retention requires maximal demand (or infiltration) to maximise effectiveness, and no volume was able to restore the surface ponding (flood) volume to that for the 1993 scenario. Detention was more effective at reducing flooding than retention. In the allotment case. Allotment based tanks 5 kL or greater fitted with a 10 or 20 mm orifice could almost maintain the flood volume, with a similar relationship found for the street scale case.

Discussion and Summary

The impact that WSUD can have on the projected infill runoff conditions has revealed several aspects of WSUD performance. The results explored the impact of both retention and detention scenarios on the mean annual runoff volume, peak flow rate and estimated flood volume. Key findings included:

- The installation of on-site retention (e.g. rainwater tanks or infiltration systems) could almost maintain mean annual runoff volume; performance was a function of tank size and anticipated reuse. As an example, the installation of 5 kL retention with 100 L/d demand on every new home in the redeveloped catchment (equivalent to one 10 kL tank on a redeveloped lot with 200 L/d demand) could only go 33% of the way to restoring the mean annual runoff volume of the catchment prior to redevelopment.
- The installation of retention in a lumped manner at the street scale was much more effective at reducing the mean annual runoff volume compared to the allotment scale. However, daily demand or infiltration was required to be up to 2000 L/day to maintain runoff volume, with a storage size of at least 5 kL per upstream redevelopment (or 2.5 kL per home).
- The peak flow rate of the catchment could not be maintained to within 10% of the 1993 baseline scenario using allotment based retention. As an example, the installation of 5 kL retention on every new home in the redeveloped scenario with 100 L/day reuse could only

reduce the peak flow rate of a 50% AEP event by 25% of that required to maintain this peak flow to the 1993 level.

- The peak flow rate of the catchment could nearly be maintained by allotment detention, but the overall performance was better than retention; for example, the use of 5 kL detention on every new home draining by gravity and with a 20 mm orifice plate could reduce the peak flow rate of a 50% AEP event by 80% of that required to maintain this peak flow to the 1993 level.
- When retention was installed in a lumped manner, performance was better than at the allotment scale, however to fully maintain the original peak flow rate at the catchment outlet required the highest level of assumed reuse or infiltration (50 000 L/d).
- When detention was installed in a lumped manner, performance was again better than detention at the allotment level; 6 kL per redeveloped allotment (or 3 kL per new home) with an equivalent emptying time to allotment based detention tanks with a 30 mm orifice were able to maintain peak flow rates in the catchment in this scenario.

The key finding in this aspect of the research is that both retention and detention performed better when lumped at the street scale compared to equivalent alternatives at the allotment scale. This is because the lumped scenarios considered tanks with larger connected impervious areas than assumed at the allotment. Allotment tanks were assumed to be above ground, limiting connected impervious area to newly developed roofs, however even if below ground tanks are assumed, the lumped tanks have a greater impervious area connection because lumped tanks can intercept runoff from existing public roadways (representing 16% of the catchment area) and impervious area on existing allotments.

5.2.3 Case 3: Comparing the Impact of WSUD on Flat and Sloped Catchments

The effect of flat and moderate catchment slopes on the effect of infill development and the effectiveness of WSUD was explored using the methods described in Section 4.2.3. The results of this investigation in terms of runoff volume and peak flow rate are reported in the following sections. Flooding was not assessed as the characteristics of each catchment are different limiting the ability to make a meaningful comparison of flood impact at any given point in either catchment.

Runoff Volume

A comparison of the increased runoff volumes due to infill development, and the extent to which this is moderated by WSUD in the flatter Frederick Street catchment and the moderately sloped Paddocks catchment are compared in Table 14.

Table 14 - Mean annual runoff volume from the Frederick Street and Paddocks catchments in the calibrated and redeveloped scenarios, including the effect of WSUD.

	Frederick Street WSUD 1[1]	Paddocks WSUD 1[1]	Frederick Street WSUD 2[2]	Paddocks WSUD 2[2]
1993 (ML/year)	65.7	88.0	65.7	88.0
Redeveloped, no WSUD (ML/year)	101.5	135.4	101.5	135.4
Increase due to infill (%)	54.5	53.8	54.5	53.8
Redeveloped with WSUD (ML/Year)	84.9	111.5	81.3	105.9
Effectiveness of WSUD (%)[3]	46.5	50.5	56.6	62.2

[1] WSUD 1 - 5 kL retention on redeveloped allotments, 500 L/day demand
[2] WSUD 2 - 5 kL retention lumped to catchment end point, 500 L/day demand
[3] Effectiveness is a measure of the extent to which pre-infill runoff volume is maintained as a percentage of the total volume reduction required.

The results indicate that the effects of assuming similar redevelopment on flat and moderate sloped residential catchments has similar effects on runoff volume. The redevelopment of 1 in every 2 lots in each catchment causes the mean annual runoff volume to increase by about 54% in each case. Likewise, the implementation of WSUD in the form of equivalent retention at the allotment or street scale lead effectively halved the extent of the runoff volume increase in each catchment. The higher slope catchment showed a slightly higher retention volume when fitted with an equivalent WSUD scenario. The cause of this is not clear.

Peak Flow Rate

A comparison of the increased peak flow of runoff with varying AEPs due to infill development in the flat Frederick Street catchment and the moderately sloped Paddocks catchment in presented in Table 15.

Table 15 - A comparison of the increased peak flow of runoff with varying AEPs due to infill development in the flatter Frederick Street catchment and the moderately sloped Paddocks catchment

Catchment	Scenario	86% AEP	63% AEP	50% AEP	20% AEP	11% AEP
Frederick Street	**Existing**	0.84	1.00	1.11	1.53	1.77
	Redeveloped	1.14	1.36	1.49	1.81	1.89
	% increase	*35.6*	*37.0*	*34.0*	*17.7*	*6.3*
Paddocks	**Existing**	1.36	1.70	1.98	2.71	3.69
	Redeveloped	1.97	2.37	2.75	3.73	3.94
	% increase	*45*	*40*	*39*	*37*	*7*

The results for the increase in peak flow following the occurrence of infill development indicate that while the extent of the increase in peak flow was greater, the relationship between AEP and % increase was similar in all cases. The percentage increase was greater in the catchment with slope, but the percentage increase reduces as the frequency of the event increases.

A comparison of the impact of different WSUD on peak flow rates of varying magnitude when installed on the flat catchment and on the sloped catchment is presented in the following figures. Figure 24 compares the effect of WSUD scenario 1, where retention tanks of 5 kL were installed on each redeveloped allotment, with 500 L/day reuse per allotment. Figure 25 compares the effect of WSUD scenario 2, where lumped retention tanks of 5 kL per redeveloped allotment were placed at each subcatchment outlet with 500 L/day reuse per allotment. Figure 26 compares the effect of WSUD scenario 3, where 5 kL detention tanks were placed on each redeveloped allotment, each with an emptying time equal to that provided by a 20 mm orifice. Figure 27 compares the effect of WSUD scenario 4, where lumped detention tanks of 5 kL per redeveloped allotment were placed at each subcatchment outlet with an emptying time equal to that of a single 5 kL tank per redeveloped allotment with a 20 mm orifice.

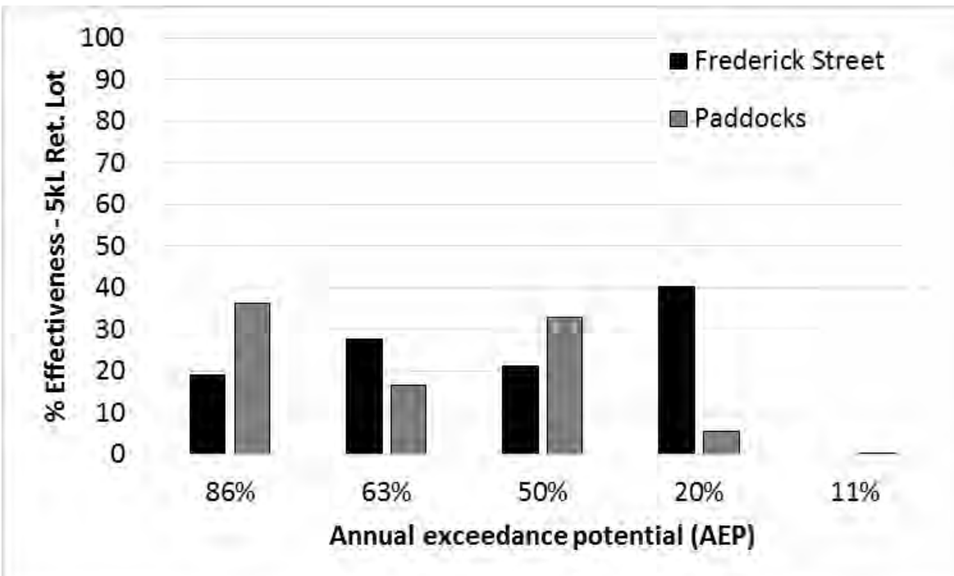


*Figure 24 – A comparison of the effect of WSUD Scenario 1 on the Frederick Street and Paddocks catchment, where **retention tanks of 5 kL were installed on each redeveloped allotment**, with 500 L/day reuse per allotment*

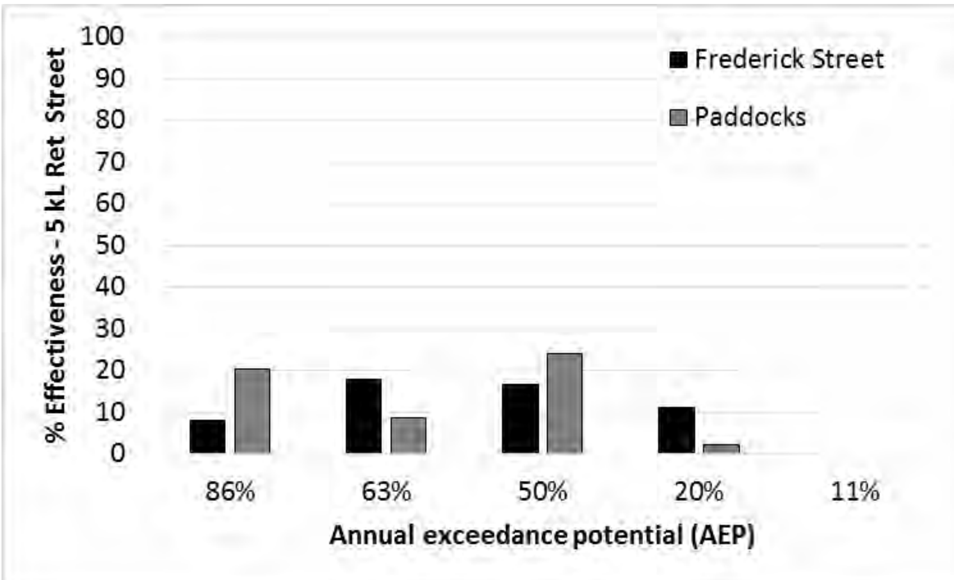


Figure 25 – A comparison of the effect of WSUD Scenario 2 on the Frederick Street and Paddocks catchment, where ***lumped retention tanks of 5 kL per redeveloped allotment*** *were placed at each subcatchment outlet with 500 L/day reuse per allotment.*

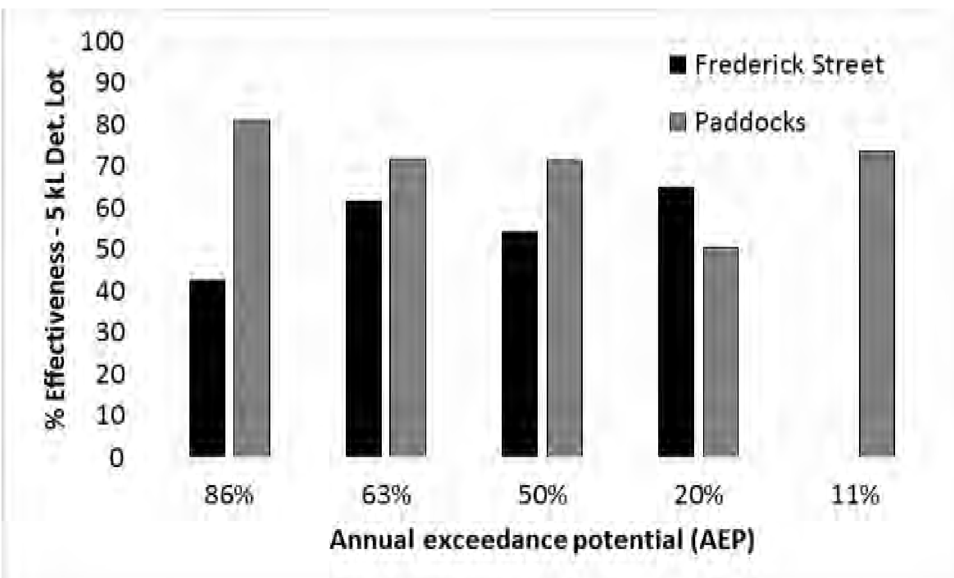


Figure 26 – A comparison of the effect of WSUD Scenario 3 on the Frederick Street and Paddocks catchment, ***where 5 kL detention tanks were placed on each redeveloped allotment****, each with an emptying time equal to that provided by a 20 mm orifice.*

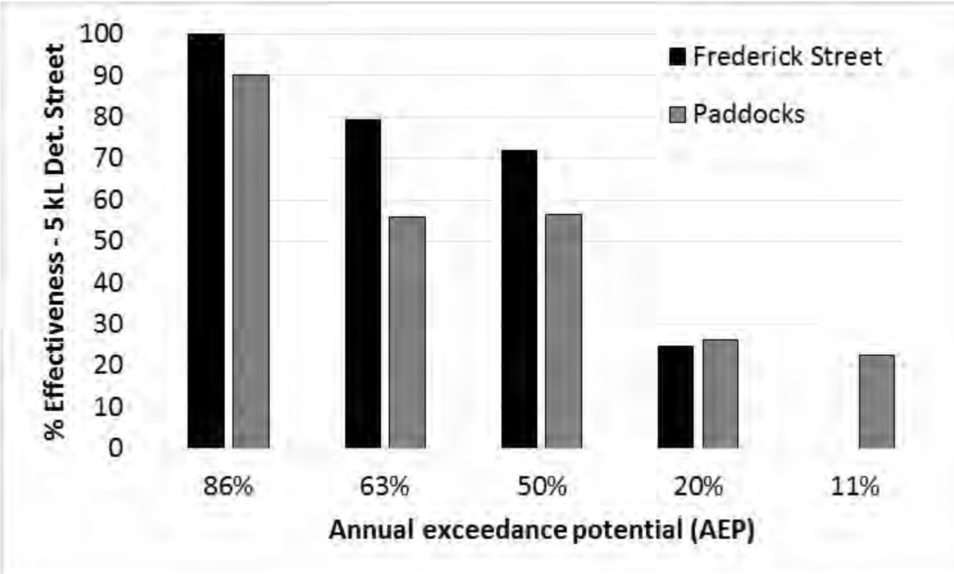


*Figure 27 – A comparison of the effect of WSUD Scenario 4 on the Frederick Street and Paddocks catchment, where **lumped detention tanks of 5 kL per redeveloped allotment** were placed at each subcatchment outlet with an emptying time equal to that of a single 5 kL tank per redeveloped allotment with a 20 mm orifice.*

The results show that for the retention based WSUD Scenarios 1 and 2 there is a similar relationship regardless of the catchment. There was no obvious difference in the effectiveness between the identical layouts of retention in either catchment. As the AEP of the event increases, there is no clear trend in either case, but it is notable that the impact of retention diminished to nil for the 11% AEP. For the detention based WSUD scenarios 3 and 4 there was a similar finding – neither catchment showed an obviously greater WSUD effectiveness when the same levels of WSUD were applied. There was little relationship apparent with decreasing frequency in the case of Scenario 3, but Scenario 4 (with lumped detention storages) showed a gradual reduction as frequency decreased toward the 11% AEP.

Discussion and Summary

Key findings from the assessment of WSUD effectiveness with respect to slope included:

- When relatively equal amounts of infill were assumed, the increase in mean annual runoff volume was almost identical for the flat and moderately sloped case.
- The impact of 5 kL retention tanks on each redeveloped allotment had a similar impact. Like the case of the flat catchment, equivalent lumped retention performed better than allotment based measures.
- The increase in peak flow rates due to infill development were consistently larger on the moderately sloped catchment compared to the flat catchment. For example, the 50% AEP peak flow rate increased by 34% on the flat catchment compared to 39% on the sloped catchment; for the 20% AEP, these values were 17.7% (flat) and 37% (sloped). In both cases, the percentage increase was lower as the AEP became less frequent.
- Despite the difference in peak flow rate, there was no clear difference in the effectiveness of identical retention or detention layouts on either the flat or sloped catchment.

To explore the impact of slope as a variable further, it is recommended that more catchment case studies be undertaken to more rigorously identify whether slope has an impact on the effectiveness of

WSUD, especially for larger catchment areas. This study was however limited by the availability of data for calibration of simulation tools.

5.2.4 Case 4: Comparing the Impact of WSUD during Rainfall with Higher and Lower AEP

The effect of low and high intensity rainfall conditions on the impact of infill development and the effectiveness of WSUD was explored as described in Section 4.2.4. The results of this investigation are reported for runoff volume, peak flow rate and flooding in the following sections.

Runoff Volume

An indication of the impact of lower and higher intensity rainfall events on the increased runoff volume produced by infill development has been determined using examples of actual rainfall events which roughly correspond with the frequency of events determined using the continuous time series. The results are presented in Table 16.

Table 16 – The effect of infill development on the runoff volume produced by increasing rainfall intensity

Event frequency (AEP, %)	Date	Duration (hours)	Total rain (mm)	Runoff volume, pre-infill (m^3)	Runoff volume, post infill (m^3)	Increase (%)
86	25/03/2002	2.5	10.4	1420	2217	56.1
63	23/04/2012	2.5	9.4	1296	1979	52.7
50	4/07/1990	6	28.2	3955	6077	53.7
20	29/06/2010	9.5	47.6	7793	11360	45.8
11	6/06/2003	5	32.4	5571	7252	30.2

As may be anticipated, higher intensity rainfall events generally produced more runoff; the exception being the 20% AEP and 11% AEP – the runoff volume was lower for the 11% AEP because although the event included a higher rainfall intensity, the consideration of the event in full required a much shorter duration. It is apparent however that the percentage increase in the runoff volume produced by less frequent, higher intensity events gradually diminished as frequency reduced. The results for individual events therefore reflect the results for the overall mean annual runoff volume reported in Section 5.2.1.

Peak Flow Rate

The relative impact of higher and lower intensity rainfall events on the peak flow rate from a catchment subject to infill development was previously shown in Table 11 (Page 45). The percentage increase data from this table have been plotted in Figure 28 to demonstrate the relative impact of event frequency on the percentage increase in flow rates. The results indicated that the peak flow rate of runoff increased by 36% for the more frequent 86% AEP storms, but by only 6 % for the less frequent 11% AEP storm. Overall, these results suggest that smaller, more frequent flow events (form lower intensity rainfall) are strongly affected by infill development, but larger, less frequent flow events from high intensity rainfall are not strongly affected.

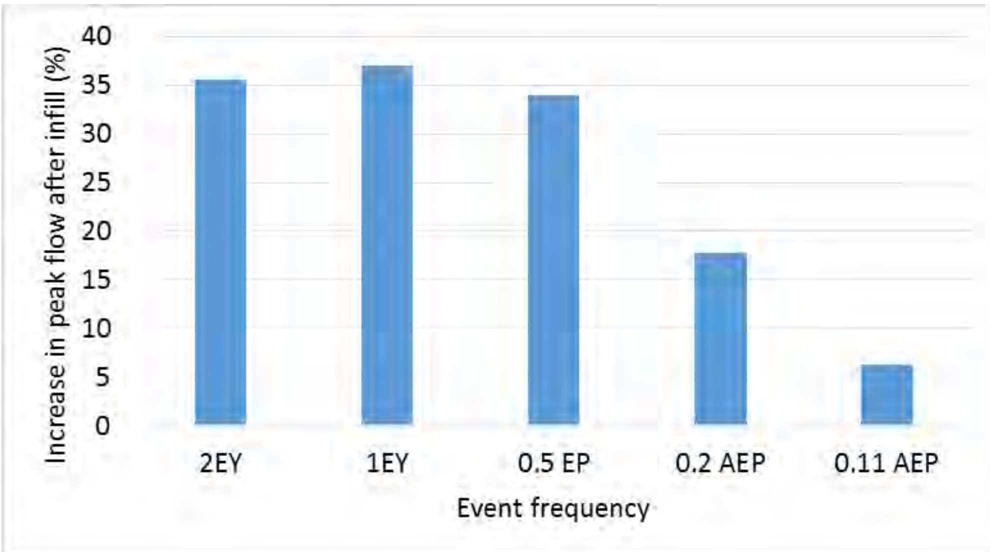


Figure 28 – Comparing the influence of runoff frequency (and rainfall intensity) on the percentage increase in peak flow rate due to infill development in the Frederick Street catchment

The relative impact of higher and lower intensity rainfall events on the ability of WSUD to preserve the peak flow rate in catchments experiencing infill is compared for some more successful scenarios in the Figure 29 (for the case of 10 kL lumped retention tanks) and Figure 30 (for the case of 5 kL lumped detention tanks). In each case, the plot compares the impact of the stated tank size with respect to predicted flow frequency. The percentage effectiveness on the y-axis represents the percent to which the pre-infill peak flow rate was maintained by the WSUD measure – for example, 100% indicates that the peak flow rate was fully maintained, 50% indicates that the peak flow rate was reduced to half way between the pre-infill peak flow rate and the post-infill peak flow rate (with no WSUD in place). Anything higher than 100% indicates that the measure reduced the peak flow rate to a level below the estimated pre-infill peak flow rate.

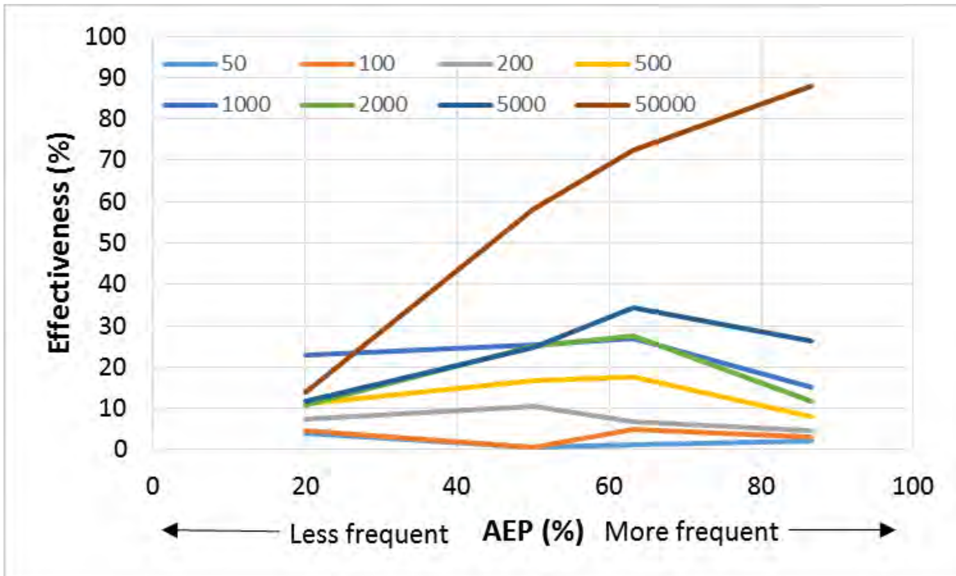


*Figure 29 – Comparison of the **percentage effectiveness of 5 kL retention tanks** (with varying reuse) to maintain peak flow rates in the Frederick Street catchment with respect to flow frequency, where high AEP indicates lower intensity rainfall and low AEP indicates high intensity rainfall.*

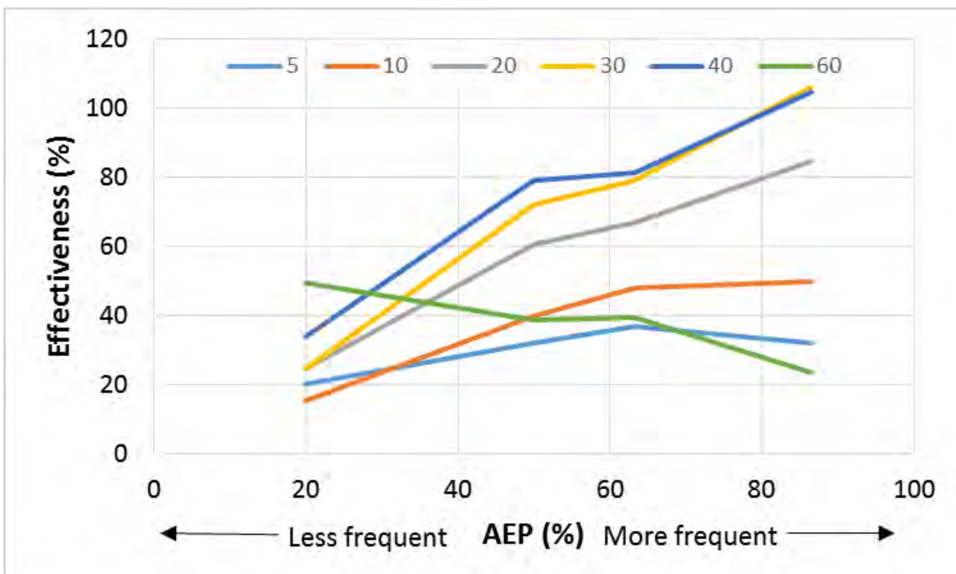


*Figure 30 – Comparison of the **percentage effectiveness of 5 kL detention tanks** (with varying emptying time) to maintain peak flow rates in the Frederick Street catchment with respect to flow frequency, where high AEP indicates lower intensity rainfall and low AEP indicates high intensity rainfall.*

The results indicate that the peak flow rate produced by lower AEP flows produced from lower frequency, higher intensity rainfall are less influenced by WSUD than higher AEP flows produced by high frequency, lower intensity rainfall.

Overall, the results for the impact of WSUD on peak flows from WSUD and infill are similar. The results indicate that infill has a greater influence on more frequent, lower intensity rainfall runoff peak flows, and that the ability of WSUD to maintain these peak flows is also greater for these same rainfall events.

Flooding

The relative impact of higher and lower intensity rainfall events on the flooding at the nominated sag point in the Frederick Street catchment prior to and following infill development was previously shown in Table 12 (Page 46). This data is plotted in Figure 31.

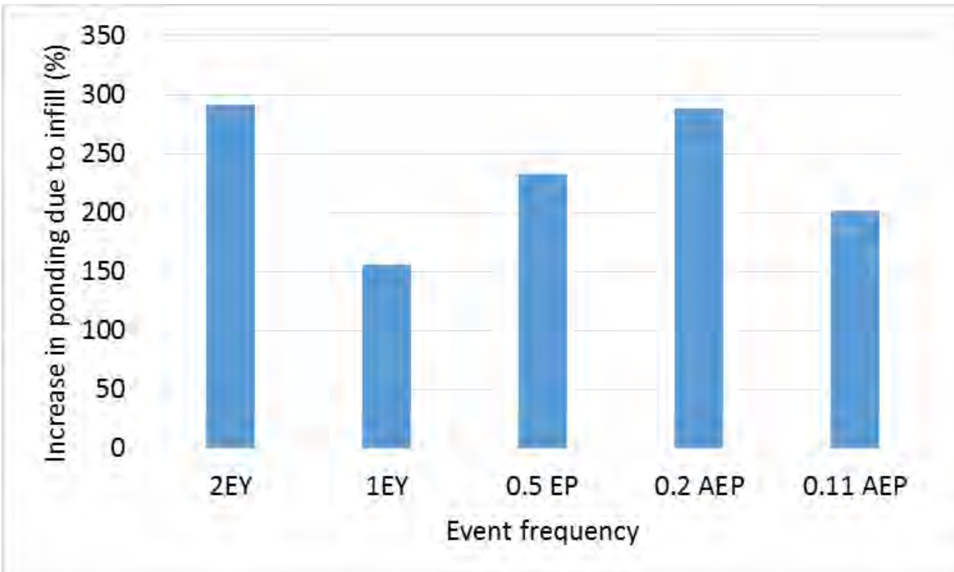


Figure 31 - Comparing the influence of runoff frequency (and rainfall intensity) on the percentage increase in flooding at one sag point of due to infill development in the Frederick Street catchment

The percentage increase data from this table and figure do not suggest a simple relationship exists between event magnitude and the level of ponding at the selected sag point. This may be because the level of ponding is subject to more influence than simply rainfall – for example, ponding may occur at other points upstream in more intense storms, which leads to a net increase in ponding, but this can either increase or reduce the volume of flooding at a single point of interest studied downstream. A better understanding might be achieved by investigating ponding throughout the catchment, or in even more detail, by investigating the extent of flooding in the catchment using a 2-dimensional modelling tool.

The relative impact of higher and lower intensity rainfall events on the ability of WSUD to preserve the ponding volume at the Fredrick Street sag point in catchments experiencing infill is compared for some more successful scenarios in Figure 32 (for the case of 5 kL lumped retention tanks) and Figure 33 (for the case of 5 kL lumped detention tanks). In each case, the plot compares the impact of the tank and it's emptying characteristics with respect to predicted flow frequency. The percentage effectiveness on the y axis in this case represents the percent to which the pre-infill ponding volume at the nominated sag pint was maintained by the WSUD measure – for example, 100% indicates that the pond volume was fully maintained, 50% indicates that the ponding volume was reduced to half way between the pre-infill ponding level and the post-infill ponding level (with no WSUD in place).

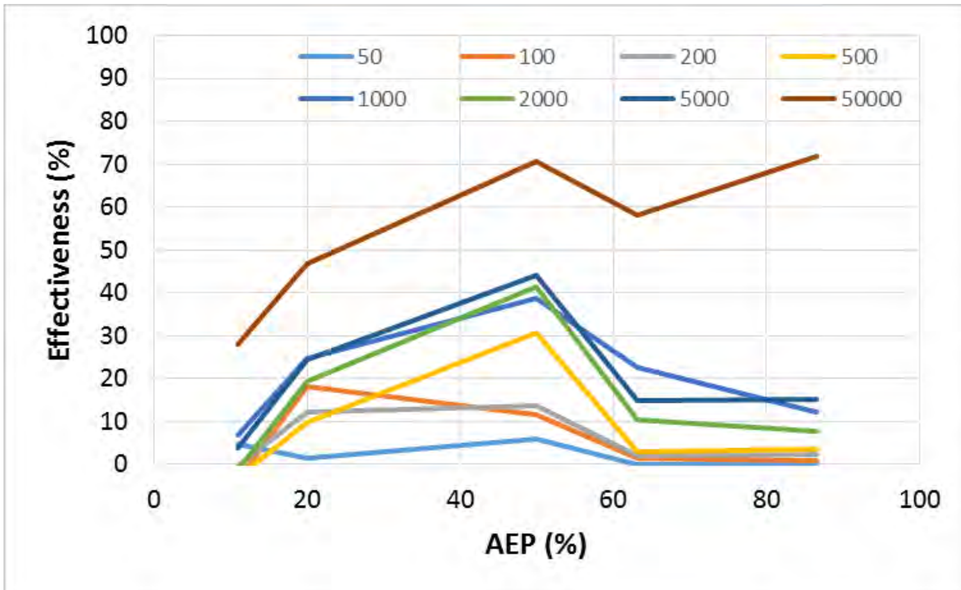


Figure 32 – Comparison of the percentage effectiveness of 5 kL retention tanks (with varying emptying time) to restore ponding volume at a sag point in the Frederick Street catchment with respect to ponding frequency, where a high AEP indicates lower intensity rainfall and low AEP indicates higher intensity rainfall.

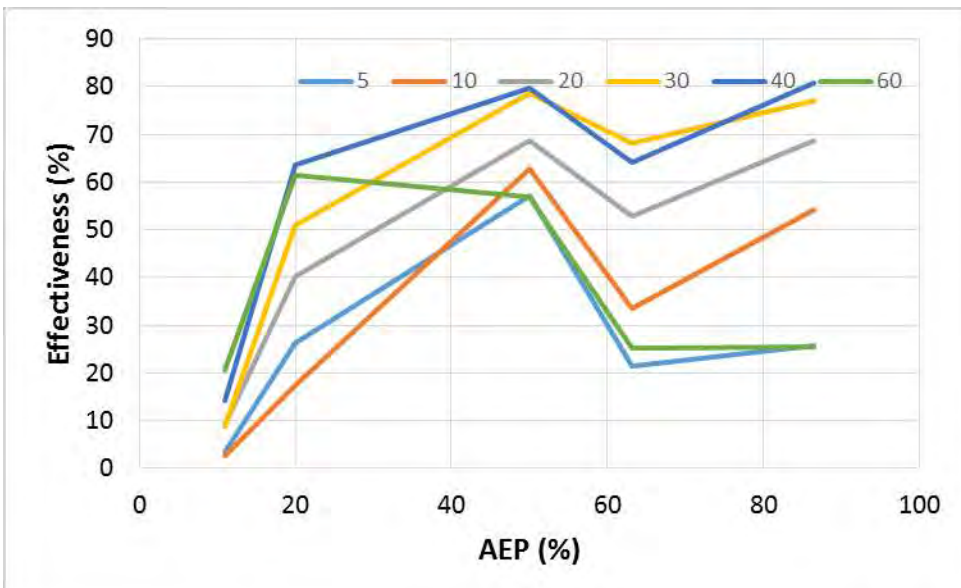


Figure 33 - Comparison of the percentage effectiveness of 5 kL detention tanks (with varying emptying time) to maintain ponding volume at a sag point in the Frederick Street catchment with respect to ponding frequency, where a high AEP indicates lower intensity rainfall and low AEP indicates higher intensity rainfall.

The results indicate that WSUD has a greater impact on more frequent ponding events (corresponding with more frequent, low intensity rainfall) than it does on the less frequent ponding events (corresponding with less frequent, high intensity rainfall. However the relationship is not as straightforward as was determined for flow rate in Figure 29 and Figure 30. This is likely due to the same reasons explained above for the effect of infill development on flooding; it is expected that ponding in other areas of the catchment makes the relationship more complex when assessing ponding volume.

Discussion and Summary

Key findings from the assessment of the impact of higher and lower intensity rainfall events on the runoff volume, peak flow rate and flooding in the Frederick Street catchment included:

- Infill development increased the runoff volume produced by individual events within the simulated time series, and the percentage increase in event runoff volume gradually diminished as the AEP of the event became less frequent.
- Similar findings were apparent for the individual event peak flow rates. For example, the peak flow rate of runoff increased by 36% for the more frequent 86% AEP flow rate, but by only 6 % for the less frequent 11% AEP events.
- The implementation of WSUD in the form of retention or detention had a greater impact on less frequent storm events. In other words, the events most affected by infill development are those which are more frequent, but these are also the events that can be more efficiently maintained by WSUD measures.
- Unlike runoff volume and peak flow rate, the percentage increase in the maximum flood volume at the selected assessment point of the Frederick Street catchment was not obviously related to event intensity.

Overall, these results suggest that smaller, more frequent flow events (from lower intensity rainfall) are strongly affected by infill development, but larger, less frequent flow events from high intensity rainfall are not strongly affected. This is likely because less frequent events begin to be influenced by pervious area runoff; when pervious areas contribute to the total runoff volume, the impact of additional connected impervious area is less apparent – runoff volume and peak flows may be similarly influenced by impervious area and saturated pervious area, with the exception that simulation tools typically assume a higher roughness for pervious areas, as was the case in this project.

It is recommended that future research look more closely at the relationship between WSUD effectiveness, rainfall intensity and flood volume throughout a catchment. In this project, flooding was examined using the level of ponding at one point in the catchment. The lack of a relationship with rainfall intensity and WSUD implementation may be because the level of ponding at a given point in a catchment is subject to more influence than simply rainfall intensity – for example, ponding may occur at other points upstream in more intense events, which leads to a net increase in ponding across the catchment, but this increase may or may not influence flooding at a single point of interest. A better understanding might be achieved by investigating ponding throughout the model, or in more detail, by investigating the extent of flooding throughout the catchment using a 2-dimensional modelling tool.

5.2.5 Case 5: Comparing the Impact of WSUD on catchments with Higher and Lower Infiltration Rate

The installation of WSUD in areas with lower and higher soil infiltration properties was explored as described in Section 4.2.5. It should be noted that the installation of WSUD in areas with different soil infiltration properties is only going to have an impact on runoff where infiltration based retention storages are employed, as soil properties will not affect the emptying rate of detention based WSUD, nor will infiltration rate affect retention storages which are dependent on water reuse demand.

Runoff Volume

An indication of the impact of infiltration rate on the runoff volume was previously illustrated in Figure 14 (Page 47) for the retention case when distributed throughout the catchment on redeveloped allotments, and in Figure 15 (Page 48) for the retention case when lumped storages are placed at the street scale. The results indicated that as the assumed infiltration rate increased, all retention solutions became more effective. For the distributed case (tanks on each redeveloped allotment), the annual runoff volume could not be maintained regardless of storage volume size or assumed infiltration rate. However, in the lumped tank case (lumped storages at the street scale), there were several design arrangements which were able to maintain the mean annual runoff volume. These cases have been highlighted in Table 17. Overall, these results illustrate the importance of applying enough demand to, or allowing for enough infiltration to occur from a storage to ensure it is empty enough when rain occurs to retain runoff volumes.

*Table 17 – Mean annual runoff volume (ML/year) from **lumped retention** based WSUD scenarios. Scenarios which maintained the mean annual runoff volume are highlighted.*

	Pre-infill mean annual runoff volume (target)				66	ML/year	
	Post-infill mean annual runoff volume				102	ML/year	
Demand (L/day)	**50**	**100**	**200**	**500**	**1000**	**2000**	**5000**	**50000**
2 kL	98.27	95.96	92.85	88.12	84.04	86.08	88.12	33.09
3 kL	98.00	95.42	91.58	85.15	80.08	72.40	64.73	28.90
4 kL	97.85	95.08	90.73	82.96	76.85	68.60	60.35	25.80
5 kL	97.69	94.77	90.12	81.27	69.85	63.29	56.73	23.44
6 kL	97.54	94.54	89.65	79.92	71.81	62.67	53.54	21.47
7 kL	97.35	94.35	89.27	78.88	71.81	61.27	50.73	19.81
8 kL	97.23	94.19	88.92	77.96	68.15	58.19	48.23	18.40
9 kL	97.12	94.00	88.65	77.23	66.69	56.33	45.96	17.18
10 kL	96.92	93.88	88.35	76.58	65.38	54.65	43.92	16.12

Peak Flow Rate

An indication of the impact of infiltration rate on the peak flow rate occurring from the catchment was previously illustrated in Table 13 (Page 49), Figure 16 (Page 50) for the 50% AEP runoff flow rate estimate with retention at the allotment scale and Figure 18 (Page 51) for the 50% AEP runoff flow rate estimate with retention in the form of lumped storages placed at the street scale. The results indicated again that increasing the rate of infiltration (or demand) improved the ability of the retention system to reduce catchment peak flows. Like the case for runoff volume, installing retention systems on the redeveloped allotment was not able to maintain runoff peak flow rates, however lumped versions of equivalent storage volume at the street scale (with an increased contributing impervious area) were able to maintain peak flow rates in some cases. These cases have been highlighted in Table 18. It should be noted that these cases are specific to the 50% AEP runoff flow rate estimate, and that only the very highest demand rate was able to achieve the target. Furthermore, there were no cases where the runoff flow rate could be maintained for higher less frequent AEPs (e.g. 20% or lower in the scenarios examined) despite the generous rate of demand (or infiltration) of 50 000 L/day.

*Table 18 – **50% AEP peak runoff flow rate estimate** for the Frederick Street catchment (m^3/s) from **lumped retention** based WSUD scenarios. Scenarios which maintained the peak flow rate are highlighted.*

	Pre-infill peak flow rate				1.11	m^3/s		
	Post-infill peak flow rate				1.49	m^3/s		
Demand (L/day)	**50**	**100**	**200**	**500**	**1000**	**2000**	**5000**	**50000**
2 kL	1.49	1.48	1.48	1.48	1.48	1.48	1.48	1.39
3 kL	1.48	1.48	1.48	1.48	1.48	1.46	1.45	1.35
4 kL	1.48	1.48	1.47	1.47	1.45	1.43	1.41	1.33
5 kL	1.48	1.48	1.45	1.42	1.39	1.39	1.39	1.27
6 kL	1.48	1.48	1.45	1.41	1.40	1.38	1.36	1.21
7 kL	1.48	1.48	1.45	1.40	1.40	1.36	1.32	1.14
8 kL	1.48	1.49	1.45	1.40	1.36	1.32	1.28	1.08
9 kL	1.48	1.48	1.42	1.40	1.34	1.30	1.26	1.06
10 kL	1.48	1.47	1.42	1.40	1.35	1.29	1.24	1.01

Flooding

An indication of the impact of infiltration rate on the ponding volume at the Frederick Street catchment sag point was previously illustrated in Figure 20 (Page 52) for the 50% AEP ponding volume estimate with retention at the allotment scale and Figure 22 (Page 53) for the 50% AEP ponding volume estimate with retention in the form of lumped storages placed at the street scale. The results indicated again that increasing the rate of infiltration (or demand) improved the ability of the retention system to reduce the flooding impact in the catchment. In the illustrated cases, there was no retention demand (or tank size) that was capable of fully restoring the ponding volume that occurred with a 50% AEP in the catchment without infill.

Discussion and Summary

A comparison of the impact of implementing WSUD on catchments with higher and lower infiltration rate was relevant only to retention based measures which allow for infiltration. The results of this comparison were presented in Section 5.2.5. The key outcomes of the analysis were:

- As the assumed infiltration rate of soil increases (as was reflected by demand in L/d) allotment based and lumped scale retention both become more effective at reducing runoff volume. Lumped retention measures represent a more effective means of restoring runoff volumes to the levels prior to redevelopment. Generally speaking, storages which could infiltrate greater than 2000 L/d and were sized to consist of at least 5 kL volume per redeveloped allotment could maintain the mean annual runoff volume.
- Increasing the rate of infiltration (or demand) improved the ability of the retention system to reduce catchment peak flows. As in the case of runoff volume, allotment based retention could not maintain the catchment peak flow rates in the range of assumed parameters of this project, but lumped retention was more effective. However, unlike the runoff volume case, catchment peak flow rates could only be maintained with the very highest assumed demand (50 000 L/d) and for the larger tanks (minimum 8 kL per redeveloped allotment in the contributing subcatchment area)

- Increasing infiltration rates also led to reduced flood volumes at the nominal assessment point of the Frederick Street catchment. In this case however, neither the allotment based nor lumped retention systems, regardless of infiltration rate and tank size, could maintain the assessment point flood volumes back to levels estimated prior to redevelopment.

5.2.6 Case 6: Comparing the Effect of Assessing WSUD Impact in Smaller to Larger Catchment Scales

The methodology used to explore the impact of infill and the restorative effect of WSUD on the runoff flow rate and peak flow rate occurring from different catchment sizes was described in Section 4.2.6.

Runoff Volume

An indication of the impact of infill development on the mean annual runoff volume from the allotment street and catchment scale is compared in Table 19.

Table 19 – The impact of infill development on mean annual runoff volume at the allotment, street and catchment (Frederick Street) scale

Scenario	Allotment (ML/year)	Street (ML/year)	Catchment (ML/year)
Original	0.08	2.09	65.7
Redeveloped	0.28	4.10	101.5
% increase	255	96	54

The results in Table 19 indicate that the mean annual runoff volume by analysing the implementation of infill at different scales produces a disproportionate estimate of the increase in runoff volume. This is because the street and catchment scale simulation have considered a greater proportion of impervious area as fixed – such as roads, paths and allotments not subject to redevelopment – which are not present in the allotment scale.

The impact of WSUD on the predicted mean annual runoff volume when simulated at the allotment, street and catchment scale are compared for the 50% AEP event in Figure 34 (retention scenarios). Results for detention scenarios are not compared because detention is not intended to influence the mean annual runoff volume.

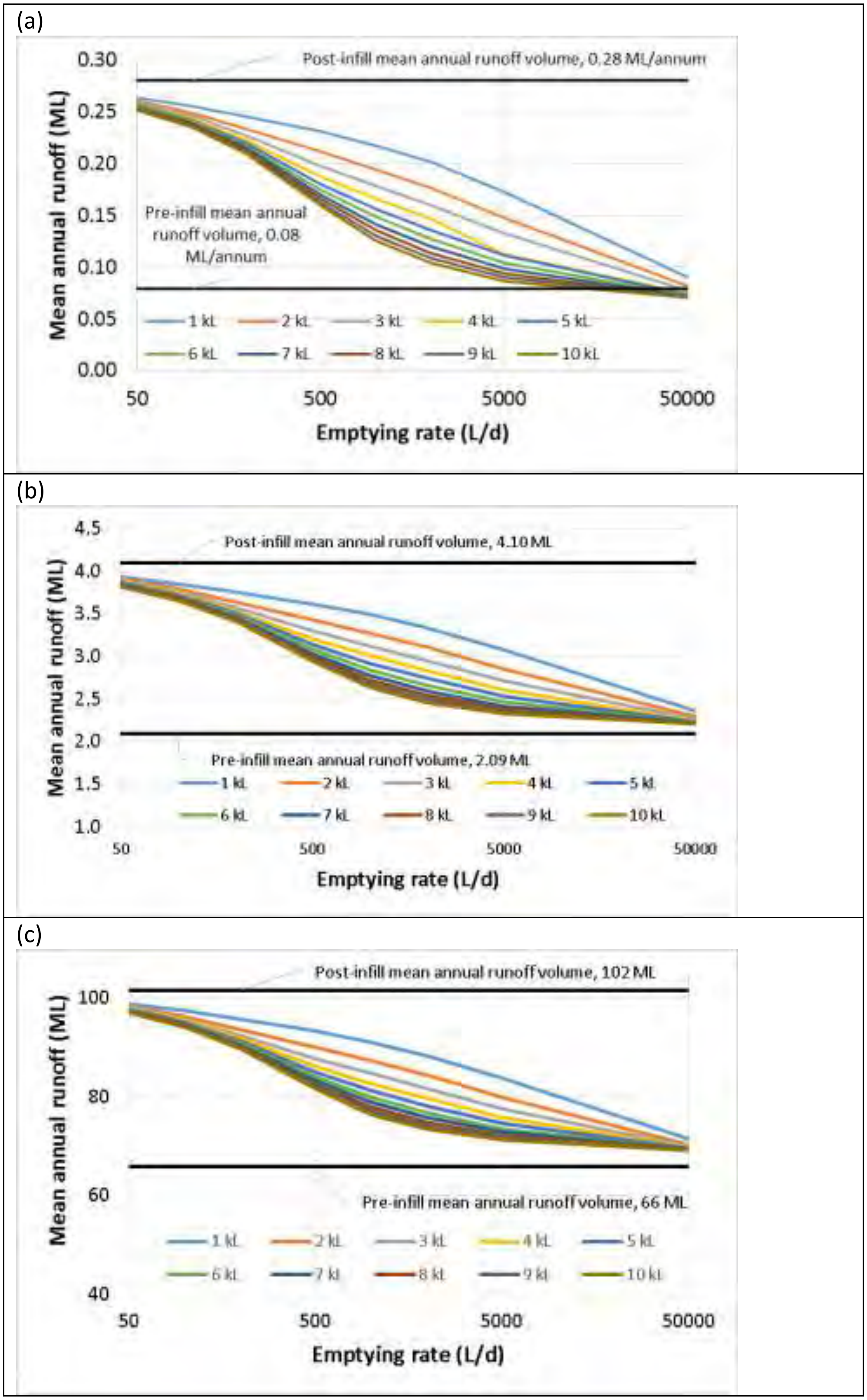


Figure 34 – Comparison of the influence of on-site retention on mean annual runoff volume when simulation of a redeveloped site is undertaken at the (a) allotment (b) street and (c) catchment scale.

The results showed that there was little difference in the predicted ability of WSUD in the form of on-site retention to conserve the mean annual runoff volume whether it be assessed at the allotment, street or catchment scale.

Peak Flow Rate

The impact of infill development when it was assessed at the allotment, street and catchment scale are presented in Table 20.

Table 20 – Summary of the estimated impact of infill development on peak flow rates based on simulation of infill at the allotment, street and catchment scale

Catchment scenario		**86% AEP**	**63% AEP**	**50% AEP**	**20% AEP**	**11% AEP**
Allotment	Existing (L/s)	1.9	2.2	2.5	4.1	6.2
	Redeveloped (L/s)	6.5	7.9	8.6	11.3	13.7
	% increase	*243*	*255*	*250*	*178*	*121*
Street	Existing (L/s)	48.0	58.7	64.4	103.5	117.3
	Redeveloped (L/s)	93.2	112.9	124.0	180.5	213.2
	% increase	*94*	*92*	*93*	*74*	*82*
Catchment	Existing (m^3/s)	0.84	1.00	1.11	1.53	1.77
	Redeveloped (m^3/s)	1.14	1.36	1.49	1.81	1.89
	% increase	*36*	*37*	*34*	*18*	*6*

The results for peak flow rate are similar to those found for the mean annual runoff estimation. The results indicate that as the scale of the simulated catchment grew, the predicted effect of infill development, as indicated by the percentage increase in peak flow rates, diminished.

The impact of WSUD on the predicted peak flow rate when simulated at the allotment, street and catchment scale are compared for the 50% AEP event in Figure 34 (retention scenarios).

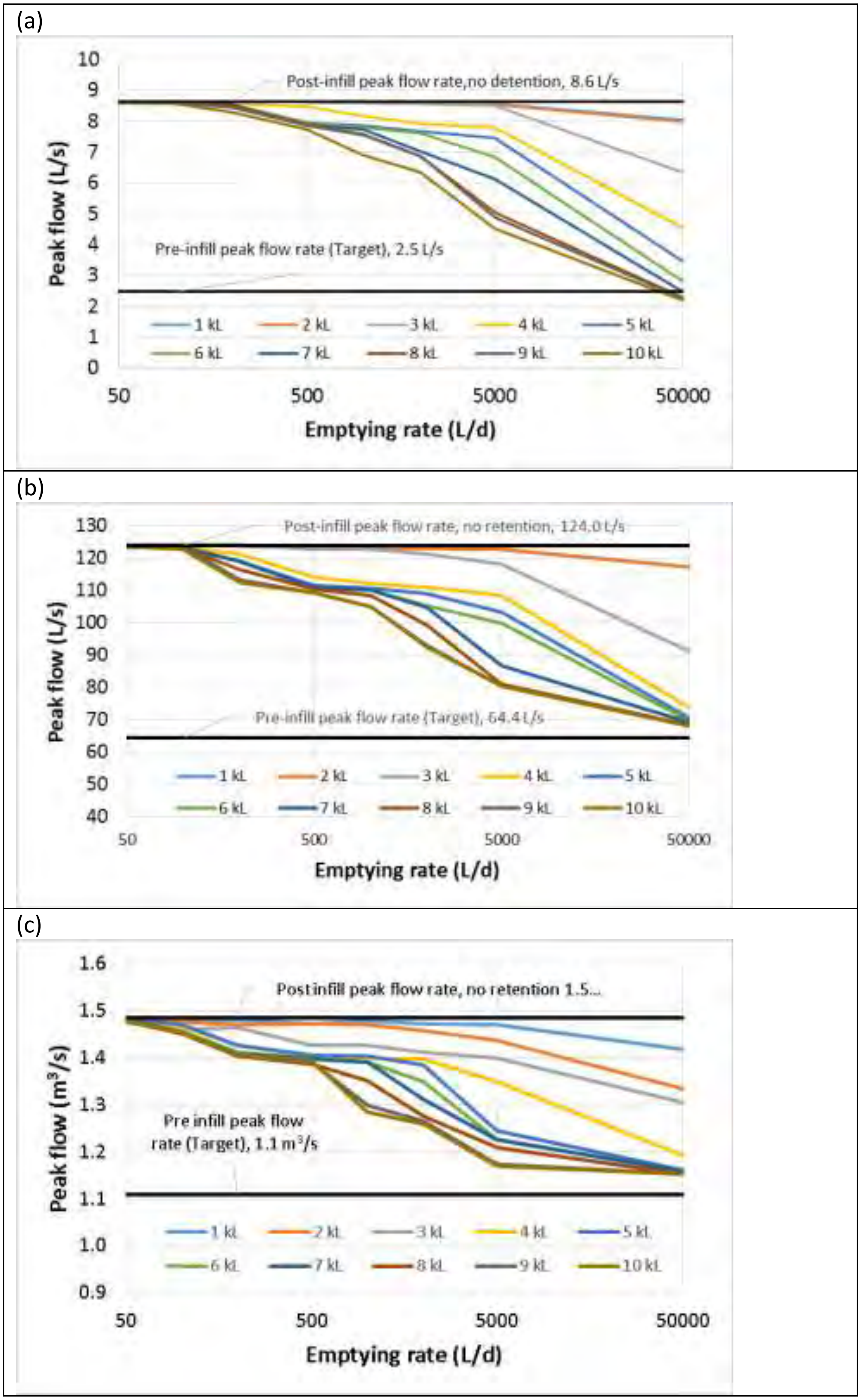


Figure 35 – Comparison of the influence of retention on the 50% AEP peak flow rate from a catchment when simulation of a redeveloped site is undertaken at the (a) allotment (b) street and (c) catchment scale.

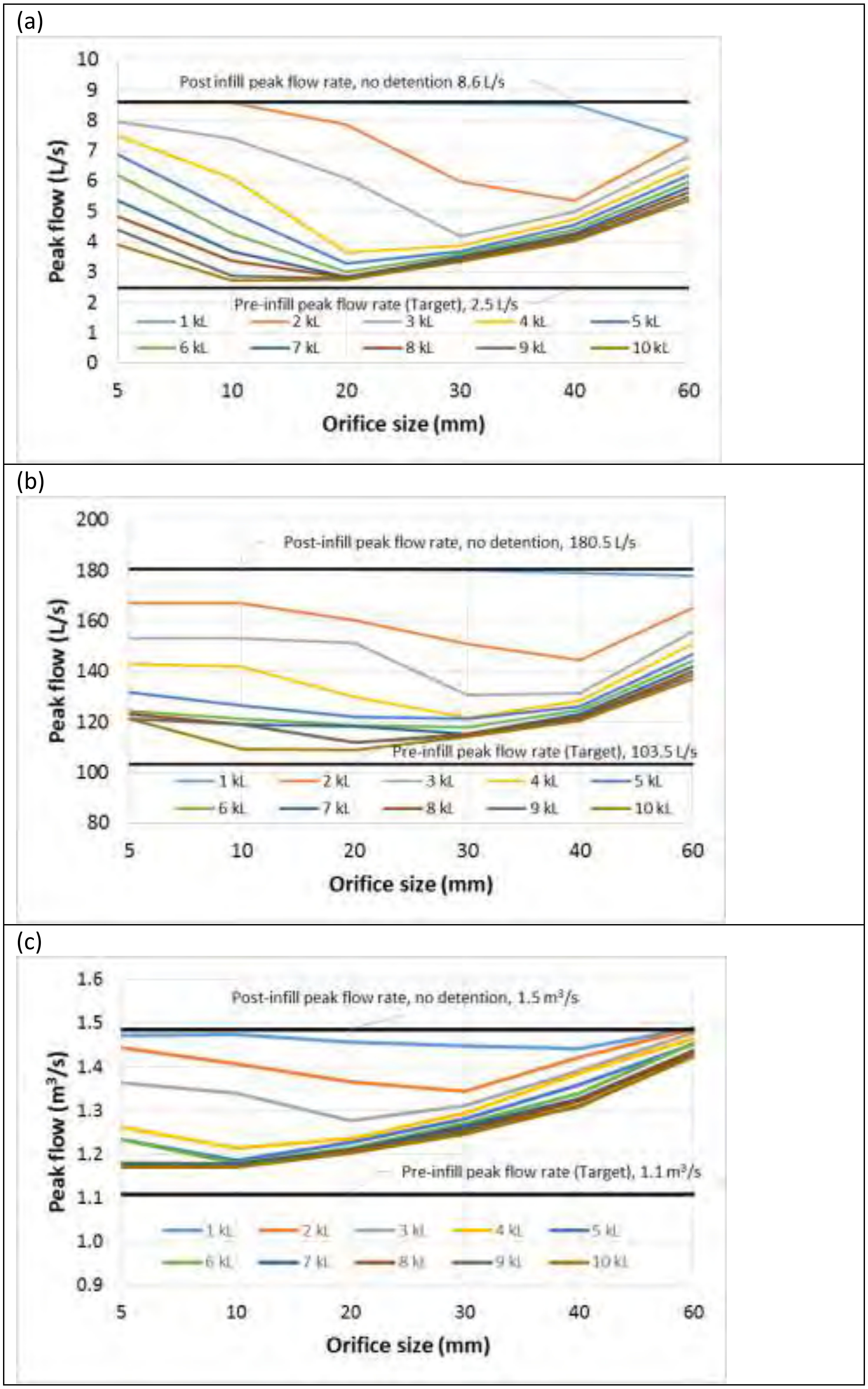


Figure 36 – Comparison of the influence of detention on the 50% AEP peak flow rate from a catchment when simulation of a redeveloped site is undertaken at the (a) allotment (b) street and (c) catchment scale.

The results of the simulation at different scales for both retention and detention indicate that the simulated WSUD is more likely to achieve the target pre-infill peak flow rate when simulated at the allotment scale than at the larger scale. Retention scenarios still improved with greater reuse, however the required orifice size (or emptying time) of detention systems appears to change when larger catchments are simulated. For example, the best orifice size for a 5 kL tank at the allotment scale is approximately 20 mm at the allotment and street scale, however this clearly changes to become closer to 10 mm when simulating the catchment scenario.

Discussion and Summary

The effect of implementing and assessing WSUD in catchments of varying size was examined by comparing the runoff volume and peak flow rate following redevelopment of a typical allotment, a typical street of allotments and the Frederick Street catchment. The results were presented in Section 5.2.6. The key results were as follows:

- The percentage increase in mean annual runoff volume due to infill development reduced as the catchment area increased in this study.
- The impact of WSUD on the mean annual runoff volume was broadly similar at every scale, indicating that the ability of retention based WSUD measures to maintain mean annual runoff volume is assessed fairly when the allotment, street or catchment scale are considered in isolation of other scales (using the approaches in this report).
- The results indicate that as the scale of the simulated catchment grew, the predicted effect of infill development, as indicated by the percentage increase in peak flow rates, diminished.
- This trend was not apparent when WSUD implementation was simulated on these same catchment scales. Estimation of peak flow rates following the implementation of retention and detention indicated that WSUD was not clearly more or less beneficial when simulated at the allotment scale than at the larger scale. Relationships for retention remain generally consistent and improved with greater reuse. However the required orifice size (or emptying time) of detention systems changed according to the size of the catchment being considered to control peak flow rates. For example, the best orifice size for a 5 kL detention tank on an allotment was 20 mm at the allotment and street scale, however this was 10 mm when simulating the catchment scenario.

The disproportionate increase in the runoff volume and peak flow rate with respect to catchment scale can be attributed to multiple factors. First, the runoff volume and peak flow rate from an allotment can be expected to increase by a greater amount because the percentage change in impervious area is proportionally greater for an allotment compared to a street or catchment. For example in larger catchment areas, as lot by lot infill gradually progresses, the impervious area attributable to the public road reserve, existing redeveloped homes and homes on allotments too small to redevelop remains fixed, which is not the case when assessment of runoff volume and peak flows is considered for the development parcel only. Furthermore, some of this increased runoff may be attributable to modelling assumptions – the single allotment scenario involved redevelopment of the whole catchment, the street scenario involved redevelopment of 1 in every 2 allotments, while the Frederick Street catchment involved redevelopment of 1 in 2 allotments where redevelopment was feasible. As such, further research is required to explore the impact of catchment scale on runoff volume in a more consistent manner.

It should also be recognised that the results presented are for a full residential catchment. Future research would be useful to explore whether the impacts of catchment scale on the estimated effectiveness of WSUD change for different land use due to the difference in the layout of impervious areas.

5.2.7 Comparison of the Cost Effectiveness of Applied WSUD Systems

The costs of applying WSUD solutions for retention and detention were assessed using the approaches summarised in Section 4.2.14. The final costing for each option is summarised in Table 21.

Table 21 – Estimated cost of WSUD solutions compared to a complete upgrade of the stormwater network with a dual pipe

Name	Description	Costs ($ in 2016)		
Option 1	Stormwater upgrade works – 'dual pipe' including 2 km pipe and 78 pit/junctions	*892,000		
WSUD Volume per redeveloped allotment		**2 kL**	**5 kL**	**10 kL**
Option 2	Allotment rainwater tanks (low use retention)	598,000	682,000	804,000
Option 3	Allotment soakaways (low to medium use retention)	338,000	-	463,000
Option 4	Allotment detention tanks	260,000	339,000	*469,000
Option 5	Lumped detention in street, large pipes	360,000	*877,000	*1,740,000
'*' indicates that this measure may be considered successful at returning peak flows and flooding to levels prior to redevelopment				

Discussion and Summary

The results of the cost comparison indicated the following:

- Allotment scale WSUD can go some way to alleviating the need for a stormwater system upgrade based on previous analysis, and the cost estimate for these systems was always lower than that estimated for a conventional upgrade of the entire stormwater pipe upgrade
- The cheapest and most effective means to reduce peak flow rates was detention systems at the allotment, however it should be noted that these make no contribution to reducing runoff volume.
- The use of enlarged sections of concrete pipe at each stormwater pit to provide street scale detention was the most expensive means considered to minimise peak flows and flooding at the catchment scale

It should be noted however that the costing excluded key activities which were explicitly listed in Section 4.2.14. Key among these assumptions was the exclusion of design, construction supervision and approvals costs. While the design and approval efforts of a stormwater upgrade system may be expected to be significant, the cumulative design and approval efforts for distributed systems on redeveloped allotments is also expected to be significant and reliable costing requires estimation of hours spent by consulting engineering, local government engineers and local government or private certifiers.

A further limitation on the study findings is that the requirements of detention are likely to vary based on the scale of the catchment considered. This outcome was to some extent evident based on the results for WSUD effectiveness at different catchment scale presented in Section 5.2.6. The required orifice size in this case changed in the order of 10 mm to achieve optimal results when comparing the allotment scale (730 m^2) and catchment scale (approximately 45 Ha). The impacts for a much broader catchment should also be considered.

It is also noted that targeted WSUD may be rolled out in certain catchment locations where flooding and peak flows are having greater impact. While this may be cheaper, it is not in line with the current development policy agenda which is targeting a more uniform development policy across the state for example, the SA government acknowledged support for Reform 7 of the Expert Panel on Planning Reform (Government of South Australia, 2015).

5.2.8 Additional Constraints on WSUD Implementation Assumed in this Research

The implementation of WSUD is subjected to a number of constraints which have not been fully explored as part of this research, but which are noted here for the benefit of practitioners and future researchers. Firstly, it is noted that detention performed better than retention for the management of peak flows in this study. However, it should be noted that the simulation and cost scenarios assumed that detention tanks drained via gravity to the street. This, in effect, means that detention tanks must be located at the surface, which limits the connected impervious area, and the limitation of connected impervious area further limits the performance of retention or detention based WSUD (Myers et al., 2014).

The ability of on site or street scale measures such as biofilters or subsurface storages on the allotment to drain runoff into street drainage is also a significant constraint that must be considered. The performance of storage based systems for mitigation of peak flows is strongly dependent on a emptying the storage in a relatively ‘short’ period via slow release to the street system or infiltration to the soil on-site. If the storage is below the street gutter level it is not possible (without a pump and source of power) to drain storages to the street. Natural (gravity) discharge to the street system requires the invert of the storage system to be above the gutter invert. This would be possible if the site was substantially elevated, but this is not typical in central to west Adelaide or across the northern plains. Alternatively, if a stormwater pipe exists in front of the allotment, a direct connection to the pipe is technically possible but may be prohibitively expensive. In this report, the number of allotments which have access to a subsurface drainage pipe in the Frederick Street catchment (part of the South Western suburbs drainage scheme) and the Paddocks were counted. It was found that in the Frederick Street catchment, 18 % of homes were adjacent to a road with a subsurface drain. In the Paddocks, 24% of homes were built adjacent to a road with a subsurface pipe. This indicates that for these catchments, the ability of allotment based measures or street scale measures like biofilters to be implemented is limited to 18% and 24% of the catchment. Implementation of these measures elsewhere will require additional pipe drainage infrastructure at the street scale, on site infiltration or restriction to measures which can dispose of treated/detained inflow gutter drainage.

An additional constraint on implementing the WSUD system at have been proposed in this research are policy based. Many local governments apply conditions on how stormwater is managed and disposed during development, including infill, at the allotment level. These conditions, such as clearance distances and drainage to the street gutter, can restrict the ability to apply WSUD measures. In the case of onsite disposal via infiltration to the soil, council conditions prevent infiltration systems (also known as soakaways, raingardens or infiltration measures) being constructed within a specified distance of building and allotment boundaries. For example, the City of Charles Sturt and Minister's Specification SA 78AA (2003) for On-Site Retention requires a setback distance of 3.0 m from buildings and boundaries. To meet this requirement this would need a minimum open space distance of 6.0 m plus the size of the soakaway. As infill consumes allotment area with new roof and paved area, many redeveloped allotments are small and have less than 6 m in setback distance at the front or the rear of the allotment.

5.3 Comparison of 'design' storm approach with continuous simulation to assess flood benefits by WSUD measures

5.3.1 Direct Comparison of the Output from Applying a Design Storm Simulation and Continuous Simulation Approaches

The following sections compare the results of using a design storm simulation and continuous simulation to assess the performance of equivalent retention and detention based WSUD storages at the allotment and catchment scale.

Allotment Scale

The results in Figure 37 compare the peak flow rate at the outlet of an allotment prior to infill, post infill, and post infill with retention based WSUD. The results are specific to peak flows from either the 50% AEP storm (in the design case) or the 50% AEP flow rate (from the continuous runoff record). The results in Figure 41 shows the same comparison for a detention based WSUD case.

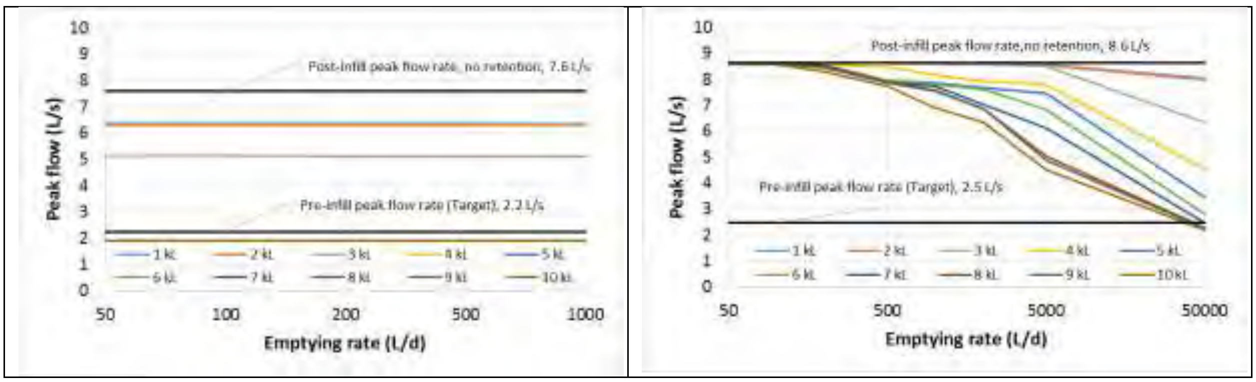


*Figure 37 – The 50% AEP peak flow rate from a pre infill, post infill and post-infill with **retention based WSUD at the home allotment**, comparing the results of estimation using a 50% AEP design storm (left) and estimation based on partial series analysis of the continuous flow records*

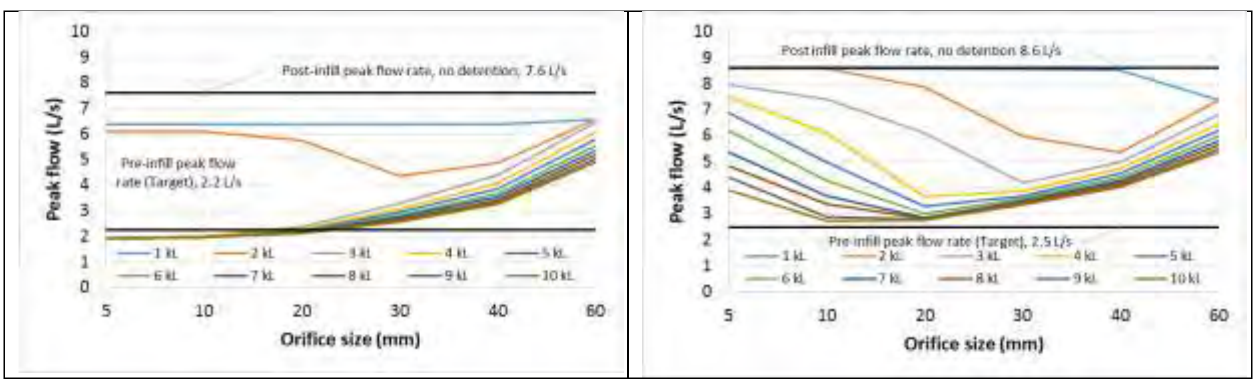


*Figure 38 – The 50% AEP peak flow rate from a pre infill, post infill and post-infill with **detention based WSUD at the home allotment**, comparing the results of estimation using a 50% AEP design storm (left) and estimation based on partial series analysis of the continuous flow records*

The results in Figure 37 show that when retention is assumed to be empty, then all tanks equal to or greater than 4 kL in volume were effective at maintaining peak flow rates when infill occurred on the allotment. However, consideration of continuous rainfall and the related filling and emptying of allotment retention tanks indicates that the capacity of retention reduces considerably. The results in Figure 41 are similar – in this case, all tanks equal to or greater than 3kL could restore peak flow rates with a suitable orifice size when the design storm technique was applied, however success was limited when continuous simulation was used to estimate the effectiveness of detention.

Note also that the peak flow rate of the undeveloped catchment was different in the design storm case (2.2 L/s) and in the continuous case (2.5 L/s), and similarly for the post development peak flow rate. This is because the basic assumption of each design approach is related, but slightly different. In the design storm approach, the peak flow is estimated as that which occurs when a design storm is simulated over the catchment. In the continuous simulation case, the peak flow estimate is the peak flow rate which occurs with a 50% AEP. The reader is reminded that a 50% AEP design storm does not necessarily produce a 50% AEP peak flow rate, and nor does a 50% AEP peak flow rate always occur in relation to a 50% AEP storm – the nature of a catchment dictates the frequency of runoff for a given storm.

Catchment Scale

The results in Figure 39 compare the peak flow rate at the outlet of an allotment prior to infill, post infill, and post infill with retention based WSUD. The results are specific to peak flows from either the 50% AEP storm (in the design case) or the 50% AEP flow rate (from the continuous runoff record). The results in Figure 40 shows the same comparison for a detention based WSUD case.

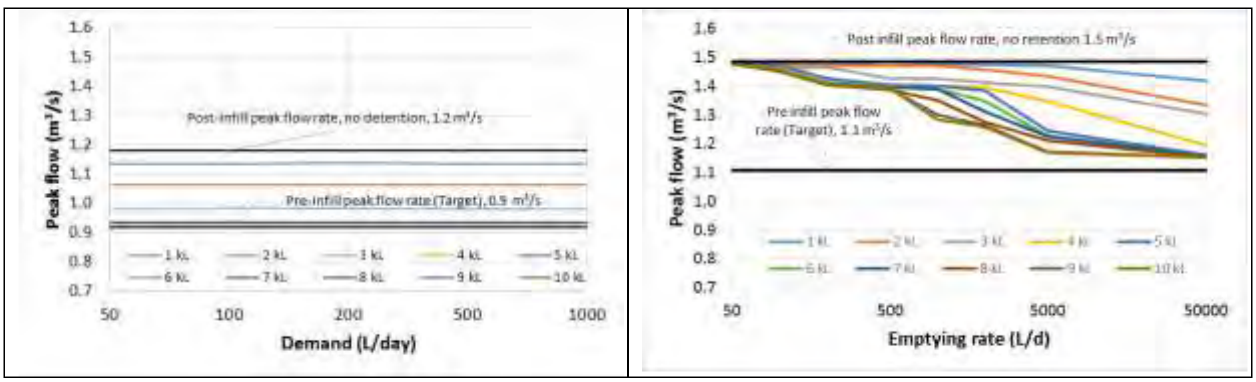


*Figure 39 – The 50% AEP peak flow rate from a pre infill, post infill and post-infill with **retention based WSUD at the home allotment**, comparing the results of estimation using a 50% AEP design storm (left) and estimation based on partial series analysis of the continuous flow records*

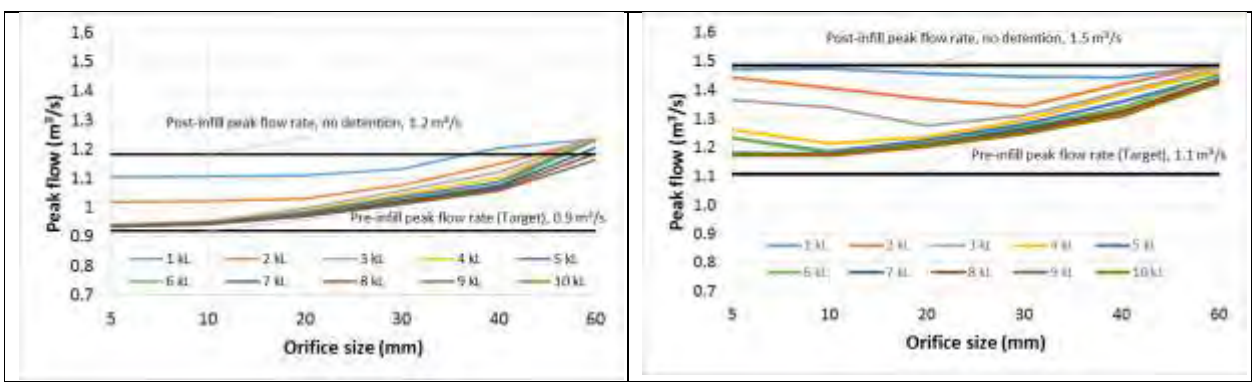


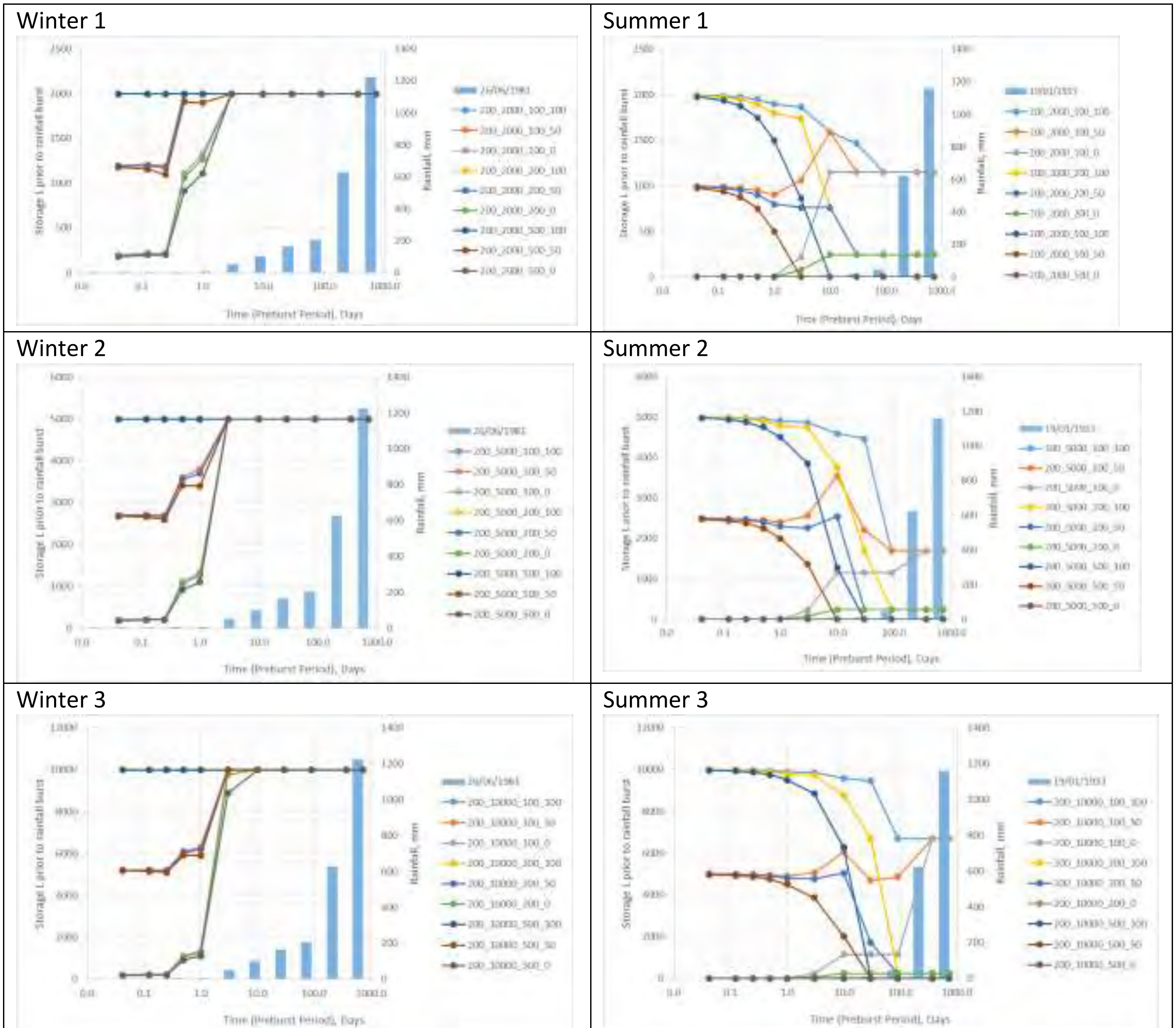
*Figure 40 – The 50% AEP peak flow rate from a pre infill, post infill and post-infill with **detention based WSUD at the home allotment**, comparing the results of estimation using a 50% AEP design storm (left) and estimation based on partial series analysis of the continuous flow records*

The results in Figure 39 show that when retention is assumed to be empty, then all tanks equal to or greater than 4 kL in volume were almost effective at maintaining peak flow rates when infill occurred at the catchment scale. However, consideration of continuous rainfall and the related filling and emptying of allotment retention tanks indicates that the capacity of retention reduced. The results in Figure 40 were different – detention scenarios needed a small orifice (5 mm) to be as effective as retention, but were still less effective for the continuously simulated case. Again, however, the peak flow rate of the pre-infill catchment was different in the design storm case (0.9 m^3/s) and in the continuous case (1.1 m^3/s), and similarly different for the post development peak flow rate. The reader is reminded that a 50% AEP design storm does not necessarily produce a 50% AEP peak flow rate for a given catchment; nor does a 50% AEP peak flow rate always occur in relation to a 50% AEP storm – the nature of a catchment dictates the frequency of runoff for a given storm.

Regardless of the difference in how the pre-infill and post-infill peak flow rates were calculated, these results emphasise the need to consider the storage conditions of any storage based WSUD feature prior to an event occurring before applying the measure for flow management. Ignoring this by adopting a blanket design approach may lead to inadequate design – the assumption of available storage where there is in fact no storage. The following section expands on this research by attempting to estimate, on an average basis, how much storage can be expected to be available for the Greater Adelaide case.

5.3.2 Pre-Burst Rainfall Analysis

The methodology used to determine the status of a typical storage tank attached to 200 m^2 of impervious area at an allotment was presented in Section 4.3.2. The results in Figure 41 show examples of the depth in the storage immediately prior to the beginning of a 20% AEP storm with approximately 1 hour duration. Cases are shown for a selection of winter and summer events.



Legend code: impervious area (m²)_Storage size (L)_Discharge rate (L/day) _ Initial storage condition (% full)

Figure 41 – Demonstration of tank storage immediately prior to the occurrence of a storm at in Adelaide based on observed storms with a 20% AEP and duration close to one hour. Storage conditions are shown for winter storms (left)and summer storms (right).

It can be seen in Figure 41 that the antecedent storage condition prior to the storm burst was full for the winter event (for all scenarios) when pre-burst periods greater than one day were examined. For the summer events the storage was part full depending up to pre-burst period of 1 year. As expected there was more water in the storages in cases where the discharge rate were low and the storage was assumed to be empty at the commencement of the pre-burst period. A review of the analysis of each event showed that a pre-burst period of at least 12 months would be required to correctly determine the storage condition prior to the burst. This observation suggests the adaption of a design pre-burst rainfall event with the design burst event is unlikely to be feasible, using design tools used by the profession (i.e. the DRAINS model). An alternative approach is to develop a relationship between storage, discharge and catchment area.

Analysis of the 12 events was carried out on the results where the storage was assumed to be full at the commencement of the 2 years for pre-bust rainfall. A mean of the pre-burst storage conditions from the 12 was determine for each scenario and is presented in Figure 42. The available storage in strongly dependent on the discharge rate. Interestingly, there is an opposite trend between the smaller (2 and 5 kL) and the large storages for the higher discharge conditions.

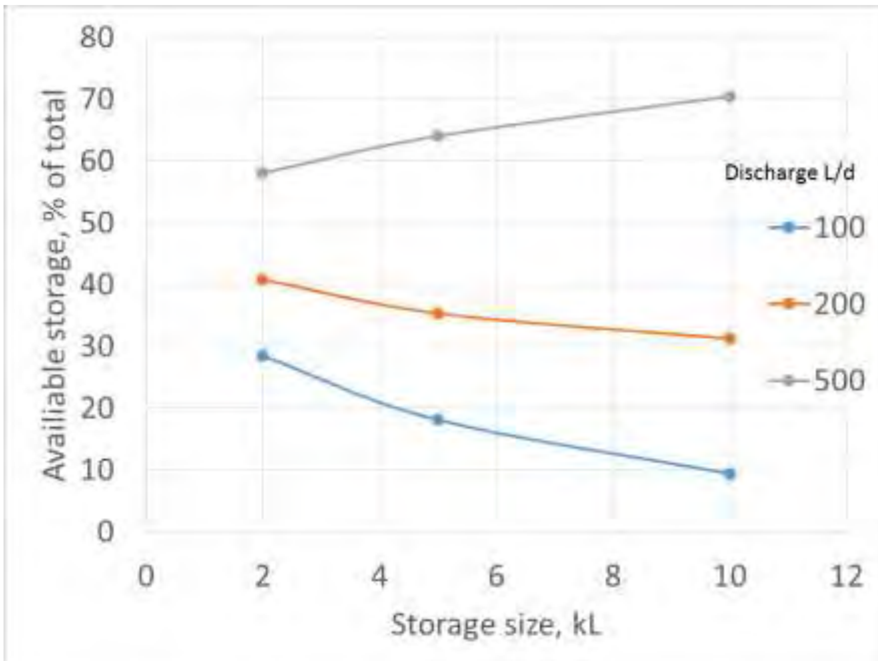


Figure 42 –Mean available storage pre-burst for each tank size examined

6. Conclusions and Recommendations

The project was proposed with three broad goals:

1. To collect evidence to quantify the impact of infill development on runoff flow rates and volumes in an urban catchment.
2. To identify and present a robust approach to compare the technical justification and cost of maintaining drainage system capacity using either a technically justified WSUD based policy measure or a conventional drainage system upgrade.
3. To compare the effectiveness of the 'design storm' approach for assessing WSUD based systems with storage with a continuous simulation approach.

The projects findings and recommendations are summarised with respect to each goal below.

6.1 The impact of infill development on runoff flow rates and volumes in an urban catchment

The project successfully used observed data and simulation tools to quantify the impact of infill development on runoff flow rates and volumes in an urban catchment. Using the Frederick Street catchment, City of Marion, as a case study, analysis of aerial photography revealed that infill development is occurring and impervious area is increasing - between 1966 and 2013 (47 years), the total pervious area decreased from 61.5% to 39.9% of the catchment area, a 35% reduction. Over the same period, the total catchment area covered by home roofing increased from 13.6% to 24.9%, an 83% increase on that in 1966. Gauged rainfall and flow records were compared in 1992/1993 and 2013/2014. This indicated that over the 20 year period, the volumetric runoff coefficient of the catchment increased form 24.4% to 31.2%, producing a 10% increase in runoff volume per event. An event runoff model, ILSAX, was then used to explore the impact of the observed increase in imperviousness between 1993 and 2013 on peak flow rates. It was found that the 15% increase in impervious area measured over this timeframe led to a corresponding increase in the model directly connected impervious area of 15% to meet the more recently observed peak flow rates. The progress of infill development is expected to continue in light of the state government shift in focus from developing the urban fringe to denser urban living.

Recommendations: The evidence presented in this report emphasises the need to consider the adoption of tools or policy to reduce the impact of ongoing infill development on peak flow rate and runoff volume in catchments where it is occurring or expected to occur. Alternately, catchment managers and the broader community must accept the environmental consequences of increasing runoff volumes to receiving waters, and the ongoing reduced capacity and standard of the minor drainage system in residential areas.

6.2 Technical Justification and Cost of WSUD as a Means to Manage Runoff

The effectiveness of WSUD to manage increased flow due to infill development was explored using simulation tools to identify runoff characteristics of the Frederick Street catchment, City of Marion with ongoing infill and implementation of WSUD to manage increased flow rates. Where required, other catchments in the Adelaide region were also examined to compare results. There were six potential influences on WSUD effectiveness which were explored, focussing on runoff volume, peak flow rate and in some cases flooding:

1. The effect of ongoing infill in a residential catchment
2. Implementation of WSUD with infill development, in the form of retention or detention systems at the allotment and street (lumped) scale
3. Implementation of this WSUD, but comparing a flat and sloped catchment response
4. Implementation of this WSUD, but comparing higher and lower intensity storm events
5. Implementation of this same WSUD, but comparing catchment with higher and lower infiltration rate capacity
6. Implementation of this same WSUD, but comparing the results of assessment at site, street an catchment scale

6.2.1 The effect of ongoing infill

The study found that the mean annual runoff volume in 1993 increased by 54.5% due to the assumed infill of 1 in every 2 homes in the Frederick Street catchment. Without any WSUD, the peak flow rate of combined surface and pipe flow at the catchment outlet increased, and the increase was higher for more frequent events. For example, the peak flow increased by approximately 35% for the 86% AEP, 63% AEP and 50% AEP storms, but by only 18% for the 20% AEP storm and by 6% for the 11% AEP storm. The volume of flooding at a selected assessment point also increased due to infill development. The volume of flooding increased for all event AEPs by a factor of 1.5 to 3 but did not appear to be proportional to event magnitude.

6.2.2 The effect of deploying WSUD with Infill

The impact that WSUD can have on the projected runoff with infill development was explored by implementing onsite and street scale retention (e.g. rainwater tanks or infiltration systems) and onsite or street scale detention tanks. The installation of on-site retention could go some way to maintaining mean annual runoff volume; performance was a function of tank size and anticipated reuse. As an example, the installation of 5 kL retention with 100 L/d demand on every new home in the redeveloped catchment (equivalent to one 10 kL tank on a redeveloped lot with 200 L/d demand) could only go 33% of the way to maintaining the mean annual runoff volume of the catchment prior to redevelopment. The installation of retention in a lumped manner at the street scale was more effective at reducing the mean annual runoff volume compared to the allotment scale. However, daily demand or infiltration was required to be up to 2000 L/day to maintain runoff volume, with a storage size of at least 5 kL per upstream redevelopment (or 2.5 kL per home). The peak flow rate of the catchment could not be maintained to within 10% of the 1993 baseline scenario using allotment based retention. As an example, the installation of 5 kL retention on every new home in the redeveloped scenario with 100 L/day reuse could only reduce the peak flow rate of a 50% AEP event by one quarter of that required to maintain the 1993 levels. When retention was installed in a lumped manner, performance was better than at the allotment scale, however to fully maintain the original peak flow rate at the catchment outlet required the highest level of assumed reuse or infiltration (50 000 L/d).

The peak flow rate of the catchment could not be maintained by allotment detention, but the overall performance was better than retention; for example, the use of 5 kL detention tanks on every new home draining by gravity through a 20 mm orifice plate could reduce the peak flow rate of a 50% AEP storm by 80% toward that required. When detention was installed in a lumped manner, performance was again better than retention; 6 kL per redeveloped allotment (or 3 kL per new home) with an equivalent emptying time to allotment based detention tanks with a 30 mm orifice were able to restore peak flow rates in the catchment in this scenario.

- Both retention and detention performed better when lumped (equivalent allotment volume) at the street scale compared to equivalent alternatives at the allotment scale, largely because of the increased connected impervious area. It is recommended that any policy requiring on site or street based retention or detention measures to manage runoff include requirements which ensure that adequate connected impervious area drains to the WSUD measure.
- In the absence of continuous simulation and frequency analysis, careful consideration is required when selecting the target annual exceedance potential which will be preserved by WSUD – the selection of a level too high may be unachievable using the retention and detention methods outlined in this report.
- While retention is effective a reducing runoff volume, detention performed better than retention where peak flow rate and flooding was considered. This is attributed to the increased availability of space prior to a given event in detention based systems. It is recommended that combined measures are considered where all aspects of runoff management are valued by the community.
- Further research is recommended to explore the impacts of flooding, including the coupling of a two-dimensional flood model with the output of targeted events from continuous simulation to adequately explore impact of WSUD on surface flooding across the catchment.
- The WSUD effectiveness findings are limited to the Adelaide region in South Australia. Similar studies are recommended to explore the impact of WSUD in other Australian and international catchments before broad ranging recommendations about the effectiveness (or otherwise) of WSUD can be determined.

6.2.3 Comparing the Impact of WSUD on Flat and Sloped Catchments

The impact of catchment slope on was examined by comparing the impact of infill and WSUD effectiveness for the selected retention and detention solutions in a redeveloped flat catchment (Frederick Street) and in a redeveloped, moderately sloped catchment (the Paddocks, City of Salisbury). Based on relatively equal amounts of infill, the increase in mean annual runoff volume was almost identical for the flat and moderately sloped case. The impact of 5 kL retention tanks on each redeveloped allotment had a similar impact on this volume, and like the case of the flat catchment, equivalent lumped retention performed better than allotment based measures.

The increase in peak flow rates (including runoff conveyed at the surface and pipe flow) following infill development were consistently larger on the moderately sloped catchment compared to the flat catchment. For example, the 50% AEP peak flow rate increased by 34% on the flat catchment compared to 39% on the sloped catchment; for the 20% AEP, these values were 17.7% (flat) and 37% (sloped). In both cases, the percentage increase was lower as the AEP reduced (became less frequent). Despite the difference in peak flow rate, there was no clear difference in the effectiveness of identical retention or detention layouts on either the flat or sloped catchment. To further explore this, it is recommended that more catchment case studies be undertaken to more rigorously identify whether slope has an impact on the effectiveness of WSUD, especially for much larger catchment areas.

- Slope does not appear to significantly influence the increase in runoff volume nor WSUD effectiveness at maintaining peak flow rates. Based on this, slope is not suggested to be a significant factor that should inhibit WSUD policy from being implemented at the broader scale.
- This finding has been based on only two catchments, however, and further research should be undertaken to determine whether surface and hydraulic slope can influence flooding at smaller and larger scale (i.e. at the allotment/street scale and in catchments greater than 200 Ha).

6.2.4 Comparing the Impact of WSUD with Lower and Higher AEP events

High intensity and low intensity rainfall was compared by comparing actual events form the simulated time series (based on the frequency of the events in the observed outflow time series). Infill development increased the runoff volume and peak flow rate produced by these individual events within the simulated time series. Like the results of the partial series analysis, the percentage increase in event runoff volume and peak flow rate gradually diminished as the AEP of the event became less frequent. For example, the peak flow rate of runoff increased by 36% for the more frequent 86% AEP event, but by only 6 % for the less frequent 11% AEP. The implementation of WSUD in the form of retention or detention also had a greater impact on less frequent storm events. In other words, the events most affected by infill development are those which are more frequent, and these are also the events that can be more effectively restored by WSUD measures. Unlike runoff volume and peak flow rate, the percentage increase in the maximum flood volume at the selected assessment point of the Frederick Street catchment was not identified to be related to event intensity.

These findings can be attributed to runoff properties. Less frequent events can be influenced by pervious area runoff, and when pervious areas contribute to the total runoff volume, the impact of additional connected impervious area is less apparent. This is because runoff volume and peak flows are similarly influenced by impervious area and saturated pervious area.

6.2.5 Comparing the Impact of WSUD on catchments with Higher and Lower Soil Infiltration Rate

The impact of implementing WSUD in catchments with higher and lower infiltration rate was relevant only to retention based measures which allow for infiltration. As the assumed infiltration rate of soil increases (reflected by demand in L/d on this study) allotment based and lumped scale retention both became more effective at reducing runoff volume. Lumped retention measures represented a more effective means of restoring runoff volumes to the levels prior to redevelopment. Generally speaking, storages which could infiltrate greater than 2000 L/d and were sized to consist of at least 5 kL volume per redeveloped allotment could maintain the mean annual runoff volume. Increasing the rate of infiltration (or demand) improved the ability of the retention system to reduce catchment peak flows. As in the case of runoff volume, allotment based retention could not maintain the catchment peak flow rates in the range of assumed parameters of this project, but lumped retention was more effective. However, unlike the runoff volume case, catchment peak flow rates could only be maintained with the very highest assumed demand (50 000 L/d) and for the larger tanks (minimum 8 kL per redeveloped allotment in the contributing subcatchment area). Increasing infiltration rates also led to reduced flood volume, and neither the allotment based nor lumped retention systems, regardless of infiltration rate or tank size, could maintain the assessment point flood volumes to the 1993 baseline.

- Investigate options to increase the emptying time of retention measures.
- During this project is became apparent that a combined retention/detention measure referred to as a 'jump up sump' was one means of managing on site flows. Future research is recommended to explore this measure specifically for on-site flow management to manage increased volume and peak flow rates with infill development.

6.2.6 Comparing the Effect of Assessing WSUD Impact in Smaller to Larger Catchment Scales

The effect of implementing and assessing WSUD in catchments of varying size was examined by comparing the runoff volume and peak flow rate following redevelopment of an allotment, street of allotments and subcatchment (Frederick Street). The percentage increase in mean annual runoff volume due to infill development reduced as the catchment area increased in this study. The results indicate that as the scale of the simulated catchment increased, the predicted effect of infill development on peak flow rate diminished. This trend was not apparent when WSUD implementation was simulated on these same catchment scales. Estimation of peak flow rates following the implementation of retention and detention indicated that WSUD was not clearly more or less beneficial when simulated at the allotment scale than at the larger scale. Relationships for retention remained consistent. However the required orifice size (or emptying time) of detention systems changed according to the size of the catchment being considered to control peak flow rates. For example, the best orifice size for a 5 kL detention tank on an allotment was 20 mm at the allotment and street scale, however this was 10 mm when simulating the catchment scenario.

- For target setting purposes, the impact of WSUD on the mean annual runoff volume was broadly similar at every scale, and the ability of retention based WSUD measures to maintain mean annual runoff volume may therefore be assumed to be fairly examined regardless of the development scale being proposed.
- A similar relationship for peak flow management was found for the allotment to catchment scale scenarios – based on the continuous simulation technique, there was little change in the ability of WSUD with respect to the scale of assessment.
- Further research should be conducted by adopting an even larger catchment for assessment and looking closely at flooding before the catchment size for WSUD performance assessment is ignored in WSUD policy settings to maintain flood and flow conditions.

6.2.7 Comparing the Cost of Effectiveness of WSUD at Site Scale and Catchment Upgrade Works

Cost effectiveness was assessed for five WSUD implementation scenarios. Results indicated that allotment scale WSUD, which could contribute to but not fully restore the need for a stormwater system upgrade, was always cheaper than the cost of a conventional upgrade of the 'minor' system. The cheapest and most effective means to reduce peak flow rates was detention systems at the allotment. However it should be noted that detention makes no contribution to reducing runoff volume. The use of enlarged sections of concrete pipe, as storage, at each stormwater pit to provide street scale detention was the most expensive means considered to minimise peak flows and flooding at the catchment scale. These results may be heavily influenced by the costing assumptions. Key among these assumptions was the exclusion of design, construction supervision and approvals costs. The findings are also based on comparing the WSUD scenarios with an upgrade of the whole drainage system, where detailed investigation may reveal that only a partial system upgrade is required. A further limitation on the study findings is that the requirements of detention are likely to vary based on the scale of the catchment considered.

- The study indicated that on site retention and detention are cheaper than implementing a full system upgrade. Further research into the cost of partial upgrade works and considering typical fees for of design, construction supervision and approvals costs. For example, allotment retention may be cheaper to design, supervise and assess for one allotment, but the cumulative cost of doing this for hundred's or more homes during the development process may impact on cost effectiveness.
- Any cost benefit should be suitably weighed against the inability of WSUD measures at the allotment to fully maintain existing flow conditions in a catchment experiencing infill.

6.3 The Impact of Using Event Based and Continuous Modelling Approaches for WSUD System Design

The effectiveness of WSUD to maintain post-infill peak flow rates to pre-infill levels at the allotment and catchment scale was examined using a performance estimate based on a design storm simulation (where storage was assumed to be empty) and an estimate based on continuous simulation and subsequent partial series analysis of a simulated flow timeseries (which was the basis of all results in this report). Simulation was conducted assuming on-site measures only. The results indicated that each approach produced a significantly different performance estimate. For example, the assumption of an empty storage indicated that both retention and detention with a 4 kL volume could effectively maintain the 50% AEP peak flow rate for retention and detention cases. The continuous simulation however indicated that much larger storages were required in both cases to achieve best results and these were still not maintaining peak flow rates for the relatively frequent 50% AEP event. This is because the design storm simulation does not take into account the constant filling and emptying of storages. Based on this, further investigation was undertaken to identify is some prior capacity may be assumed as a compromise between the design and continuous simulation approaches.

The analysis was undertaken for using a long time series of rainfall recorded for Adelaide, combining records at West Terrace and Kent Town. The rainfall record was used to predict the condition of a rainwater tank of varying size fitted to 200 m^2 of impervious area with varying reuse conditions. Results showed that for all winter events the tank was near to or full prior to storm bursts with a 20% AEP (similar to the 5 year ARI). Analysis indicated that at least 12 months prior conditions were required to reach an equilibrium (where the assumed storage volume of the tank ceased to impact the ability of the tank to capture the 20% AEP storm).

- The adoption of a design storm based approach to examine the performance of WSUD systems is strongly influenced by the underlying assumption of storage available prior to design storm bursts.
- Analysis of long time series rainfall indicates that up to one year is required for the assumed initial storage of a tank to cease being relevant (based on rainfall data for central Adelaide, South Australia)
- It is suggested that the adoption of a design pre-burst rainfall event with the design burst event is unlikely to be feasible, using design tools used by the profession. An alternative approach is to develop a relationship between storage, discharge and catchment area.

7. References

ARGUE, J. R., GOOD, K. & MULCAHY, D. E. 1994. Planning, Instrumentation and Data for an Urban Drainage Network in Adelaide, South Australia. *Water Down Under '94.* Adelaide, SA: Institution of Engineers, Australia.

ASCE TASK COMMITTEE ON DEFINITION OF CRITERIA FOR EVALUATION OF WATERSHED MODELS OF THE WATERSHED MANAGEMENT COMMITTEE & IRRIGATION DRAINAGE DIVISION 1993. Criteria for Evaluation of Watershed Models. *Journal of Irrigation and Drainage Engineering,* 119**,** 429-442.

BALL, J., BABISTER, M., NATHAN, R., WEEKS, W., WEINMANN, E., RETALLICK, M. & TESTONI, I. (eds.) 2016a. *Australian Rainfall and Runoff: A Guide to Flood Estimation,* Canberra, ACT, Australia: Commonwealth of Australia (Geoscience Australia), 2016.

BALL, J., BABISTER, M., RETALLICK, M. & WEINMANN, E. 2016b. Introduction. *In:* BALL, J., BABISTER, M., NATHAN, R., WEEKS, W., WEINMANN, E., RETALLICK, M. & TESTONI, I. (eds.) *Australian Rainfall and Runoff - A Guide to Flood Estimation.* Canberra, ACT: Commonwealth of Australia (Geoscience Australia).

CHIEW, F. H. S., WANG, Q. J., MCCONACHY, F., JAMES, R., WRIGHT, W. & DEHOEDT, G. 2002. Evapotranspiration maps for Australia. *Hydrology and Water Resources Symposium.* Melbourne, Victoria, Australia: Institution of Engineers, Australia.

CUNNANE, C. 1978. Unbiased plotting positions — A review. *Journal of Hydrology,* 37**,** 205-222.

GHAFOURI, R. A. 1996. *Deterministic analysis and simulation of runoff in urban catchments.* PhD, University of Wollongong.

GOVERNMENT OF SOUTH AUSTRALIA 2010. The 30 year plan for greater Adelaide: A volume of the South Australian planning strategy. Adelaide, SA, Australia: Government of South Australia, Department of Planning and Local Government.

GOVERNMENT OF SOUTH AUSTRALIA 2015. Transforming our Planning System: Response of the South Australian Government to the final report and recommendations of the Expert Panel on Planning Reform. Adelaide, SA, Australia: Government of South Australia.

GOVERNMENT OF SOUTH AUSTRALIA 2017. The 30 Year Plan for Greater Adelaide. Adelaide, SA, Australia: Government of South Australia, Department of Planning, Transport and Infrastructure.

JAIN, S. K. & SUDHEER, K. P. 2008. Fitting of Hydrologic Models: A Close Look at the Nash-Sutcliffe Index. *J. Hydrol. Eng.,* 13**,** 981.

KEMP, D. 2002. *The development of a rainfall-runoff-routing model.* PhD, University of Adelaide.

KEMP, D. & LIPP, W. R. 1999. Predicting Storm Runoff in Adelaide – How Much do we Know? *Hydrological Society of South Australia, Living with Water Seminar.* Adelaide, SA, Australia.

KRAUSE, P., BOYLE, D. P. & BÄSE, F. 2005. Comparison of different efficiency criteria for hydrological model assessment. *Advances in Geosciences,* 5**,** 89-97.

LADSON, A. R. 2008. *Hydrology: An Australian Introduction,* South Melbourne, Vic., Oxford University Press.

LEE, H. & BRUCE, D. 1995. Where does the water go?: the use of ARC/INFO to provide data for rainfall runoff and flow modelling in an urban environment. *Ninth Annual Australian Conference for ESRI & ERDAS Users.* Sydney, NSW, Australia.

MAIDMENT, D. R. 1993. *Handbook of hydrology,* New York, McGraw-Hill.

MARSDEN JACOB AND ASSOCIATES 2007. The cost-effectiveness of rainwater tanks in urban Australia. Canberra, ACT, Australia: National Water Commission.

MYERS, B., PEZZANITI, D., KEMP, D., CHAVOSHI, S., MONTAZERI, M., SHARMA, A., CHACKO, P., HEWA, G. A., TJANDRAATMADJA, G. & COOK, S. 2014. 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. Adelaide, South Australia: Goyder Institute for Water Research.

PETRUCCI, G., DEROUBAIX, J.-F., DE GOUVELLO, B., DEUTSCH, J.-C., BOMPARD, P. & TASSIN, B. 2012. Rainwater harvesting to control stormwater runoff in suburban areas. An experimental case-study. *Urban Water Journal,* 9**,** 45-55.

PEZZANITI, D. 2003. Drainage system benefits of catchment wide use of rainwater tanks. Adelaide, South Australia: Urban Water Resources Centre.

PHAM, H., SHAMSELDIN, A. & MELVILLE, B. 2014. Statistical Properties of Partial Duration Series and Its Implication on Regional Frequency Analysis. *Journal of Hydrologic Engineering,* 19**,** 1471-1480.

PHILLIPS, B. C., THOMAS, C. & PINTO, M. 2016. Comparing design storm burst and embedded design storm approaches in the Narellan Creek Catchment, NSW. *37th Hydrology & Water Resources Symposium 2016: Water, Infrastructure and the Environment.* Engineers Australia.

PILGRIM, D. H. (ed.) 1987. *Australian Rainfall & Runoff - A Guide to Flood Estimation,* Barton, ACT: Institution of Engineers, Australia.

PILGRIM, D. H. (ed.) 1999. *Australian Rainfall and Runoff,* Barton, ACT, Australia: The Institution of Engineers, Australia.

RAWLINSONS (ed.) 2015. *Rawlinsons Australian Construction Handbook,* Perth, WA, Australia: Rawlinsons Publishing.

RIGBY, T., BOYD, M., ROSO, S. & VANDRIE, R. 2005. Storms, storm bursts and flood estimation: a need for review of the AR&R procedures. *Australasian Journal of Water Resources,* 8**,** 213-221.

ROSSMAN, L. A. 2010. Storm Water Management Model User's Manual Version 5.0. Cincinnati, Ohio: National Risk Management Research Laboratory, United States Environmental Protection Agency.

SCOTT, P. 1994. *Modelling of a gauged catchment for ILSAX and RAFTS application using photogrammetry.* Civil Engineering Honours, University of South Australia.

TOMLINSON, G. W., FISHER, A. G. & CLARK, R. 1993. The Paddocks. Adelaide, SA, Australia: Engineering and Water Supply Department.

Appendix A – Conference Paper

A Verification of the Hydrological Impact of 20 Years of Infill Development in an Urban Catchment

David J Kemp
Adjunct Senior Research Fellow, Centre for Water Management & Reuse, University of South Australia
E-mail: david.kemp@unisa.edu.au

Baden R Myers
Research Engineer, Centre for Water Management & Reuse, University of South Australia
Email baden.myers@unisa.edu.au

Abstract

The Frederick Street, Glenelg catchment was instrumented in the early 1990s with an in-pipe flow meter and two pluviometers. It is a fully developed urban catchment of 48.7ha, with most of the area covered with residential development built in the late 1940s and the 1950s. Data from the catchment was used by the first author at that time to calibrate the ILSAX and RAFTS models, and to compare the calibrated directly connected impervious area with that surveyed in the field. After a few years of service the station closed.

More recently a project has commenced to investigate the impact of infill development. In 2013 the instrumentation was reinstalled and there is now sufficient new data available to recalibrate the models using the same methodology and compare the change in hydrological behaviour with that expected by the change in contributing areas, particularly the directly connected impervious area.

This paper discusses the analysis and the results, which show a significant increase in peak flows and runoff volume in the 20 years between the initial calibration and the present time. The change is consistent with that predicted based on an analysis of actual increase in directly connected impervious area determined using aerial photographs of the catchment. The results give confirmation of the impact of infill development on drainage standards and annual discharge volumes.

Keywords: Urban hydrology, infill development, hydrological modelling.

1. INTRODUCTION

The 30 Year Plan for Greater Adelaide (Department of Planning & Local Government, 2010) was released by the South Australian Government in February 2010. It is the principal policy mechanism to plan for population growth of 560,000 people and 258,000 new dwellings across the Greater Adelaide region over the next 30 years. Underpinning the principles and objectives of the Plan is the recognition of the major challenges facing many urban regions today, including the need to make better use of existing infrastructure, reduce sprawl by promoting more compact urban forms and the impact of an ageing population.

In common with other metropolitan strategies in Australia, the Plan calls for a shift in the share of new dwellings built on the urban fringe to infill areas in established suburbs to help reduce the outward sprawl of the Adelaide urban area and to promote a more compact city. Currently, the ratio of infill to fringe development is 50:50, but the Plan stipulates a ratio of 70:30 is to be achieved over the 30 year period.

Local governments in SA have been preparing stormwater management plans to identify and prioritise drainage works required to cope with current and future development levels. For example, the Marion Holdfast Stormwater Management Plan (Tonkin Consulting, 2013) identified that the extent of infill development in the catchment being considered was a significant issue. It noted that "the approximate net effect of this (infill) is to increase the impervious surfaces of the residential areas in the catchment from approximately 800 ha to 1000 ha, an increase of 25%" (Tonkin Consulting 2013: 16). One of the recommendations of the plan was to "Cooperate with other agencies to develop and conduct stormwater quality monitoring and reporting programs." (Tonkin Consulting 2013: 79).

The Frederick Street, Glenelg catchment, which lies within the area considered by the aforementioned stormwater management plan, was instrumented in the early 1990s with an in-pipe flow meter and two pluviometers. Data from the catchment was used by the first author at that time to calibrate ILSAX and RAFTS models developed for the catchment, and to compare the calibrated directly connected impervious area with that surveyed in the field (Kemp, 2002). After a few years of service the station closed. As part of the monitoring recommended by the stormwater management plan, the Frederick Street catchment rainfall and pipe flow stations were reinstated and an opportunity arose to verify the actual impact on peak flows and flow volumes with 20 years of infill development.

2. THE FREDERICK STREET CATCHMENT

The 48.7 Ha Frederick Street catchment, shown in Figure 1 is 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%.

The contributing areas of the catchment were determined in 1993 from digitised aerial photography, with site verification by students of the University of South Australia. Bruce et al (1994) describes the methodology. Myers et al (2014) summarises the characteristics of the catchment based on the 1993 data collection.

Table 1 – Summary of catchment properties for the Frederick Street catchment

Catchment property	Value
Total area (Ha)	48.7
Directly connected impervious area (%)	30.4
Indirectly connected impervious area (%)	17.1
Pervious area (%)	52.5

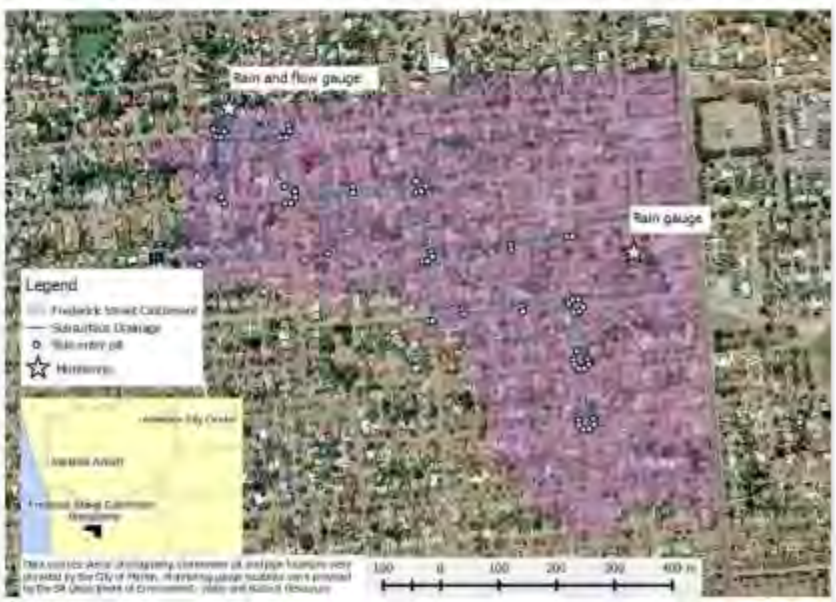


Figure 1 The Frederick Street catchment

There were originally six rainfall and two flow monitoring stations initiated in 1992, which were spread across a larger catchment extending to the coast. In August 2013 a rainfall station at the upper end of the catchment (A5040556) was reinstated, plus the in-pipe flow measurement and rainfall station at Frederick Street (A5040561).

3. EXPECTED CHANGES IN ANNUAL RUNOFF VOLUMES

The Goyder Institute for Water Research was established in July 2010 as a partnership between the South Australian Government through the Department of Environment, Water and Natural Resources, CSIRO, Flinders University, the University of Adelaide, and the University of South Australia. The Goyder Institute for Water Research has funded an investigation into the potential role of Water Sensitive Urban Design (WSUD) in urban water service provision (Myers et al, 2014).

One of the catchments examined for the potential role of WSUD was the Frederick Street catchment. Prior to reinstating the rain and flow gauges, simulations were conducted using the SWMM model which attempted to determine peak flow and runoff volumes from three development scenarios in the catchment. These were:

- 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), Bruce et al (1994) and Lee and Bruce (1995).
- 2013 Development – the calibration case model with additional development included based on a survey of aerial photos of the catchment in February 2013. The increased directly connected impervious area is due not only to increased roof area, but extra driveways and paving that is connected to the street drainage system.
- 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 Consulting, 2013). However a lot analysis within the Frederick Street catchment indicated that the growth in housing allotments was 0.65% per annum. The latter figure was used in this analysis.

A summary of the final properties of the catchment for each scenario is shown in Table 2.

Table 2 – 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

An example of the development that has taken place to 2013 is shown in Figure 2.

On the basis that the increase in runoff volume would be closely related to the percentage of directly connected impervious area it would be expected on the basis of Table 2 to be of the order of 15% a increase from 1993 until 2013, and 31.5% from 1993 until 2040, unless controls were put in place.

Figure 2 Example of Redevelopment in the Frederick Street Catchment

4. MEASURED CHANGES IN RUNOFF VOLUME

In the initial analysis (Kemp, 2002) both the ILSAX and the RAFTS models were calibrated on the catchment. In that study there was little runoff from the pervious area evident from stormwater events within the catchment, with only two of the 12 events modeled having any pervious area runoff. This is consistent with the behavior of other Adelaide urban catchments (Kemp and Lipp, 1999).

Given that there is little contribution from anything but the directly connected impervious area it would be expected that there would be a significant correlation between total rainfall and runoff over time. Therefore, if a double mass curve of cumulative runoff volume for the catchment was plotted against cumulative rainfall volume for a defined period, a good correlation should be found with the slope of the line being close to the percentage directly connected impervious area. If infill development has an impact on runoff volume this relationship should change over time as infill development progresses. Data from two periods were therefore examined, from August 1992 until November 1996, and from August 2013 until April 2015.

Figure 3 and Figure 4 show the resulting plots, with good correlation found in both cases. The 2013 to 2015 data was however more consistent. The slope of the two trend lines shows the volumetric runoff coefficient for the catchment, being 28.4% for the 1992 to 1996 period, and 31.2% for the 2013 to 2015 period. There has therefore been a measured increase of 10% in runoff volume in the catchment due to infill development that has occurred.

This can be compared with the expected increase by the assessment of the change in directly connected impervious area in Table 2. This predicted an increase of 15%, but the difference could be due to measurement error, particularly given the relatively poor fit of the 1992-1996 data. It is of note that the volumetric runoff coefficient in the 1990s was lower than that expected by the field study described by Argue et al (1994) and Lee and Bruce (1995), if the volumetric runoff coefficient reflects the percentage connected impervious area (30.4%).

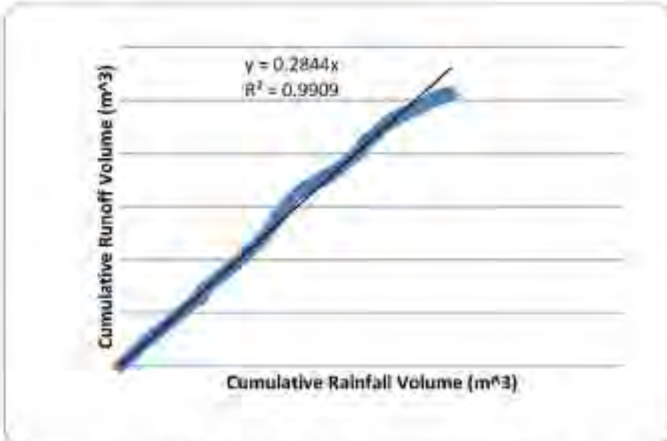


Figure 3 Frederick Street Mass Curve 1992-1996

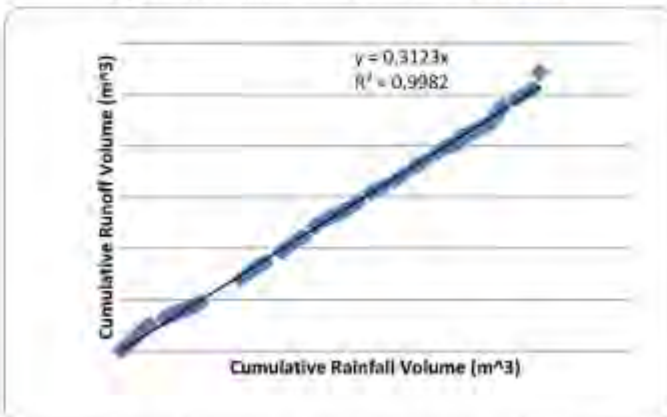


Figure 4 Frederick Street Mass Curve 2013-2015

5. CHANGE IN RESPONSE TO RAINFALL EVENTS

5.1. Calibration of 1992 and 1993 Events

In the initial analysis data from the seven largest storms recorded in 1992 and the five largest storms of 1993 were fitted to the ILSAX model at the Frederick Street gauging station (AW504561). The highest rainfall intensity was between 2 and 5 years ARI for 5 minute duration, using the 1987 IFD analysis. The fitting procedure was as follows:

- Storms with runoff from the impervious area only were identified, by examining the percentage runoff (runoff volume/rainfall volume);
- The 1992 storms having only an impervious area runoff component were fitted first, by the use of the sensitivity adjustment available within the ILSAX model to transfer directly connected impervious area to supplementary paved area. For example a –10% sensitivity adjustment transfers 10% of the directly connected impervious area to supplementary paved area, without affecting the total catchment area. A paved area depression loss of 1mm was used, as recommended by the ILSAX manual;

- The other storms were then modelled, using the best fit for the directly connected impervious area sensitivity adjustment. The initial loss for the impervious area was set to model the start of the rise of the gauged flow, and the initial loss for the pervious area was set to start the contribution from the pervious area where the fitted flow deviated from the gauged flow, assuming no pervious area runoff. Continuing loss on the pervious area was used to best model the total runoff volume. The apparent lag of the pervious area runoff was adjusted by altering the pervious area roughness value 'n'.

It was found that there was no one directly connected impervious area sensitivity adjustment factor that can be applied to all storms to give a good match between predicted and observed flows and volumes. The effect of the constant initial loss was first investigated, but this was considered not to have a major effect. A range of sensitivity adjustment to the directly connected impervious area of 0% to -15% was examined, and an adjustment of -10% was chosen to model the storms with pervious area runoff on the basis that this adjustment was in the mid-range of the best fits for the above storms, and by inspection produced the best overall fit of the shape of the hydrographs. The result of the calibration is given in Table 3.

Table 3 1992 and 1993 ILSAX Model Calibration

Date	Peak Flow (m^3/sec)		ratio	Volume(m^3)		Ratio	Pervious runoff
	Measured	Predicted		Measured	Predicted		
3/07/1992	0.336	0.287	0.85	1383	1357	0.98	0%
1/08/1992	0.306	0.314	1.03	909	1019	1.12	0%
11/07/1992	0.128	0.142	1.11	981	971	0.99	0%
19/07/1992	0.316	0.288	0.91	784	656	0.84	0%
30/08/1992	1.078	1.069	0.99	3461	3158	0.91	11.0%
31/08/1992	0.349	0.368	1.05	647	563	0.87	0%
18/12/1992	1.242	1.249	1.01	5837	5801	0.99	21.6%
24/05/1993	0.322	0.344	1.07	762	912	1.20	0%
30/08/1993	0.534	0.654	1.23	1163	1350	1.16	0%
19/09/1993	0.652	0.656	1.00	970	976	1.01	0%
30/09/1993	0.312	0.255	0.82	644	617	0.96	0%
17/10/1993	0.548	0.495	0.90	762	955	0.97	0%
Mean			1.07			0.96	

5.2. Calibration of 2013 and 2014 Events

Data from the largest eight events was used in the calibration of the same ILSAX model. Initial inspection of the events showed that there was no evidence of pervious area runoff in any event. Thus only the directly connected impervious area was used as a sensitivity adjustment, with a +5% adjustment being found to give the best overall fit to peak flow and volume. The result of the calibration is given in Table 4.

The calibration of the same ILSAX model on the catchment from two periods twenty one years apart has shown that the adjustment to the directly connected impervious area has changed from -10% to +5%, a very significant change. Since the initial model directly connected impervious area was 30.4%, the change represents an increase of 16.7%.

Table 4 2013 and 2014 ILSAX Model Calibration

Date	Peak Flow (m^3/sec)		Ratio	Volume(m^3)		Ratio
	Measured	Predicted		Measured	Predicted	
21/08/2013	0.145	0.242	1.67	811	747	0.92
12/09/2013	0.334	0.462	1.38	2459	3297	1.34
13/02/2014	0.414	0.491	1.19	5169	6116	1.18
14/02/2014	0.362	0.359	0.99	6302	5641	0.90
29/04/2014	0.582	0.514	0.88	4314	3855	0.89
2/05/2014	0.505	0.402	0.80	2923	2436	0.83
5/05/2014	0.347	0.264	0.76	1109	867	0.78
9/05/2014	0.28	0.252	0.90	4113	3403	0.83
Mean			1.07			0.96

5.3. Change in Peak Flow and Event Runoff Volume

All events from both periods of record were then run with the sensitivity adjustment of -10% and +5% to give an indication of the impact of the change in the catchment to overall peak flow and runoff volume from storm events. It was found that for all events but two the ratio was very similar, indicating an increase in peak flow of 16.8% and runoff volume of 16.3% over the 21 years. The two events showing a smaller increase were the two events in 1992 that showed pervious area runoff. It would be expected for these two events that the increase would not be tied so closely to the increase in directly connected impervious area.

The increase of the order of 16% in event peak flow and volume are close to those expected from the assessment of the increase in the percentage of directly connected impervious area in the catchment, outlined in Section 3.

6. IMPACTS OF THE CHANGES

The changes in catchment response due to infill development are significant without controls being put in place. In 1993, there were 555 allotments in the catchment area. By 2013, there were 632 allotments, including at least 77 new homes. These additional structures associated paved areas and additions to existing housing stock all have an effect on flood damage. The Stormwater Management Plan for the entire Marion-Holdfast catchment that includes Frederick Street states that from the current situation (2013) until a future (2040) case;

> The floodplain mapping for the future scenario 1 in 5 year ARI flood, which assumes no controls over development, shows a noticeable increase in the amount of water in the road network during a 1 in 5 year ARI storm event. Calculated flood loss damages for the 5 year storm increase significantly from $1.2 million to $4.9 million, a 300% increase.
>
> Tonkin Consulting (2013:42)

The increase in damage reflects an increase in the number of houses flooded above floor level from 9 to 52 (Tonkin, 2013). When climate change was considered:

> The current predicted increase in rainfall intensity of around 3% due to climate change is slight and its impact will be dwarfed by the impact of the increasing imperviousness of the catchment arising from current development trends.
>
> Tonkin Consulting (2013:42)

Myers et al (2014) found that at a sample location within the Frederick Street catchment all events with a frequency greater than 2.5 year ARI caused flooding (overflow from the minor drainage system). In the 2013 scenario, this frequency increased to include all events greater than 2.2 year ARI, and in the

2040 scenario, this frequency had increased to include all events greater than 1.7 year ARI. The original design standard (in the late 1960s) was a 5 year ARI.

7. SUMMARY

It has been verified by gauging of rainfall and runoff on an urban catchment that the effect of infill development is very significant. In the last twenty years there has been an increase of 10% in the volumetric runoff coefficient of the catchment, with a 16% increase in event peak flow and runoff volume. This is consistent with the measured increase in catchment directly connected impervious area. It is predicted that by 2040 without development controls there will be a total increase of 31.5% in directly connected impervious area, leading to a proportional increase in event and annual runoff volume, and event peak flows. There is potentially an increase of 300% in damages due to a 5 year ARI storm event in the Marion-Holdfast catchment.

The University of South Australia's Centre for Water Management & Reuse is currently undertaking a project with support of six Local Government authorities in Adelaide to determine means of addressing the impact of infill development, including the implementation of Water Sensitive Urban Design measures on an allotment of street scale.

8. ACKNOWLEDGMENTS

The authors wish to acknowledge the financial support for this project that has been received from the Goyder Institute for Water Research, the Stormwater Management Fund, the Adelaide and Mount Lofty Ranges NRM Board and six Local Government Authorities in Adelaide.

9. REFERENCES

Argue, J. R., Good, K. & Mulcahy, D. E. 1994. Planning, *Instrumentation and Data for an Urban Drainage Network in Adelaide, South Australia*. Water Down Under '94. Adelaide, SA: Institution of Engineers, Australia.

Department of Planning & Local Government, Government of South Australia (2010) *Planning the Adelaide we all want: Progressing the 30-year Plan for Greater Adelaide*, Adelaide, February 2010.

Bruce, D., Lee, H., and Argue, J. (1994) *Marion - Glenelg Quantity/Quality Stormwater Monitoring Project - Catchment Characteristics Using GIS, Progress Report* University of South Australia. Spatial Measurement and Information Group (SMIG), September 1994.

Lee, H. & Bruce, D. (1995). *Where does the water go?: the use of ARC/INFO to provide data for rainfall runoff and flow modelling in an urban environment*. Ninth Annual Australian Conference for ESRI & ERDAS Users, Sydney, NSW, Australia.

Kemp, D.J and Lipp, W.R. (1999) *Predicting Storm Runoff in Adelaide – How Much do we Know?* Hydrological Society of South Australia, Living with Water Seminar, Adelaide, October 1999.

Kemp, D.J (2002) *The Development of a Rainfall-Runoff-Routing (RRR) Model* PhD Thesis, The University of Adelaide, August 2002.

Myers B, Pezzaniti D, Kemp D, Chavoshi S, Montazeri M, Sharma A, Chacko P, Hewa GA, Tjandraatmadja G and Cook S (2014) *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.* Goyder Institute for Water Research Technical Report Series No. 14/19, Adelaide, South Australia.

Tonkin Consulting (2013), *Stormwater Management Plan - Coastal Catchments Between Glenelg and Marino*. Adelaide, SA: Tonkin Consulting on behalf of the City of Marion and the City of Holdfast Bay.

Appendix B – Frederick Street Catchment Model

The Frederick Street catchment model that was used to investigate the characteristics of flow volume and flow rate in the pre-infill, post-infill and post infill with WSUD scenarios of this report was based on a calibrated model developed for a prior study into the performance of some WSUD in this catchment for the Goyder Institute for Water Research (Myers et al., 2014). Full details on the catchment, model development, calibration and verification are provided below in a format slightly modified from the original reporting by Myers et al. (2014).

Introduction

There are several areas in the Adelaide metropolitan area where the standard of the underlying drainage system has been reduced by infill development. The 44.7 Ha Frederick Street catchment, illustrated in Figure B 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.

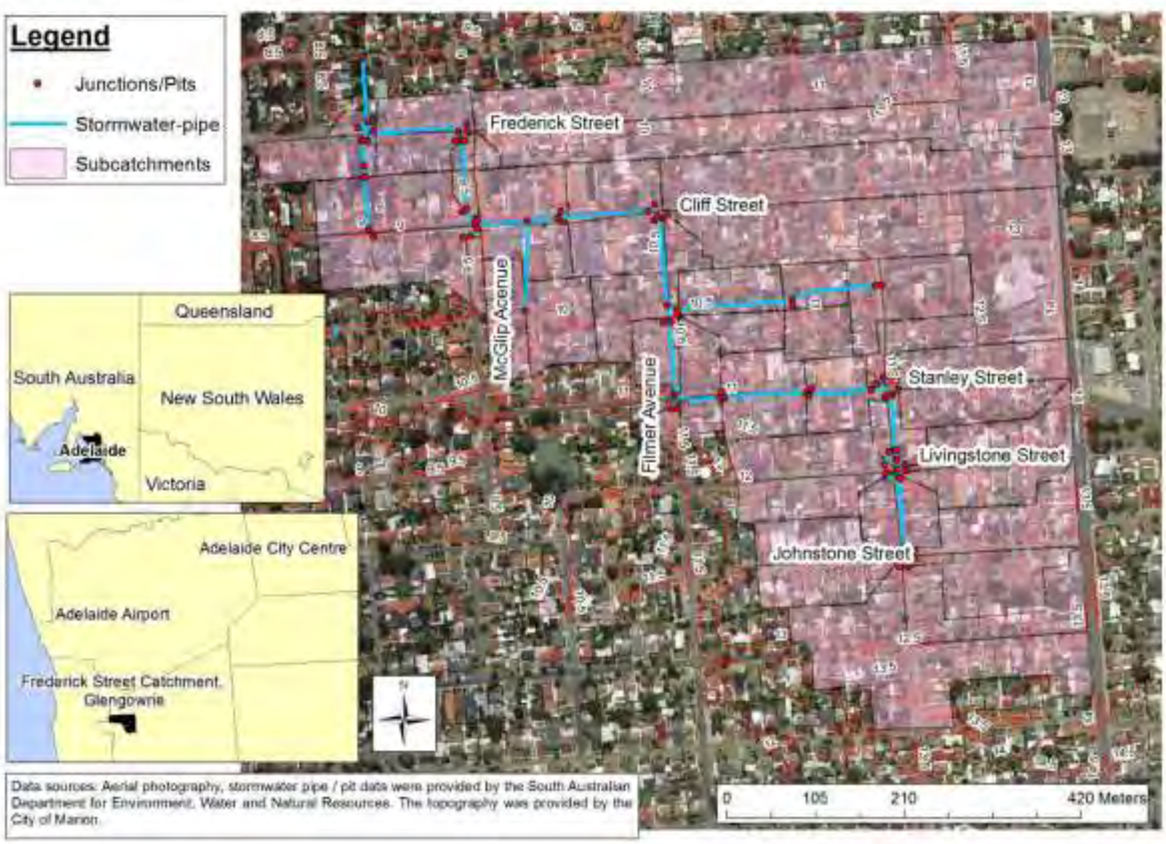


Figure B 1 – Location of the Drain-18 catchment, indicating selected roads

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 B 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.

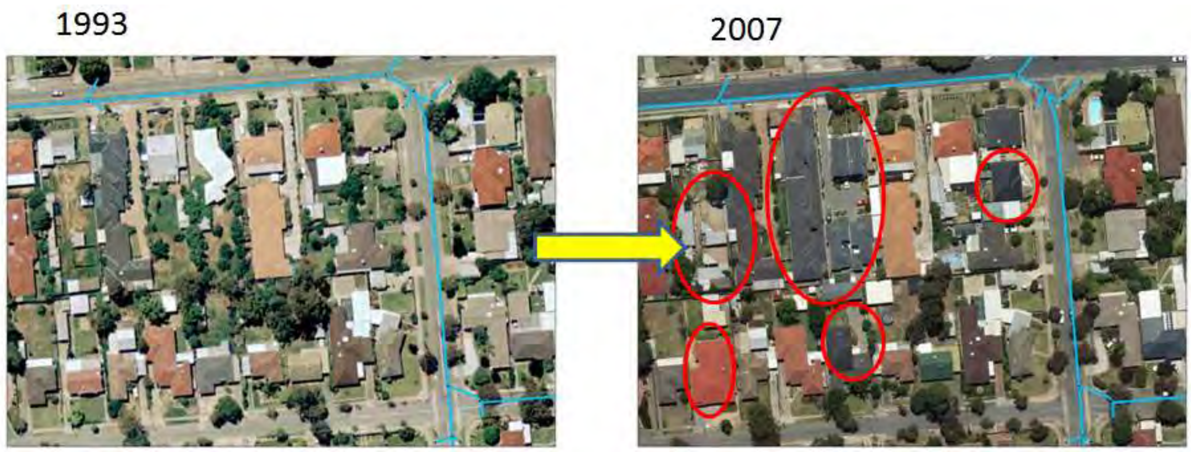


Figure B 2 – Examples of redevelopment on the Corner of Cliff Street and Filmer Avenue in the Frederick Street catchment

Frederick Street 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

Model Assembly

<u>Previous Modelling</u>

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' (Lee and Bruce, 1995, 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 B 1.

Table B 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 shown in Figure B 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 B 1). Figure B 2 summarises the characteristics of the catchment based on the 1993 data collection.

Table B 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 B 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 B 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 a similar parameter to 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 B 3, and their locations with respect to the Frederick Street catchment are depicted in Figure B 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 B 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 B 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 B 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 B 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.

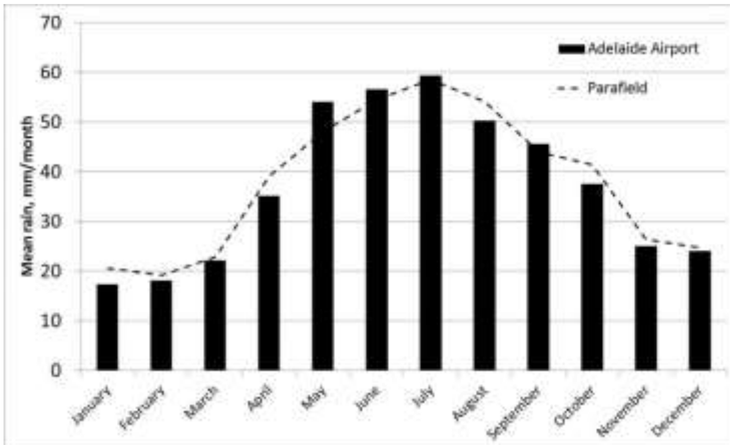


Figure B 4 – Mean monthly rainfall at the Adelaide Airport BOM gauge (023034)

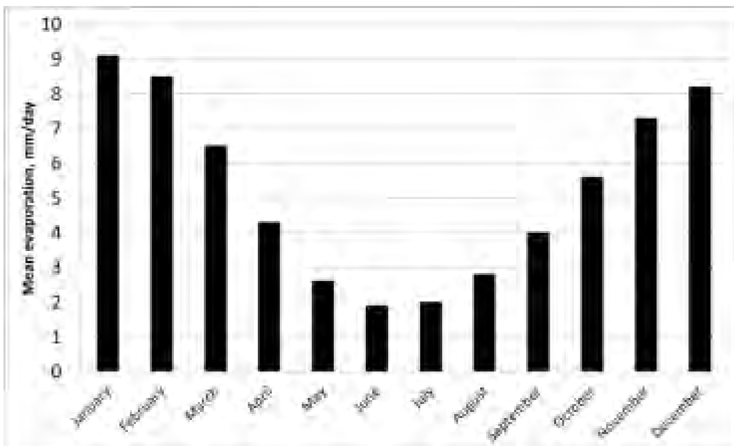


Figure B 5 – Mean daily evaporation at the Adelaide Airport BOM gauge (023034)

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

<u>Other Assumptions</u>

- 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.

Model Calibration - Approach

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. Visual examination of the simulated and observed hydrographs was also supported by quantitative analysis. 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, 2008). 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 from the ASCE Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee and Irrigation Drainage Division (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 Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee and Irrigation Drainage Division (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.

Model Calibration – Frederick Street

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). For this model, , 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 B 4. Figure B 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 B 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

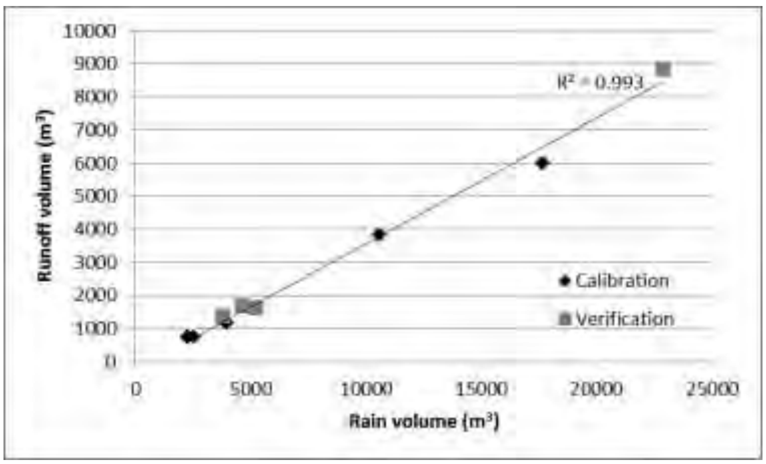


Figure B 6 – Comparison of rainfall volume and runoff volume for the calibration and verification events of the Frederick Street model

The plot in Figure B 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 B 5. The results of the model calibration for the events in Table B 4 are shown in Table B 6.

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

Event	Observed Peak Flow (m^3/s)	Simulated Peak Flow (m^3/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 B 7 and Figure B 8 illustrate the fit of the simulated to the observed hydrograph.

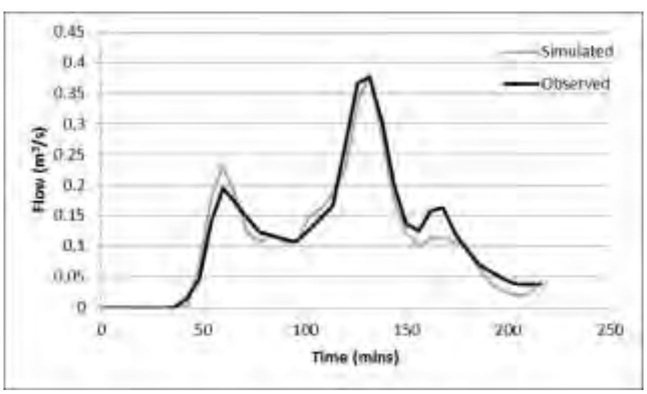


Figure B 7 – Comparison of the observed and simulated flows for event Cal1 (Frederick Street)

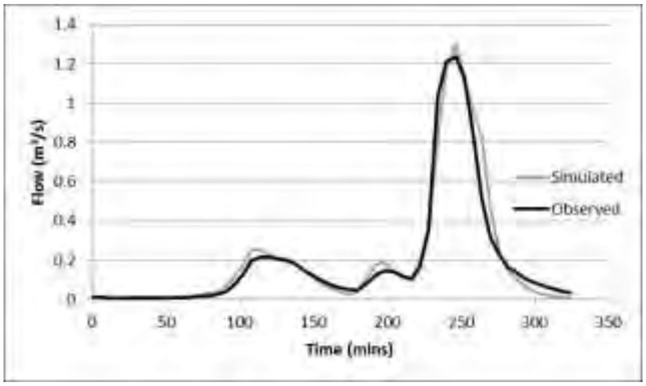


Figure B 8 – Comparison of the observed and simulated flows for event Cal5 (Frederick Street)

Model Validation – Frederick Street

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 B 7. A plot of observed runoff and rainfall volume for these events was previously presented in Figure B 6.

Table B 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 B 8, with selected hydrographs shown beneath. As all events show a reasonable fit to the data, the model was accepted as suitable for the purposes of this study.

Table B 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 B 9 and Figure B 10.

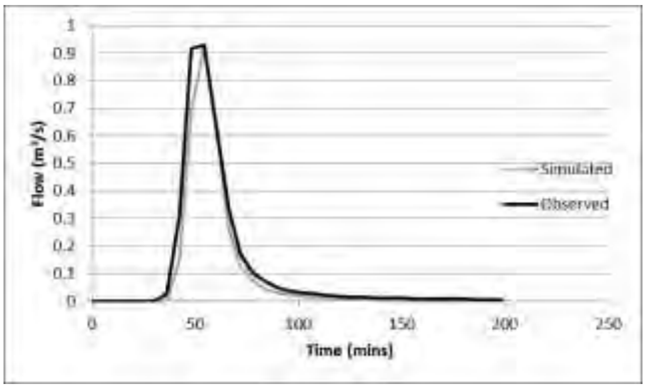


Figure B 9 – Comparison of the observed and simulated flows for event V2 (Frederick Street)

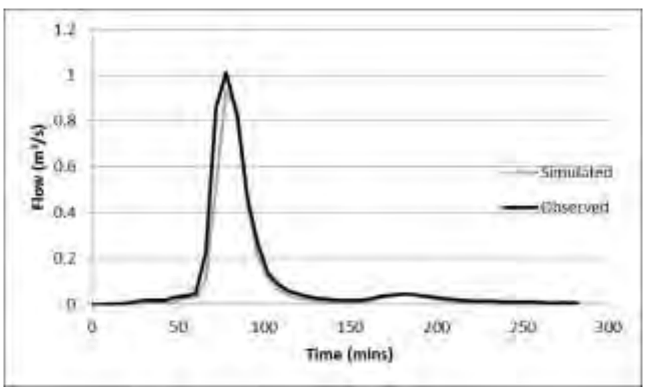


Figure B 10 – Comparison of the observed and simulated flows for event V4 (Frederick Street)

Appendix C – Frederick Street Catchment Model

The Paddocks catchment model that was used to investigate the characteristics of flow volume and flow rate in the pre-infill, post-infill and post infill with WSUD scenarios of this report (comparing the impact of catchment slope in Section 5.2.3) was based on a calibrated model developed for a prior study into the performance of some WSUD in the Paddocks for the Goyder Institute for Water Research (Myers et al., 2014). Full details on the catchment, model development, calibration and verification are provided below in a format slightly modified from the original reporting by Myers et al. (2014).

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 C 1.

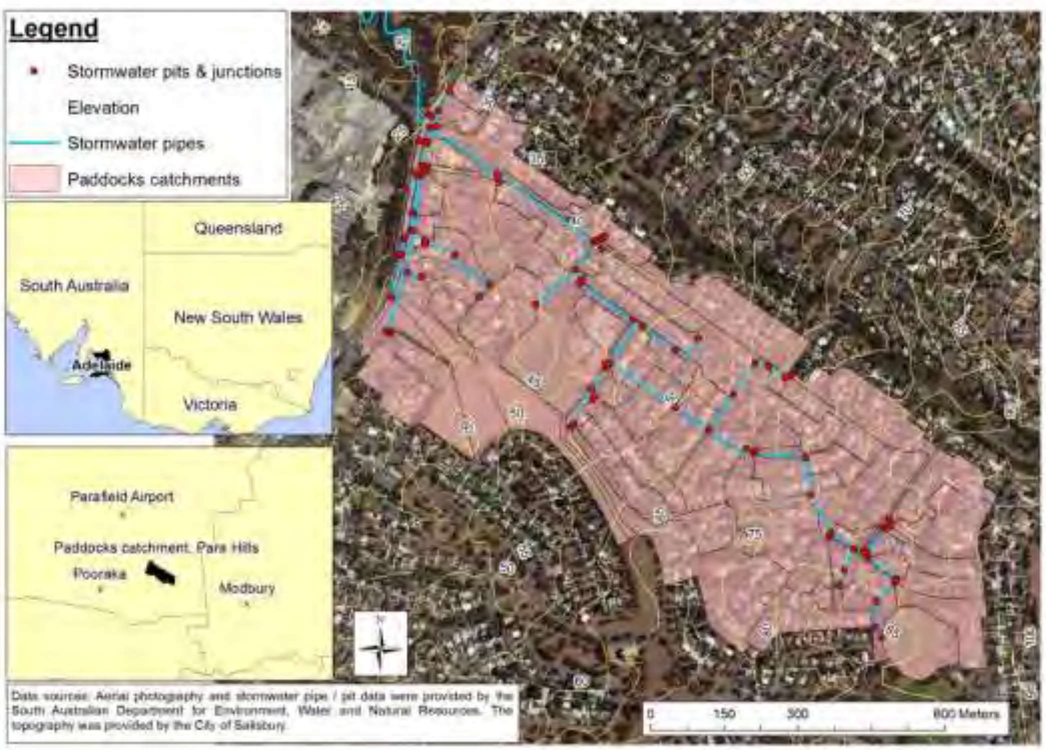


Figure C 1 – Location of the Paddocks, indicating surrounding suburbs

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.

Paddocks 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

Model Assembly

Previous Modelling

Previous modelling of the Paddocks catchment was available from Kemp (2002) and Scott (1994), both of whom 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 C 1.

Table C 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 C 2 summarises the characteristics of the catchment based on the site analysis in 1993.

Table C 2 – 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 C 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 several 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 C 3. The location of these gauges is shown in Figure C 2. All gauges lie within the catchment boundary.

Table C 3 – 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 C 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 C 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 C 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 (see Figure C 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 C 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.

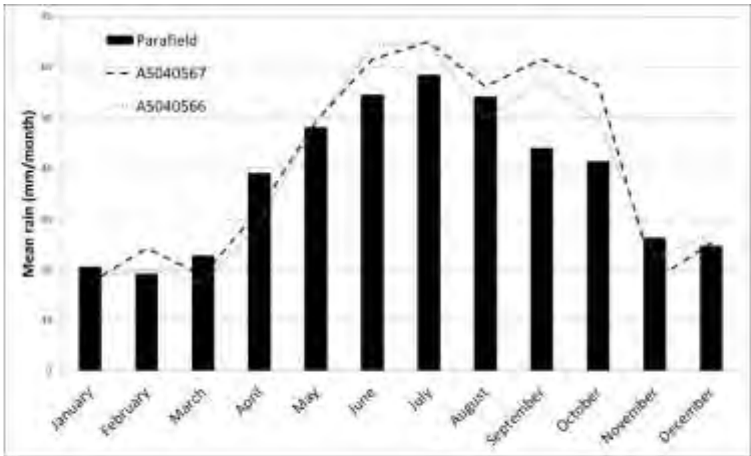


Figure C 3 – Mean monthly rainfall at the Parafield Airport BOM gauge (023013) and for the two Frederick Street catchment gauges

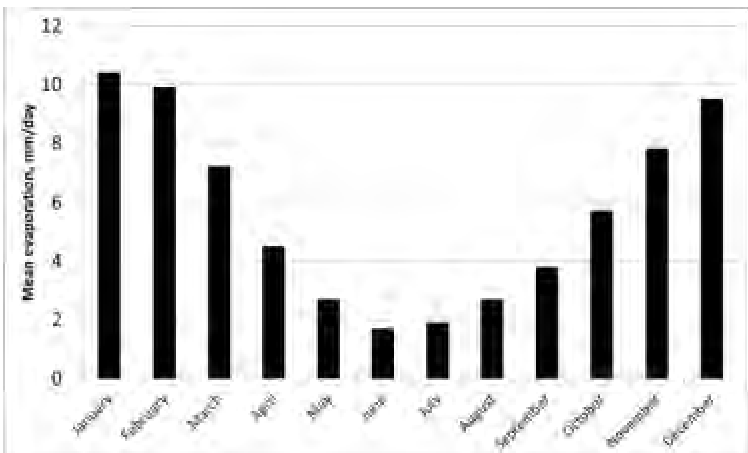


Figure C 4 – Mean daily evaporation at the Parafield Airport BOM gauge (023013)

Observed flow for the Paddocks catchment was available from the flow gauge at the entry to the Paddocks wetlands (A5050546, Table C 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.

<u>Other Assumptions</u>

- 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).

Model Calibration - Approach

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. Visual examination of the simulated and observed hydrographs was also supported by quantitative analysis. 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, 2008). 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 from the ASCE Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee and Irrigation Drainage Division (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 Task Committee on Definition of Criteria for Evaluation of Watershed Models of the Watershed Management Committee and Irrigation Drainage Division (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.

Model Calibration – Paddocks

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 C 4.

Table C 4 – 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 C 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 further below.

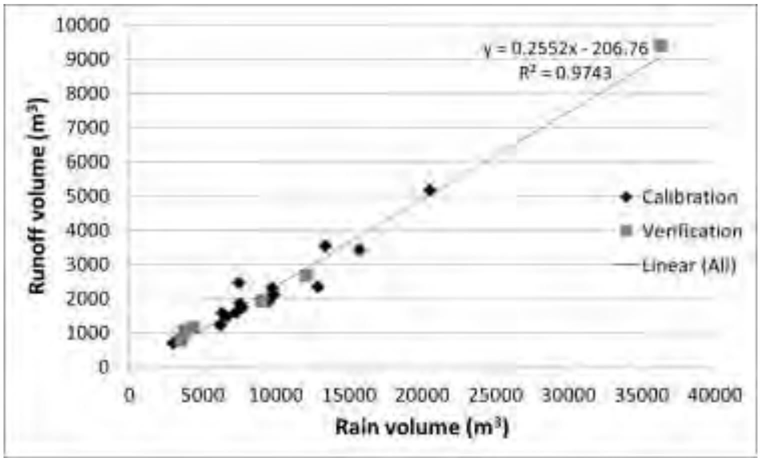


Figure C 5 – Comparison of observed rainfall and runoff volumes for calibration and verification events in the Paddocks catchment

A detailed explanation of catchment parameters used in SWMM modelling 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 C 5.

Table C 5 – 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 shown in Table C 4 are presented in Table C 6. 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 C 6 – 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

Samples of the fit of the simulated to the observed hydrograph is illustrated in Figure C 6 and Figure C 7.

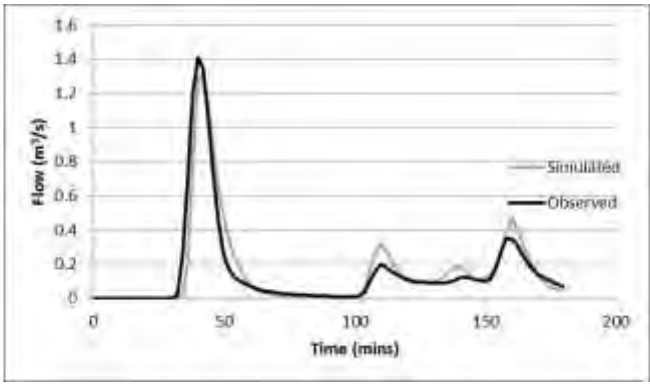


Figure C 6 – Comparison of the observed and simulated flows for Paddocks event Cal 1 (3 October 1992)

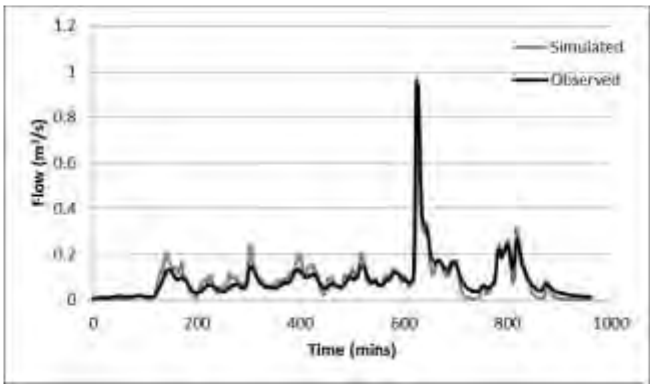


Figure C 7 – Comparison of the observed and simulated flows for Paddocks event Cal 2 (8 October 1992, #1)

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 C 7.

Table C 7 – 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 C 8, with selected hydrographs shown in Figure C 8 and Figure C 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 C 8 – 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

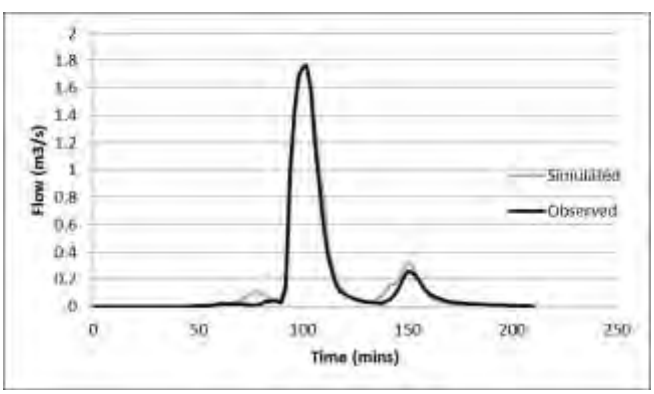


Figure C 8 – Comparison of the observed and simulated flows for Paddocks event V1

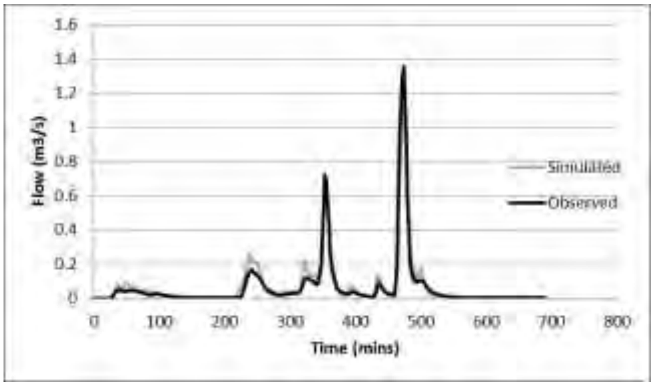


Figure C 9 – Comparison of the observed and simulated flows for Paddocks event V4

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 C 10.

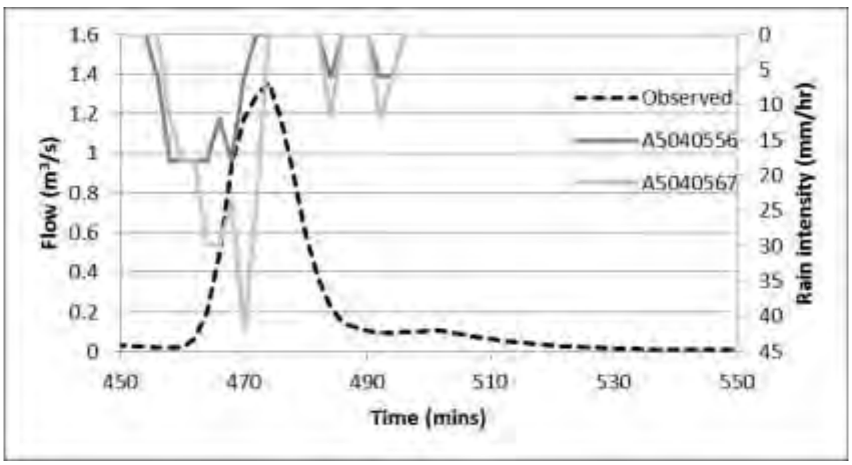


Figure C 10 – Excerpt of event V4, showing the suspicious cessation of rainfall at one gauge

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.