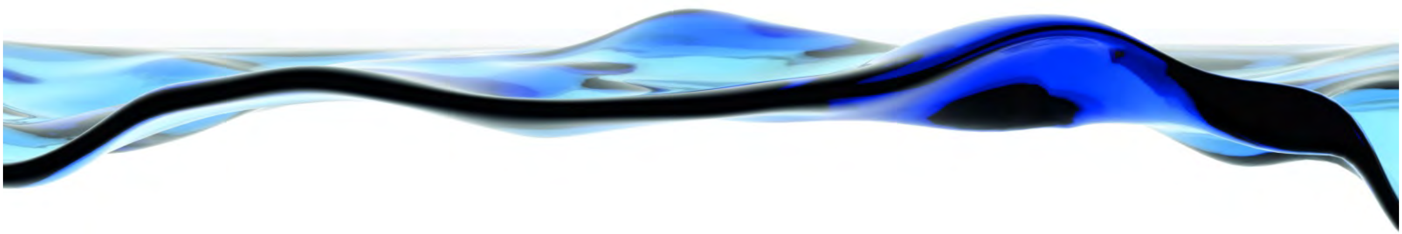
Hydrological Modelling of the Parafield Harvesting Scheme Catchment for Hazard Analysis Planning



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Executive Summary

The Parafield stormwater harvesting and Managed Aquifer Recharge (MAR) facility is operated by the City of Salisbury. The facility harvests surface runoff from the Parafield, Ayfield and Cobbler Creek catchment areas. Water is treated in detention ponds and wetlands before being stored in an aquifer and recovered for reuse. Water from the facility has a number of applications, including open space irrigation and distribution to industrial consumers. Unlike conventional water supply systems which tend to harvest water from protected natural or rural landscapes, the Parafield harvesting scheme collects water downstream of an urban environment with a conventional 'pit and pipe' drainage system. As such, the harvesting system is exposed to the deleterious impacts of residential and industrial development on runoff water quality. In this study, the potential for risk management strategies are explored for point source pollution events based on simulating the time of travel of hazardous spills and sewer overflows occurring in the catchment area of the scheme.

Based on a review of appropriate modelling tools, the PCSWMM model (a commercial variant of the EPA SWMM model) was selected as an appropriate tool to assess the transport characteristics of pollutant spills for the Parafield scheme based on the ability to model at a short timestep and for a well-developed flow routing capability. A model of the Parafield and Cobbler Creek catchment was developed based on spatial mapping data of key infrastructure, sub-catchments and interpolation where data were not available. A field study was also undertaken to measure the time of travel for a 50 L/s flow from the furthest reach of the catchment (Cobbler Creek pump station) to the flow gauge at the end of the catchment, indicating a total travel time of 90 minutes for the 6800 m distance. The model was then calibrated using observed flow data (for surface runoff properties) and the measured catchment time of travel (to calibrate conduit roughness). The main sources of error in the model included: the inability to calibrate predicted runoff flows in the Cobbler Creek catchment because it was ungauged; the limited ability to calibrate flows in the Parafield catchment because only a single flow measurement weir was available to provide observed flow data, and; rainfall data may not accurately represent rainfall in the catchment because only two rain gauges were available which were both near to but outside the catchment boundaries.

Based on an existing catchment hazard identification procedure (Page *et al.*, 2011), 18 locations in the Parafield and Cobbler Creek catchment were identified which presented a very high risk for pollutant spills and overflows. Pollutants of concern included pathogens, inorganic chemicals, organic chemicals, salinity and sodicity, nutrients and turbidity. Preliminary modelling was then undertaken to determine the most effective approach to simulating dry weather pollutant spills using PCSWMM. This process revealed that the most effective way to simulate pollutant spills was by assuming a liquid pollutant spill (CONCENTRATION) rather than assuming a dry pollutant mass (MASS). This process also revealed that high levels of water quality and flow routing errors were produced when a spill was simulated to occur without some constant baseflow to carry the pollutant downstream. Based on these findings, pollutant spills were assumed to occur as a liquid pollutant with a small baseflow, which produced more acceptable model routing errors and more conservative (shorter) predicted travel times.

Simulation of dry weather pollutant spills indicated that the travel time through the catchment was largely dependent on distance from the catchment outlet. In the Parafield catchment, the travel

time of a pollutant spill was found to be from 30 to 84 minutes, indicating a limited time to respond to spills in dry weather. Spills may therefore proceed through to the instream basin requiring cleanout. For the Cobbler Creek catchment, the travel time of pollutants from the point of the spill to the Cobbler Creek retention basin was from 18 to 155 minutes. In this case, there remains the possibility to disable the pump which diverts water from Cobbler Creek into the Parafield catchment in the event of a rainfall event following a pollutant spill in dry conditions. This will restrict the pollutant outflow from Cobbler Creek to the Parafield catchment.

Before undertaking wet weather pollutant spill simulations, preliminary modelling was undertaken to determine the most conservative assumptions for a series of variables including the assumed rainfall characteristics, the timing of a spill and the duration of spill. It was shown that larger rainfall events tended to carry a pollutant faster to the catchment outlet, but with higher levels of dilution. Faster travel times also resulted when a spill was assumed to occur in the middle of a storm compared to the beginning, and that high intensity, short duration storms produced a faster travel time than longer duration storms. Based on these results, two short duration storms (the half hour duration, three month and ten year ARI design storm for the Adelaide region) were selected to represent wet weather conditions for spill analysis with pollutant spills occurring at high risk locations in the middle of the storm event.

The simulated catchment travel time for wet weather pollutant spills was found to be even faster than the dry weather spills, with a minimum travel time of between 18 and 31 minutes in the Parafield catchment. The peak pollutant concentration at the outlet in each case was reached between 36 and 54 minutes after the spill, respectively. For the Cobbler Creek catchment, the travel time was between 12 to 43 minutes for the high risk locations closest to and furthest from the catchment outlet, respectively, and the peak concentration reached the detention basin within 24 to 78 minutes. These results indicate a limited window to allow human intervention to respond to divert all or the majority of the pollutant spill from reaching the instream basin of the Parafield harvesting scheme. Cobbler Creek flows may still be diverted during wet weather, however the Cobbler Creek pump operates automatically restricting diversion to a human intervention following a spill in this region. While pollutant dilution occurred in the catchment, there was limited time for it to occur sufficiently and peak pollutant flows occurred only short periods of time after the pollutant spill first became apparent at the catchment outlet.

Based on the observed travel time of 90 minutes from the Cobbler Creek pump, and the shorter simulated travel times for pollutants during wet weather flows at various high risk locations throughout the Parafield and Cobbler Creek catchments, it was considered that human intervention alone may be inappropriate for the management of pollutant spills due to limitations in communication of the spill occurrence and response time. Additional avenues are recommended to improve spill management. These include conducting appropriate and up-to-date hazard analysis within the catchment boundary, and the installation of online monitoring systems for parameters where technology allows (at present, this would typically include pH, turbidity and salinity). Hazard analysis is recommended as a means to remain alert to prevent or minimise the influence of hazardous events from occurring. Monitoring would allow for diversion of polluted water from the harvesting system or for the consideration of water treatment processes for harvested water so that it is fit for intended use.

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1 Introduction

Water quality is a significant consideration in the harvest and reuse of stormwater from urban catchments. Urban stormwater can be contaminated by both point and diffuse sources of pollution which vary depending on several factors, particularly landuse (Duncan, 2005). While typical urban stormwater pollutant loads have been widely reported with respect to landuse (Duncan, 2005; NRMMC, EPHC and NHMRC, 2009) stormwater harvesting schemes must also consider risks relevant to the occurrence of hazardous events causing temporary but highly concentrated point-source pollution. These events, such as road accident spills and sewer overflows, have received a high residual risk score in preliminary studies of hazard analysis and critical control points planning for stormwater harvesting schemes (Swierc *et al.*, 2005).

In this study, the potential for risk management strategies are explored for point source pollution events based on simulating the time of travel of hazardous spills and sewer overflows occurring in the catchment of an operating stormwater harvesting and reuse scheme in Salisbury, South Australia. Throughout this report, the term 'time of travel' refers to the time taken for water to flow from one part of the catchment to another. The 'time of travel' term is used in this study to avoid confusion with the 'time of concentration' term used in hydrology to refer to the time take for flow from the furthest point in a catchment to reach the outlet. The 'time of travel' differs from this concept as it refers to the time taken for water to travel from a specific point in the catchment to the catchment outlet (the point of harvest).

The Parafield stormwater harvesting and Managed Aquifer Recharge (MAR) facility is operated by the City of Salisbury. The facility harvests surface runoff from the Parafield, Ayfield and Cobbler Creek catchment areas. Water treatment is provided as it passes through the holding ponds and reed bed before being stored in aquifers and recovered for reuse. Some water is also delivered directly from the wetland to the consumer. The recovered water is also blended with recycled wastewater and reticulated via 'purple pipe' to households in Mawson Lakes where it is used for toilet flushing and garden watering. The scheme is linked to a network of other stormwater harvesting schemes in the City of Salisbury by a ring main.

Unlike conventional water supply systems which tend to harvest stormwater from protected natural or rural landscapes, the Parafield harvesting scheme collects water downstream of an urban environment with a conventional 'pit and pipe' drainage system. As such, the harvesting system is exposed to the deleterious impacts of urban environments to runoff water quality.

The goals of this particular project follow those of previous work on hazard analysis and planning of the stormwater harvesting scheme at Parafield (Swierc *et al.*, 2005; Page *et al.*, 2010). The goals of the project are:

- to develop and calibrate a rainfall runoff model for the Parafield facility catchment, that can simulate drainage hydraulics;
- to undertake modelling to quantify the impact caused by various pollutant event types; and
- develop and assess the effectiveness of catchment risk management controls.

1.1 Project Activities

The following activities were undertaken in this project:

1. Development of a model in EPA SWMM for simulating surface runoff characteristics, drainage network hydraulic and pollutant conveyance of the Parafield wetland water harvesting scheme catchment, including the Cobbler Creek catchment
2. Calibration and verification of modelled rainfall runoff characteristics, including flows and water volumes, within the catchment
3. Use of the EPA SWMM model to examine and quantify the impact caused by various pollutant event types and the effect of potential catchment risk management controls. This will include an assessment of the catchment response to adverse water quality events (such as pollutant 'spills' or diffuse pollutant build-up and wash-off) and their potential to impact on the harvesting system operation and performance.
4. Identification and assessment of catchment based risk management strategies in the EPA SWMM model (including cost estimates). These may include implementation of water sensitive urban design measures, conventional water treatment measures, first flush diversion(s) or warning systems within the catchment
5. Presentation of the model to project stakeholders and preparation of a final report detailing the model components and recommendations.

2 Background Information

2.1 The Parafield Stormwater Harvesting Scheme

The catchment of the Parafield stormwater harvesting facility is shown in Figure 2-1. The catchment was identified using geospatial information by overlaying the location of stormwater pipes in the Adelaide metropolitan area (from the South Australian Department for Water[1]) onto aerial photographs and mini-catchment data provided by the City of Salisbury (pers. comm.). The total area of the catchment was found to be approximately 2300 Ha, of which not all is harvested in full as detailed in the following sections. For the purposes of this report, the catchment has been split into two components: the Parafield catchment (to the west of Figure 2-1) and the Cobbler Creek catchment (to the east of Figure 2-1).

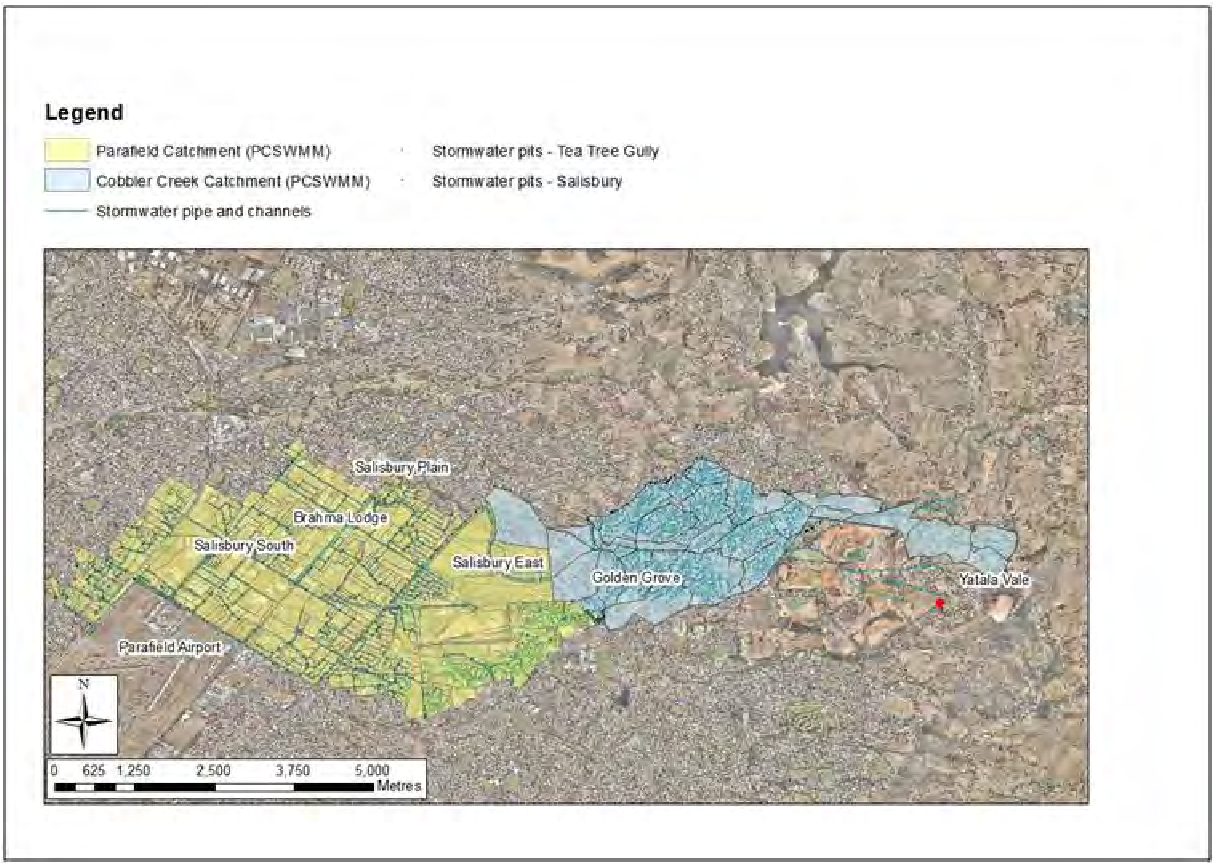


Figure 2-1 - Catchment area of the Parafield wetland scheme. The Parafield catchment is shaded yellow and the Cobbler Creek catchment is shaded blue

2.2 The Parafield Catchment

The Parafield catchment is approximately 1602 Ha in size and predominantly located in the City of Salisbury local government area (LGA) with some areas to the east located in the City of Tea Tree Gully LGA. Suburbs within the catchment boundary include Salisbury, Salisbury East, Salisbury South, Brahma Lodge and Parafield. The catchment is predominantly residential in character, but includes

[1] See https://www.waterconnect.sa.gov.au/WaterResources/SurfaceWater/Pages/default.aspx

some land zoned for commercial, industrial and rural living use. The catchment also includes some quarrying activity, and area reserved for the municipal open space system (Salisbury [City] development Plan, July 2010). All of the stormwater runoff which occurs in this catchment is harvested by the Parafield stormwater harvesting scheme.

Information on the nature of development in this catchment was largely sourced from spatial information developed by Cardno Willing NSW (2008) for the City of Salisbury in the form of 'mini-catchments' data. This data included sub-catchment areas linked to individual stormwater drainage pits in the catchment. It also included details on percent imperviousness and mini-catchment dimensions (used to estimate width).

2.3 The Cobbler Creek Catchment

The Cobbler Creek catchment refers to the catchment of the Cobbler Creek flood mitigation dam constructed in 1997 to mitigate flows from urban development in the area of Golden Grove. The Cobbler Creek catchment is approximately 697.8 ha in size (excluding the large area of quarrying activity, as discussed below). Since 2010, additional flows have been pumped into the Parafield catchment from the Cobbler Creek dam outflow, providing additional runoff to the Parafield. Water is sourced from the Cobbler Creek flood retention basin shown in the aerial photograph in Figure 2-2.

Figure 2-2 – Aerial photograph of the Cobbler Creek retention basin, indicating where flows are pumped into the Parafield catchment on the left and the temporary detention basin on the right (image courtesy of Googlemaps, 2010)

Flows are automatically pumped into the Parafield catchment on the basis of depth in the storage basin. When the height of the water in the concrete bund area of the basin reaches 3 m above the intake invert, flows are automatically released from the basin and the catchment transfer pump will automatically switch on. According to City of Salisbury, the flow rate of the pump picking up flow from the dam is approximately 50 L/s (or 0.05 m^3/s) (personal communication, City of Salisbury). Outflow not captured by the diversion pump proceeds along Cobbler Creek until it enters a concrete drain at the intersection of Barndioota Road and Lolands Road where it continues to the Little Para

River (north of the Parafield catchment boundary). As such, not all runoff from the Cobbler Creek catchment area is harvested.

The Cobbler Creek catchment includes a large portion of residential development, and a significant area of quarrying activity and agricultural production in the east. Quarrying activity is undertaken by four licenced operators including Rocla Pty. Ltd., Clay and Mineral Sales Pty. Ltd., CSR Ltd. and Austral Pty. Ltd. Observations in the Parafield and Cobbler Creek catchment have indicated that flows from Cobbler Creek are higher in turbidity than those in the Parafield catchment. This was originally assumed to be attributable to the quarrying activity within this area. According to the City of Tea Tree Gully, however (pers. comm.), this turbidity is most likely attributable to:

- ‘dragout’ material, material which is attached to truck wheels which is deposited onto local roadways by quarry traffic
- Spilled material from loaded quarry trucks
- Development, including a residential subdivision north east of the Crouch Road/Golden Grove Road intersection. This development is nearing its final stages but has a portion of disturbed soil which will no longer be developed. The site has instituted a renewed runoff management plan which should lead to improved water quality.

City of Tea Tree Gully indicated that the quarrying activity is almost all isolated from the catchment as Cobbler Creek flows are intercepted to the East of One Tree Hill Road, Golden Grove, and are diverted to a high capacity storage basin (approximately 185 m deep with a capacity in excess of 300 ML) on a site operated by Austral Bricks (PM32) between One Tree Hill Road and Ross Road. This system is designed to intercept flows of up to the 5 year ARI storm flow. A combination of evaporation, seepage and reuse (within the Golden Grove Extractive Industry Zone) was expected to ensure the pit does not overflow (pers. comm., City of Tea Tree Gully). As such, the quarrying activity and portions of the agricultural land east of these areas which drain to Cobbler Creek (based on contour maps) have been excluded from the catchment area. Flows into Slate Creek were however included in the catchment area.

2.4 Existing Modelling Data

2.4.1 Previous Modelling

Previous investigations into the Parafield stormwater harvesting scheme have included modelling of the Parafield catchment rainfall runoff characteristics. According to RCA (2001), the long term average annual yield anticipated from the scheme (prior to the addition of the Cobbler Creek catchment) was 1100 ML/annum. Initial modelling however indicated a mean annual runoff volume of 1497 ML/annum, and a harvestable volume of 1025 ML/annum.

According to RCA (2001), the Parafield catchment is among the first gauged catchments in South Australia to include a significant area of industrial development. In modelling the catchment, RCA (2001) assumed a high efficiency of rainfall / runoff conversion without showing data to support this and may have overestimated the ratio impervious to pervious area for the industrial zone. This is especially the case for some areas zoned industrial which are actually not significantly paved (such as the former Bridgestone site in the South East of the catchment, west of Cross Keys Road).

RCA (2001) also provide important data on the development of the scheme. In preparation for the Parafield stormwater harvesting scheme, for example, adjustments were made to the stormwater pipe network to include the Ayfield catchment (at the south eastern edge of what is referred to wholly as the Parafield catchment in this report). The nature of these adjustments will result in spillages of water out of the Parafield (including the Ayfield) catchment area. According to RCA (2001):

"The invert of the entrance to the Kings Road drain is approximately 1 m higher than the invert of the Ayfield drain. A combined weir/venturi system has been devised to lift the Ayfield runoff to a level where the majority of it can enter the Kings Road drain and contribute to the scheme input. When the flow rate in the Ayfield drain exceeds the capacity of the Kings Road drain (calculated at 6.9 m^3/s), spill will occur over a side weir and part of the inflow will escape down the existing drainage path (i.e. along the SE side of the airfield)."

Using stormwater pipe data updated for Salisbury City Council in 2009 this spill point is visible at the intersection of Main North Road and Kings Road. Overall, RCA (2001) estimates that spillage was expected to reduce the amount of water harvested from the scheme by about 4% of the overall Parafield/Ayfield catchment yield. This was not considered to be a 'loss' to the harvesting system however; when flow from the Ayfield catchment exceeded the capacity of the adjusted drain, it was assumed that the total Parafield/Ayfield catchment runoff would be in excess of the flow capacity of the diversion weir at the inlet to the instream basin.

RCA (2001) predicted losses from the Parafield stormwater harvesting scheme to be several fold, and provided estimates in the order of those outlined in Table 2-1.

Table 2-1 – Prediction of losses from the stormwater harvesting scheme operations (RCA, 2001)

Harvestable Water Loss Description	**Approximate Volume (ML/a)**
From the Ayfield overflow weir pipe adjustment	14
Rejected flows with salinity > 1000 mg/L (as TDS)	16
Spill from instream basin inlet	277
Evaporation from instream basin	2
Evaporation from holding storage	5
Evaporation from reed bed/wetland	28
Average retention lost via aquifer processes	130
TOTAL	**472**

RCA and CWMR (2009) subsequently reviewed the flow predictions of the Parafield scheme stormwater model produced by RCA (2001). The review conceded that flow estimates were generally accurate on a per unit area developed basis, however the levels of development assumed within the catchment were over-predicted (but expected to eventually reach the assumed levels). The effects of climate change were also not assessed in the runoff modelling.

In addition to the catchment runoff volume modelling undertaken by RCA (2001) and RCA and CWMR (2009), a simplistic Parafield catchment yield analysis was conducted by Swierc *et al.* (2005). The authors determined an approximate relationship between daily rainfall and daily runoff measured in the Parafield drain using available flow data. This analysis indicated that for every 1 mm of rainfall reported at the site per day, approximately 2.33 ML of runoff was produced at the flow gauge at the end of the catchment, corresponding to a runoff coefficient of 0.145. However, this analysis may have included a period of flow data of questionable quality (see Section 2.4.7).

2.4.2 Water Quality

According to RCA (2001), salinity of the stormwater runoff was anticipated to be approximately 50 mg/L (as total dissolved solids, TDS) during high flow events, and up to 5000 mg/L (as TDS) during some low flow events. These results were based on a relationship applied in the WaterCRESS model between salinity and daily flow.

There is a limited set of sample data on the water quality of runoff from the Parafield catchment (Page *et al.*, 2010; Swierc *et al.*, 2005). In addition, water quality at selected sites within the Parafield catchment has also been subjected to grab sampling (Page *et al.*, 2011).

2.4.3 Operational Data

The following operational data were considered necessary considerations during the compilation of this report and are included for the benefit of those undertaking future works on modelling and risk management of the Parafield scheme:

- City of Salisbury does not harvest stormwater from the Parafield scheme during the summer months. From sometime in approximately October to March (the dates are not fixed), harvest is discontinued because of water quality concerns. As such, the scheme is only effectively operating for six to seven months of the year.
- There is no diversion weir for water entering the basin to be rejected – the only option for rejecting water at the inlet of the basin (at time of writing) is for water to enter the instream basin and be subsequently pumped out into the Parafield drain.
- In some circumstances, water is supplied directly to customers after treatment in the wetland (prior to injection in the aquifer). Customers for this water include a variable component of both the City of Salisbury irrigation demand and Michell Wool Pty. Ltd. As such, the overall harvest volume of the scheme cannot be accurately determined by measuring the aquifer injection and extraction volumes. According to City of Salisbury (pers. comm., 2011) this component of demand is decreasing because minimum requirements for water quality are encouraging City of Salisbury to adopt a period of aquifer storage before supply to a greater proportion of customers
- In some circumstances, water has been extracted from the aquifer and directed back through the wetland for the purposes of flushing the wetland. This extraction and re-injection of harvested water also interferes with the determination of annual harvest volumes based on injection and extraction volumes
- According to RCA (2001): *"Runoff from the catchment is diverted from the Parafield drain by a weir constructed across the drain path. Diversion is into the 49 ML 'in-stream' basin via a series of seven 1050 mm culverts. The top water level of the in-stream basin is governed by the weir spill level. The flow transfer rate of these culverts (when the depth of stored water in the in-stream basin is less than 1.15 m and thus not causing back-up in the drain) is equivalent to a 1 in 10-year peak flow rate in the drain."*

- The maximum inflow capacity to the storage basin at Parafield is approximately 6.9 m^3/s – flows above this value are expected to overflow at the diversion weir and proceed downstream to dry creek.

It is recommended that operations management rules such as these are reviewed in detail when considering future hazard analysis works for urban stormwater harvesting schemes.

2.4.4 Catchment characteristics

Characteristics of the catchment for modelling purposes were determined manually, or extracted from pre-existing data sources. Pre-existing data sources included digital maps of the catchment which were viewed in ArcGIS or, where possible, imported directly into PCSWMM. Details of the characteristics of the catchment for SWMM modelling purposes are described in Appendix A including references to the relevant source material.

2.4.5 Rainfall

There are no rainfall gauge locations located within the boundaries of the Parafield catchment. However, there are several locations close to the catchment. A list of stations within 5 km of the Parafield catchment boundary which have current or recent records available in sub daily time increments is provided in Table 2-2. A list of stations within 5 km of the Parafield catchment boundary which have current or recent records available in daily format is provided in Table 2-3. Figure 2-5 indicates the location of these rainfall gauges relative to the catchment area. More information on rainfall gauges in South Australia can be located on the internet[2]

Table 2-2 – Details of sub-daily rainfall gauges located near the Parafield catchment

Station number	Lat (S)	Long (E)	Location	Manager*	Operation**	Elevation (m)
A5040567	34.8188	138.6440	Joslin Avenue, Ingle Farm	DfW	1992 – 2002	36
023013	34.7977	138.6281	Parafield airport	BOM	Aug 1972 – p	9.5
A5040547	34.8563	138.6843	Hope Valley Reservoir	DfW	1991 – p	100
A5040528	34.7535	138.7191	Little Para Reservoir Met	SA Water	1978 – 27/06/2011	149.2
023083	34.7111	138.6222	Edinburgh RAAF	BOM	Oct 1979 – p	17.1
*DfW refers to the South Australian Department for Water; BOM refers to the Australian Bureau of Meteorology ** 'p' refers to the present time						

[2] BOM website data,
ftp://ftp.bom.gov.au/anon2/home/ncc/metadata/lists_by_element/alpha/alphaSA_139.txt
e-NRIMS website data
http://e-nrims.dwlbc.sa.gov.au/SiteInfo/Default.aspx?site=A5040566

Table 2-3 – Daily rainfall gauges located near the Parafield scheme catchment

Station number	Lat (S)	Long (E)	Location	Manager	Data available	Elevation (m)
A5040566 523006	34.8245	138.6532	Leichardt Avenue, Ingle Farm	DfW BOM	1992 – p	88
023748	34.8307	138.7051	Adelaide (Tea Tree Gully)	BOM	1882 – p	145
023023	34.7674	138.6434	Adelaide (Salisbury bowling club)	BOM	1870 – p	32
023043	34.7607	138.6311	Salisbury (Halbury Road Alert)	BOM	1999 – p	25
023081	34.7715	138.5774	Bolivar treatment works	BOM	1972 - p	5
023026	34.8324	138.6125	Adelaide (Pooraka)	BOM	1878 – p	21
023858	34.7630	138.7765	Gould Creek (Hermitage)	BOM	1990 – p	350
023044	34.8395	138.6746	Valley View	BOM	2001 – p	90
023117	34.8368	138.6253	Walkley Heights (Bridge Road)	BOM	1996 – p	30
023096	34.8564	138.6844	Adelaide (Hope Valley Reservoir)	BOM	1979 – p	105
023116	34.8170	138.6985	Ridgehaven (Milne Road)	BOM	2001 – p	130
123700	34.8141	138.7730	Inglewood	BOM	2007 – p	372
023806	34.8067	138.7552	Upper Hermitage	BOM	1969 – p	390
023915	34.7539	138.7191	Gould Creek (Little Para)	BOM	1979 – p	155
023758	34.7398	138.8715	Kersbrook (Mabenjo)	BOM	1951 – p	282
023877	34.7863	138.8651	Kersbrook (Effluent Ponds)	BOM	1993 – p	350

The closest gauge to the Parafield catchment was considered to be Parafield airport (023013). It has a mean annual rainfall of 454 mm[3]. Figure 2-3 indicates the mean monthly rainfall at this gauge based on data collected by the BOM between 1929 and 2012. It clearly shows the occurrence of a wet winter period (April to October) and a dry summer period (November to March). For the purposes of this report, these two periods are referred to as winter and summer periods, respectively. These rainfall periods are referred to as winter and summer rainfall periods. However, rainfall records at this gauge were not considered suitable to represent the entire Parafield catchment because it is located at a comparatively low elevation (9.5 m) compared to some parts of the catchment which are located in the escarpment to the east of the catchment, which is higher than 200 m elevation at some points. Elevation was found to influence the annual amount of rainfall in the regions surrounding the Parafield catchment. To illustrate, the annual rainfall from 1960 to 1990 is shown across the catchment in Figure 2-4 based on gridded annual rainfall data from the Bureau of Meteorology. Similar patterns were shown using other gauges, but the data in Figure 2-4 shows a period which contained concurrent periods of daily rainfall data which has been quality controlled by the Bureau of Meteorology. Note that rainfall at Tea Tree Gully is higher in every year compared to gauges at lower elevation.

[3] http://www.bom.gov.au/climate/averages/tables/cw_023013.shtml

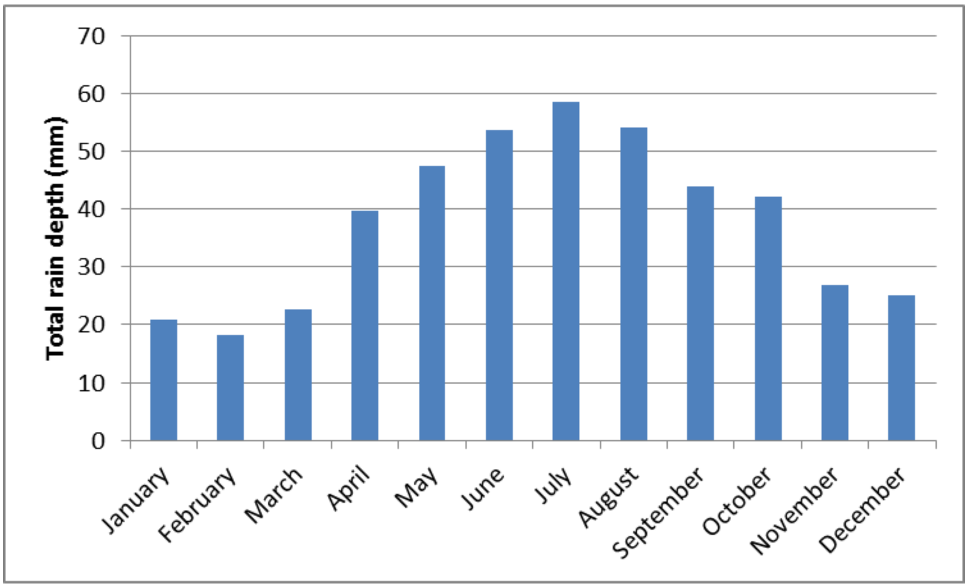


Figure 2-3 – Mean monthly rainfall based on observations at Parafield Airport (023013)

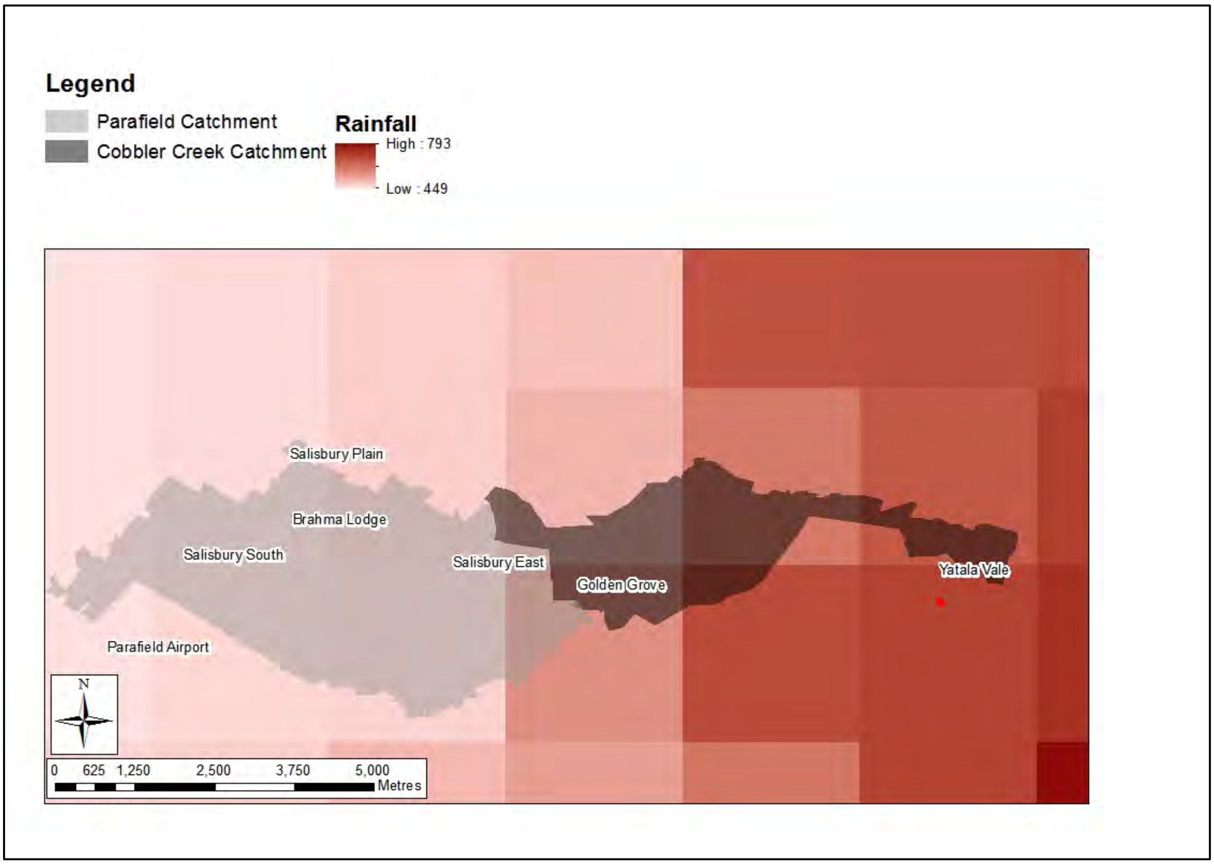


Figure 2-4 - Effect of elevation on rainfall in the Parafield catchment (based on mean annual rainfall from gridded climate data records)

Previous researchers have shown that there is little variation in rainfall data over smaller catchments based on a catchment of approximately 10 Ha in Tarrawarra, Southern Victoria, Australia (Western and Grayson, 1998). However, this cannot be assumed to be the case for the Parafield catchment, which is approximately 1602 Ha in size (excluding the additional 698 Ha catchment of Cobbler Creek). To capture rainfall variation cross the Parafield catchment using the available Pluviograph records, rainfall gauges were assigned on the basis of elevation and proximity to sub-catchments in the Parafield harvesting scheme area. All catchments which were located below 90 m elevation were considered to be represented by rainfall records at the Parafield airport (023013) rain gauge. Catchments at elevation greater than 90 m elevation were assumed to be represented by the Little Para Reservoir (A5040528) rain gauge. The mean annual rainfall at this gauge is 592 mm based on an analysis of daily rainfall data collected between 1999 and 2011. Rainfall records for Leichardt Avenue (A5040566) were also considered for use but had some low quality data patches in the calibration data period (2003 to 2005).

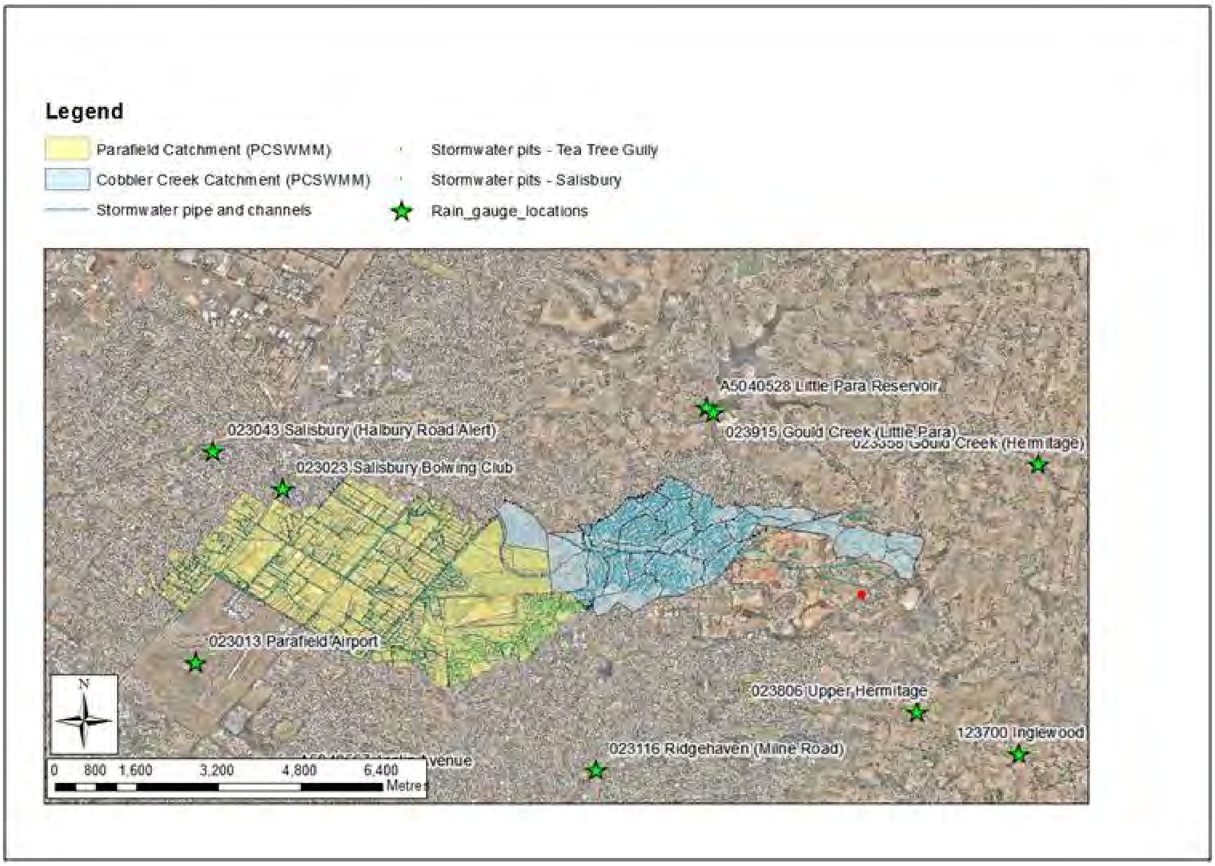


Figure 2-5 - Catchment area indicating the location of surrounding rainfall recording stations

2.4.6 Evapotranspiration

Monthly average aerial potential evapotranspiration (A-PET) was included in the model based on gridded average evapotranspiration metadata sourced from the Bureau of Meteorology[4] and was determined based on readings undertaken from 1961 to 1990. The evapotranspiration data is illustrated in Figure 2-6. The mean annual evapotranspiration was 1150 mm based on this information. The sensitivity of using evaporation pan data from the Parafield airport gauge and this evapotranspiration data was explored and it was found that there was little difference in the model

[4] Refer to http://www.bom.gov.au/climate/averages/climatology/gridded-data-info/metadata/md_ave_et_1961-90.shtml

results. When the volume and peak flow of a model run using monthly evapotranspiration data and monthly evaporation data between 1975 and 1989[5] was compared, less than 5% change in volume and peak flow was typically found for each event analysed in Section 3.4.

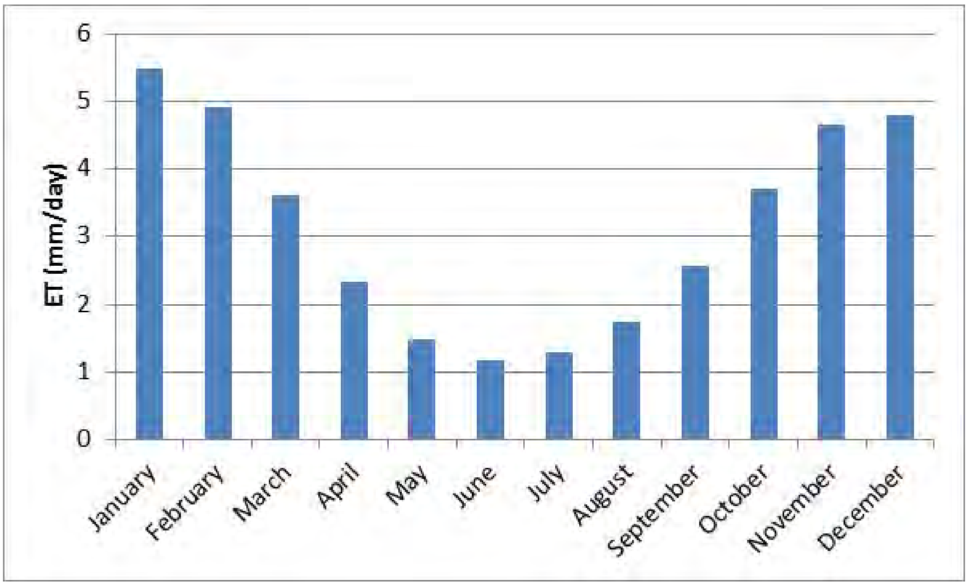


Figure 2-6 - Daily mean evapotranspiration values (mm/day) adopted in the PCSWMM model of the Parafield scheme

Other climate data such as temperature, wind speed, snow melt and aerial depletion were not considered in the Parafield model. This is because wind speed, snow melt and aerial depletion are all functions used for modelling snow melt only. Temperature is used for modelling snow melt and may also be used to determine an evapotranspiration function, however this was not required because monthly average data was used.

2.4.7 Observed Flows

Observed Flows in Parafield

A single v-notch weir has been installed near the end of the Parafield catchment by Water Data Services Pty. Ltd. The weir is situated in the drain adjacent to the western side of Parafield airport, approximately 450 m before the point of harvest. The weir measures flow from almost all of the catchment (or 1575 Ha of the 1602 Ha total of the Parafield catchment). The weir operates by measuring pressure head every six minutes in a pool created by a concrete v-notch in the Parafield drain. The weir has two names in historical data records:

- Gauge SC504902 (for readings between 2001 and 2006)
- Gauge A5041049 (for readings between 2011 and the present).

The gauge operated over a range of 0 to 5 m depth, and with an accuracy of 0.05% (i.e. approximately 2.5 mm). Flow is determined by reference to a flow depth rating table. The gauge initially operated between June 2001 and November 2006 before being decommissioned. According the RCA and CWMR (2009), there were several periods of data from the site which were questionable in data quality. These included:

[5] Available from the Bureau of Meteorology website: http://www.bom.gov.au/climate/averages/tables/cw_023013_All.shtml

1. A sustained period of low flows (approximately 3.5 ML/day) between 06 August 2001 and 12 June 2003 where flow occurred independently of rainfall; this was attributed to a wastewater discharge to the drain, evidenced by a photograph taken by Water Data Services personnel upstream of the weir. Further information on the nature of this discharge was not available.
2. Failure to measure flow in response to recorded rainfall events between
    a. October 2001 and August 2002
    b. November 2004
    c. January to May 2005
    d. October 2005

Due to uncertainty over the accuracy of this data, flow data was excluded for calibration and verification of the PCSWMM model during these periods.

The gauge was reinstated in November 2011 and flow data is being collected for the catchment. This flow data was not used in this report for modelling. However, it has been used in the calculation of catchment travel time in Section 3.3.1.

Observed Flows Outside the Catchment

There are a number of flow gauges operating near the catchment which may be used to quality control the data from the Parafield gauge. These gauges are presented in Table 2-4.

Table 2-4 – Location of flow gauges near the Parafield flow gauge (A5041049)

Number	Name	Status	Open	Close
A5040503	Little Para R US Fault	Open	06/05/1968	-
A5040504	Carisbrooke Park	Closed	06/05/1968	18/07/1984
A5040528	Little Para Reservoir	Open	08/06/1978	-
* Indicates that data is not available at the South Australian surface water archive[6]				

2.4.8 Catchment Controls

Limited catchment controls exist in some residential, industrial and quarrying areas of the Parafield and Cobbler Creek catchments. These are in the form of stormwater detention basins. The presence of these basins can be considered of benefit for risk management purposes because they will intercept spills and overflows before they reach the catchment outlet. However, these basins will need to be emptied or otherwise treated before all, or the bulk of, their contents is allowed to proceed downstream. The most obvious structural measures within the catchment consist of stormwater basins in residential areas and for runoff management of quarry sites. Figure 2-7 indicates the location of basins and their relative catchment area within the broader Parafield and Cobbler Creek catchment.

[6] https://www.waterconnect.sa.gov.au/SWA/Pages/default.aspx

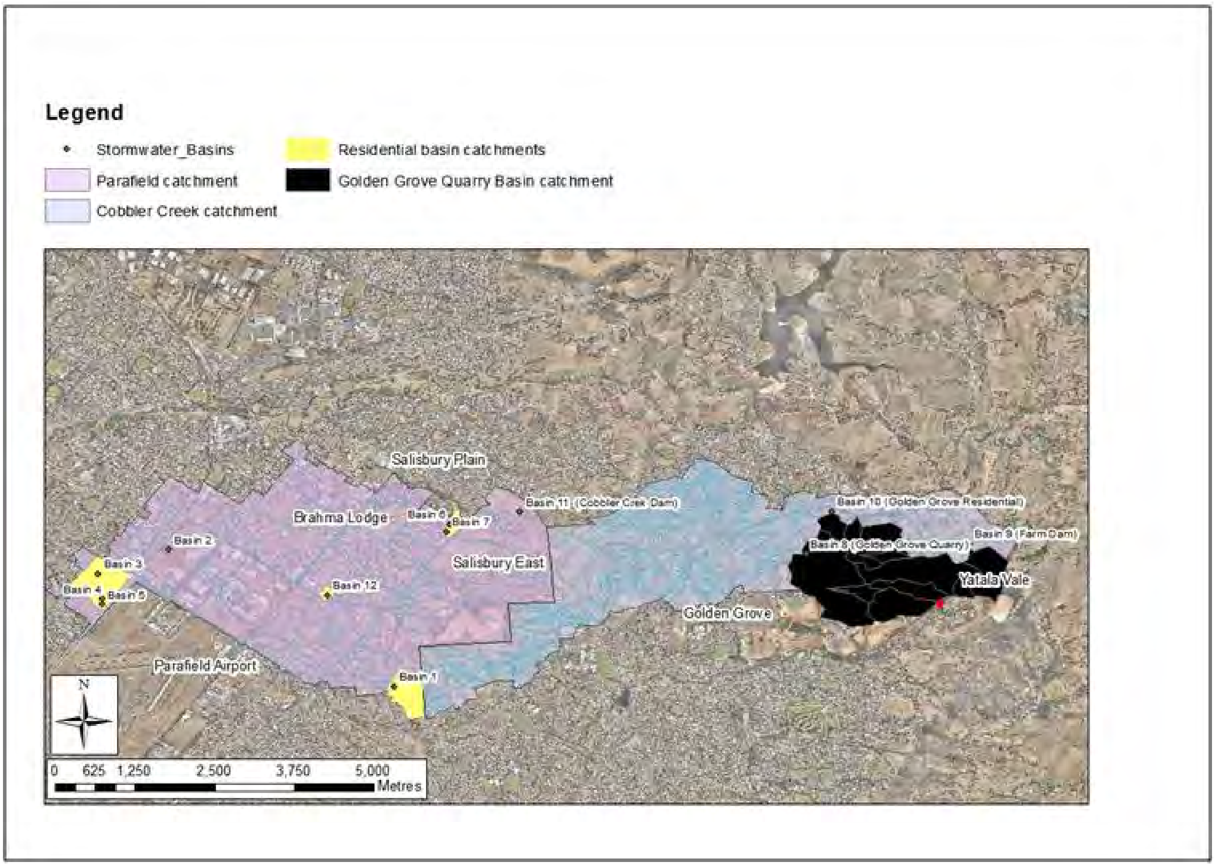


Figure 2-7 – Map of Parafield and Cobbler Creek catchments indicating the subcatchments which are connected to stormwater detention basins

For modelling purposes, the effects of the small residential detention basins has been ignored in this study, as there is limited data on their size, and their operation (i.e. at worst, case, these basins will be empty and will provide limited dilution before overflow in the event of contamination). The effects of the large stormwater basin for the quarry areas have been accounted for by the exclusion of the catchment area indicated in black in Figure 2-7 (see Section 2.3).

2.5 Summary

This section has described the existing data about the Parafield harvesting scheme and its surrounds that will be used in the development of a hydrological model. The most important data include the catchment areas of the Parafield and Cobbler Creek catchment, attributes of the catchment surface, rainfall, flow data and information about the stormwater drainage network. Additional data to that provided in this chapter is shown in Appendix A with specific reference to the hydrologic model properties. The following chapter describes the development of the hydrological model of the Parafield catchment.

3 Hydrological Modelling

3.1 Selection of a Hydrological Model

A comprehensive review of models suitable for examining the rainfall/runoff characteristics of the stormwater harvesting and reuse facilities in the City of Salisbury LGA was previously reported by CWMR and RCA (2010). The capability of several rainfall runoff models were reviewed including:

- WaterCress
- EPA SWMM
- MUSIC
- DRAINS
- StormNET
- MIKE Urban

The review produced a summary table of features available in each model for the purposes of a hazard analysis. This is reproduced in full in Appendix F with the addition of parameter consideration for the model PCSWMM. The review found that no model possessed the complete range of features considered relevant for assessing the operations of harvesting scheme modelling according to the requirements by CWMR and RCA (2011), but EPA SWMM and WaterCress possessed the greatest number of features. Since WaterCress was the only model capable of integrating stormwater, wastewater and mains water distribution into the one model, the review concluded that WaterCress should be adopted as a default tool for yield analysis for reporting to the Waterproofing Northern Adelaide Regional Subsidiary. However EPA SWMM was recommended where wastewater and mains water flows are not considered because of advanced hydraulic capability (see Appendix F).

For the purposes of this study, however, EPA SWMM has a greater hydraulic modelling capability than WaterCress, with a more rigorous rainfall/runoff and routing module for examining flows at short timesteps (less than one day). While both WaterCress and EPA SWMM can accept rainfall data with sub-daily timesteps (less than and including 6 minutes), only EPA SWMM can produce hydrographs at any requested timestep. WaterCress can only produce aggregated hourly flow volumes and a peak flow rate based on the sub-daily data. EPA SWMM also has a capability for both routing and simulating the flows and treatment of pollutants in runoff, which is not possible in the WaterCress model. WaterCress does incorporate a water quality rating for the distribution and end use decision making for water distribution, but the model does not specifically model pollutant transport. Based on the results of CWMR (2010) and the requirements of this project which will assess pollutant flows throughout the catchment culminating at the Parafield scheme, EPA SWMM was selected as an appropriate modelling tool.

It should be noted that EPA SWMM is available freely as an open source program from the United States Environment Protection Agency web page[7]. Commercial variants of EPA SWMM, based on the same hydraulic model, are also available which can assist hydraulic modellers by incorporating additional tools including the capability to import spatial information from GIS packages and enhanced calibration techniques. PCSWMM was selected as an appropriate model for this purpose. PCSWMM has been used in several published studies of rainfall runoff modelling (Smith, 2005).

[7] http://www.epa.gov/nrmrl/wswrd/wq/models/swmm/

3.2 Model Development

The development of a hydrological model of the Parafield and Cobbler Creek catchments in PCSWMM was conducted with reference to GIS data, consultation with City of Salisbury and site investigations. To construct the model, maps of stormwater pipes, stormwater pits and their contributing catchment area were sourced in GIS format. This information was used to determine the extent of the catchment. All drainage pipes that may contribute water to the Parafield Drain (including the Cobbler Creek catchment) were assumed to contribute runoff to the scheme. The boundary of the catchment was then approximated based on the boundaries of the catchment areas which feed these drains in the mini-catchment data supplied by the City of Salisbury. It should be noted that the catchment boundary that was identified for this project may be slightly different to that identified in previous studies, such as the Preliminary HACCP Plan constructed by Swierc *et al.* (2005) and the catchment yield analysis by RCA (2001). The basis for the catchment boundary used by these Swierc et al (2005) and RCA (2001) was not reported.

It was found that that the data provided by City of Salisbury and Department for Water was not wholly complete. In some circumstances, there was not enough data or data of low reliability to establish an appropriate link between model components. Common examples include the link between stormwater pits and pipes due to insufficient invert data, or catchments to stormwater pits. In these circumstances, values were approximated. The following rules were applied when approximating data in the model:

- Subcatchments without pit data were linked to the nearest and most probable pit based on invert level and location
- Where pipe or pit invert data was insufficient, pipes were assumed to link at the invert of the pit at each end. Where data was not available for pipe or pit inverts, values were approximated based on linear interpolation between the nearest pits with invert data.
- Where existing data was inappropriate, for example in cases where water was forced to flow uphill, values were replaced with linearly interpolated data of reliable data from nearby pit and pipe elevations.

When construction of the model was complete and the model could run successfully, calibration and verification of the model commenced by comparing simulated flows to observed flow data. During this process, some model parameters were considered fixed and others variable. A full list of PCSWMM parameters, their fixed or variable nature and values adopted for the calibrated model is provided in Appendix A.

3.3 Field Data

3.3.1 Measuring Catchment Time of Travel Values

To ensure that accurate time of travel data was used for the calibration of the PCSWMM model, the 'time of travel' was examined at key points in the catchment during a flow event. The field study on catchment 'time of travel' was conducted between the two points illustrated in Figure 3-1, namely the furthest point in the Parafield catchment (Pit-23138, location of the Cobbler Creek catchment transfer pump) to the Parafield drain weir (Gauge A5041049).

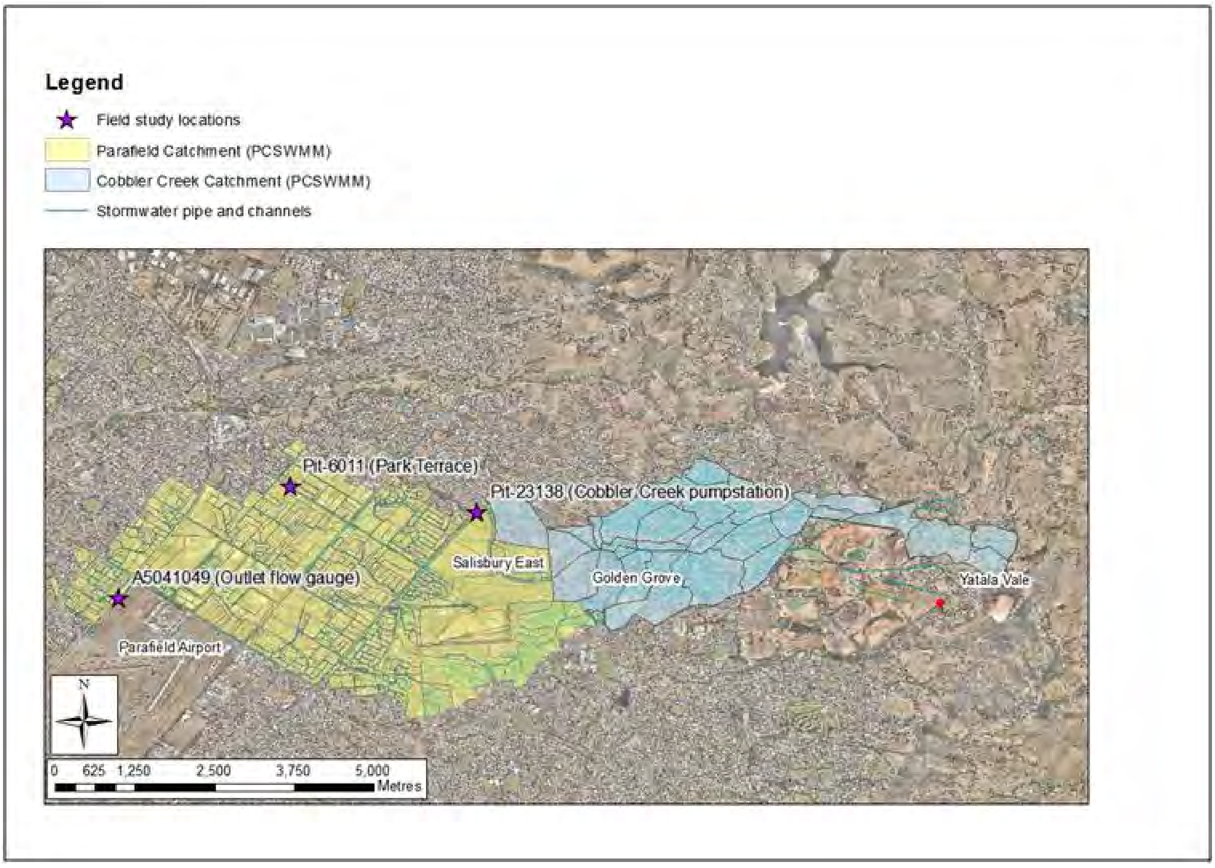


Figure 3-1 - Location of observation points for the catchment 'time of travel' field study

Through an experiment conducted in the field, time of travel was determined by manipulating flow from the Cobbler Creek pump station. As described in Section 2.3, the Cobbler Creek pump automatically switches on to capture a portion of the controlled outflow of the flood control structure spilling into Cobbler Creek. City of Salisbury allowed for the pump to be switched off and on manually for this analysis. As such, the flow path along the main drainage line from the Cobbler Creek pump was therefore able to be examined by using the pump, the gauge at the end of the catchment, and by direct observation of flows through pits where flows may be observed within the catchment.

The time of travel between each location indicated in Figure 3-1 is shown in Table 3-1. These travel time measurements were restricted to time of travel within the stormwater pipe network only. In addition, testing could only be undertaken in the days following rainfall, and the results were only applicable to flows through a 'wetted' pipe network through which water has recently flowed. As such, any losses through pipe leakage, depression storage (or ponding) are not considered using the results of this investigation.

Table 3-1 - Time of travel between key points in the catchment model

Run	From	To	Distance (m)[1]	Time (mins)
1	Pit-23138	Pit-6011	3260	40 [2]
2	Pit-6011	Gauge A5041049	3550	49 [2]
3	Pit-23138	Gauge A5041049	6810	90 [3]

[1] Determined using GIS maps of the stormwater network [2] Determined using a timing device to nearest minute; [3] Determined to nearest six minutes using flow gauge data

3.4 Calibration and Verification of the PCSWMM Model

3.4.1 Approach to calibration

The characteristics of stormwater pipes was calibrated using the pumped flow data in Section 3.3.1. Calibration and verification of the surface runoff characteristics of the PCSWMM model was undertaken by comparing measured flows from the end of catchment flow gauge SC504902/A5041049 (Section 2.4.7) with the simulated rainfall runoff flows generated using PCSWMM within the open channel where the flow gauge was constructed. The primary goal of the calibration was to achieve an accurate prediction of flow for the purposes of flow and pollutant transport modelling. Observed data variables considered in the calibration procedure included:

- the catchment 'time of travel'
- stormwater runoff volume
- stormwater runoff flow rate

Flow data was considered appropriate between June 2003 and January 2005 (Section 2.4.7). After January 2005, RCA and UniSA (2009) indicated the presence of inconsistencies in the flow data and this period was avoided.

Calibration and verification events were selected from the available data to capture a range of peak flows, flow durations and seasons (i.e. occurrence of rainfall in drier 'summer' months and wetter 'winter' months). It was also considered important to include events where the impervious area alone contributed to runoff, and where both pervious and impervious area contributed to runoff. This would typically be undertaken by selecting events with a short duration, where only the impervious area contributes to runoff, and events with a longer duration, where both the impervious and pervious area contribute to runoff. Using the initial modelling parameters, it was found that pervious area modelling parameters only influenced flow in events where the observed peak flow was greater than approximately 3 m^3/s. As such, for the purposes of this analysis, events with an observed peak flow less than 3 m^3/s were used to calibrate impervious area parameters. Events with an observed peak flow larger than 3 m^3/s were used to calibrate pervious area parameters.

Calibration and verification events were selected using the PCSWMM model by applying the automatic event selection function on observed flow data. To begin, the model was run with initial parameters for the period June 2003 to June 2005. Individual storm events were then selected and sorted from all events where the occurrence of *observed flow* had the following characteristics:

- Minimum inter-event time (time since previous flow) of 24 hours
- Minimum flow threshold of 0.001 m^3/s
- In some cases, event duration was manually adjusted to include modelled flow in excess of the observed flow.

The events selected for calibration and verification are shown in Table 3-2. Calibration was first undertaken for events which were identified to be responsive to impervious area runoff only. Calibration was then undertaken using larger storm event flows, followed by verification with other events across the range of peak observed flows. The model was calibrated by adjusting parameters of the model within limits considered reasonable by the SWMM model manual (Rossman, 2010) and with respect to the catchment itself. Further details on parameter selection is provided in Appendix A. Calibration was considered complete when the simulated flow data for a majority of the selected calibration events presented:

- A good quality fit with the observed flow rate data on a calibration plot
- A Nash-Sutcliffe calibration statistic greater than 0.7 (See Section 3.4.2)

Verification was carried out to ensure that a majority of the selected verification events satisfied these same criteria.

Table 3-2 – Characteristics of events used for calibration and verification of the PCSWMM model

No.	Date	Total rain (mm) [1]	Rain duration (hrs)	Flow duration (hrs) [2]	Peak flow (m^3/s)	Flow volume (m^3)	Description [3]
1	8/07/2003	1.4	0.5	6.5	0.731	2937	C, Winter, short duration
2	29/01/2004	0.8	0.4	11.5	0.102	569	C, Summer, short duration
3	15/05/2004	3.1	12.5	20.0	0.749	5650	V, Winter, long duration
4	22/05/2004	2.9	7.8	18.1	0.635	5024	V, Winter, long duration
5	1/06/2004	7.4	22.6	31.9	0.781	17290	V, Winter, long duration
6	21/02/2004	5.1	4.7	14.1	1.707	12090	C, Summer, short duration
7	22/04/2004	6.1	6.0	12.8	1.966	13360	C, Winter, long duration
8	29/04/2004	3.8	1.3	7.4	1.544	6576	V, Winter, short duration
9	8/07/2004	5.4	8.0	17.0	1.836	13790	V, Winter, long duration
10	3/07/2003	2.3	2.0	11.6	2.237	8873	C, Winter, short duration
11	23/07/2003	20.5	30.0	44.4	2.068	78920	C, Winter, long duration
12	28/05/2004	3.9	1.30	7.4	2.175	9286	V, Winter, short duration
13	11/07/2003	8.6	17.9	29.1	3.001	24130	C, Winter, long duration
14	9/06/2004	9.5	3.10	14.4	3.111	21820	V, Winter, short duration
15	11/06/2004	22.6	95.0	106.1	3.332	59250	V, Winter, long duration
16	4/11/2004	27.6	59.0	68.7	3.151	81080	V, Summer, long duration
17	18/06/2004	19.5	24.5	34.0	4.368	78370	C, Winter, long duration
18	23/06/2004	14.5	47.2	54.5	4.333	51500	C, Winter, long duration
19	3/01/2005	6.9	4.0	10.0	4.190	19960	V, Summer, short duration
20	26/06/2003	46.2	64.6	69.5	7.196	142300	C, Winter, long duration
21	6/12/2004	18.2	5.4	10.5	6.161	25770	V, Summer, long duration
22	8/12/2004	14.5	32.3	37.6	6.468	34050	V, Summer, long duration

[1] Event duration based on observed flow, [2] Rainfall at Parafield Airport gauge (023013), [3] 'C' indicates the event was used for calibration, 'V' indicates that the event was used for verification; Long duration is used to describe storms greater than 5 hrs.

3.4.2 Assessing Calibration Fitness

The most fundamental approach to assessing the calibration of a model is observing the 'fit' of modelled and observed data (Krause *et al.,* 2005). This is conducted by comparing observed and modelled data to determine the extent of differences between the two data sets (usually in the form of a graph). There are also a number of objective assessment tools available which attempt to measure the error between simulated and observed variables. Common objective assessments include (Ladson 2008; Wagener *et al.*, 2004):

- Sum of squared errors (SSE)
- Coefficient of efficiency or Nash-Sutcliffe criteria (NS)
- Root mean square (RMS)
- Sum of square roots (SSR)
- Sum of squares of differences (SSD)
- Sum of squared of differences of values raised to the power of 0.2
- Sum of absolute differences of the logs (SADL)
- Total volume (TV)

It should be noted that objective functions should be used cautiously. For example, Wagener *et al.* (2004) demonstrate that although these functions can assess how well a model output fits the observed data, these functions cannot interpret data visually – that is, two unique and visually different modelling solutions can present the same goodness of fit according to these objective functions. Selection of objective functions should also be undertaken with reference to the purposes of the model. For example, the SSE and NS values are typically more suitable for matching the requirements for peak flows and total volume of runoff, whereas the SSR, SSD and SADL are more appropriate for assessing a model's fitness for predicting low flows (Ladson, 2008).

Based on a review of available functions, the quality of the fit of simulated to observed data was assessed by visually assessing the plot of simulated and observed data and by reviewing the Nash-Sutcliffe calibration statistic, r^2. The Nash Sutcliffe calibration statistic was calculated by assessing paired values of simulated and observed values using the following equation (ASCE, 1993):

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

Where n represents the number of observed flow data (effectively the number of timesteps 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 Nash-Sutcliffe efficiency was selected because it is one of the most widely applied criteria to assess simulated and observed flow for hydrological models (Krause *et al.*, 2005). The Nash-Sutcliffe statistic is however known to be sensitive to the accuracy of peak flows (as opposed to low magnitude flows) as discrepancies are magnified to a greater extent.

In accordance with the recommendations of the ASCE (1993) for presenting data for single event runoff simulation, the simple percent error in peak (PEP, Equation 2) and the sum of squared residuals (G, Equation 3) are also presented for each event.

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

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

Where O_{peak} represents the observed peak flow during the event and P_{peak} represents the predicted peak.

3.4.3 Calibration Results

A summary of the simulation characteristics including the peak flow, total flow, and key efficiency criteria for each event is shown in Table 3-3. The results of the calibrated and observed flow are also presented for selected events in Figure 3-2 to Figure 3-6. A complete series of predicted and observed flow data plots are presented in Appendix B. There are some circumstances where the simulated data does not closely match the observed flow data. The possible causes for these discrepancies are presented in Section 3.5.

Table 3-3 - Summary of event characteristics and calibration statistics for events used for calibration and verification of the model

		Observed				Simulated		Fit			
No.	Date	Total rain (mm)[1]	Flow duration (hrs)[2]	Peak flow (m^3/s)	Flow volume (m^3)	Peak flow (m^3/s)	Flow volume (m^3)	Nash-Sutcliffe r^2	PEP (%)	G	Description[3]
1	08/07/2003	1.4	6.5	0.731	2937	0.369	2111	0.710	49.55	0.77	Winter, short duration
2	29/01/2004	0.8	11.5	0.103	569	0.134	674.2	0.837	-29.85	0.01	Summer, short duration
3	15/05/2004	3.1	20.0	0.749	5650	0.412	5727	0.592	45.06	1.66	Winter, long duration
4	22/05/2004	2.9	18.1	0.636	5024	0.461	5186	0.654	27.51	0.97	Winter, long duration
5	01/06/2004	7.4	31.9	0.781	17290	0.738	20270	0.795	5.58	2.36	Winter, long duration
6	21/02/2004	5.1	14.1	1.707	12090	1.831	14580	0.864	-7.26	3.27	Summer, short duration
7	22/04/2004	6.1	12.8	1.966	13360	2.078	14980	0.691	-5.71	8.04	Winter, long duration
8	29/04/2004	3.8	7.4	1.544	6576	1.694	7523	0.774	-9.73	2.74	Winter, short duration
9	08/07/2004	5.4	17.0	1.836	13790	1.730	13200	0.967	5.78	1.04	Winter, long duration
10	03/07/2003	2.8	11.6	2.237	8873	1.072	7077	0.641	52.07	8.66	Winter, short duration
11	23/07/2003	20.5	44.4	2.068	78920	2.255	77190	0.897	-9.03	10.32	Winter, long duration
12	28/05/2004	3.9	7.4	2.175	9286	2.267	9908	0.879	-4.23	2.88	Winter, short duration
13	11/07/2003	8.6	29.1	3.001	24130	3.566	24500	0.726	-18.82	14.36	Winter, long duration
14	09/06/2004	9.5	14.4	3.111	21820	4.621	27810	0.815	-48.54	15.59	Winter, short duration
15	11/06/2004	22.6	106.1	3.332	59250	4.195	63040	0.74	-25.91	37.33	Winter, long duration
16	04/11/2004	27.6	68.7	3.151	81080	3.850	87440	0.694	-22.18	43.58	Summer, long duration
17	18/06/2004	19.5	34.0	4.368	78370	3.798	66690	0.688	13.04	80.44	Winter, long duration
18	23/06/2004	14.5	54.5	4.333	51500	3.798	47200	0.873	12.34	18.998	Winter, long duration
19	03/01/2005	6.9	10.0	4.190	19960	4.107	18430	0.859	1.99	12.66	Summer, short duration
20	26/06/2003	46.2	69.5	7.196	142300	11.110	143800	0.752	-54.45	167.3	Winter, long duration
21*	06/12/2004	18.2	10.5	6.161	25770	12.470	48380	-2.1*	-102.4*	595.42*	Summer, long duration
22	08/12/2004	14.5	37.6	6.468	34050	9.363	39430	0.696	-44.76	69.09	Summer, long duration
*Characteristics of this event are discussed in Section 3.5.3, page 28.											

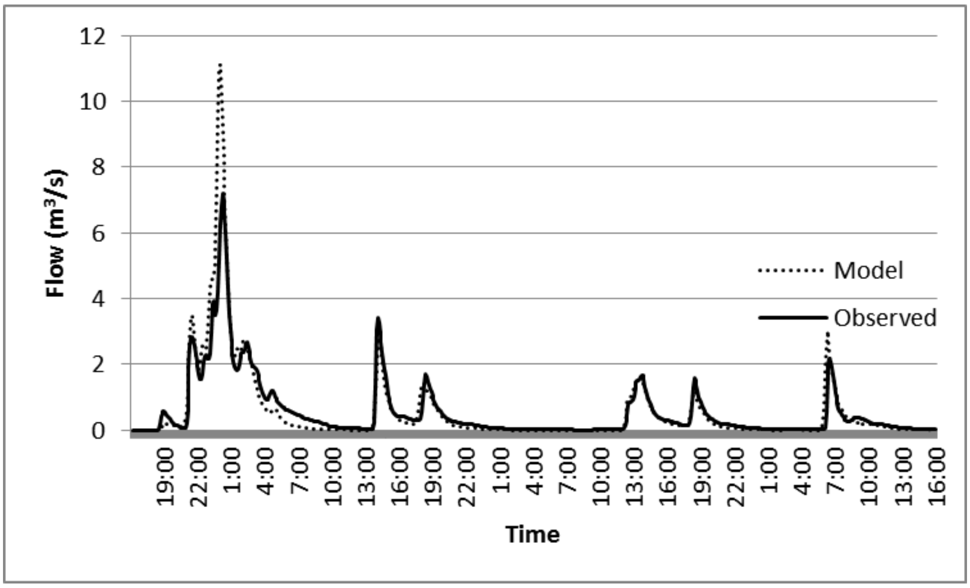


Figure 3-2 – Calibration for Event 20: 26 June, 2003 (Winter, Long)

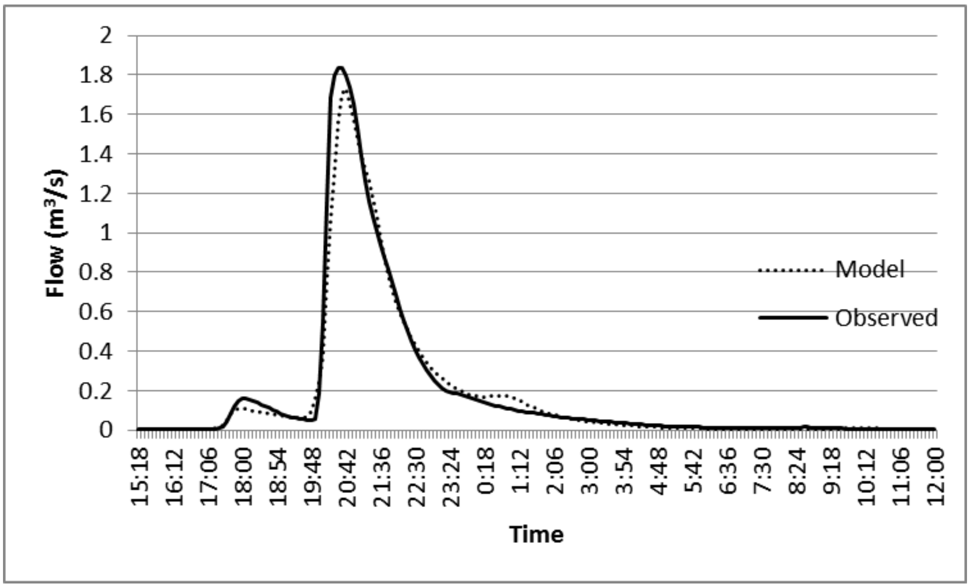


Figure 3-3 – Calibration for Event 9: 8 July, 2004 (Winter, Long)

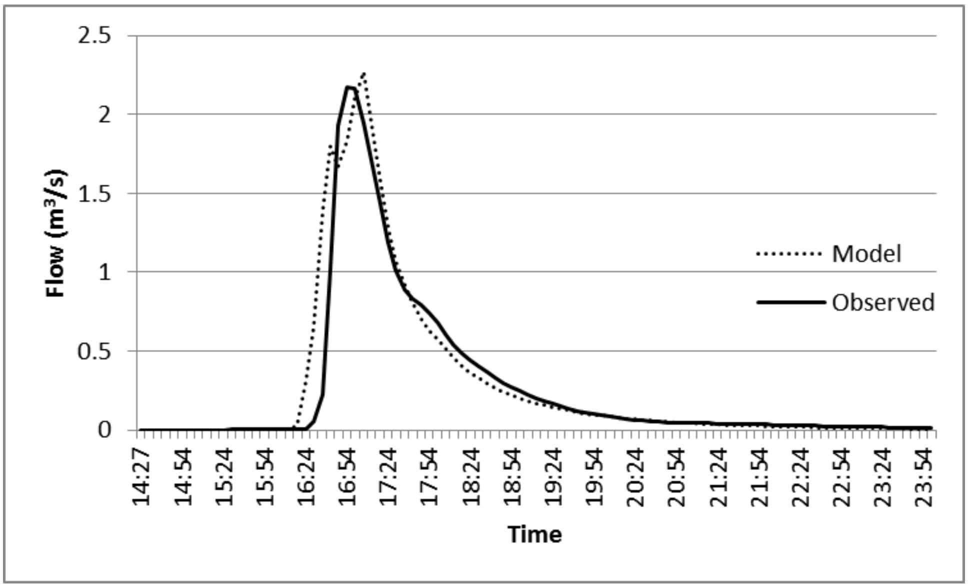


Figure 3-4 - Calibration for Event 12: 28 May, 2004 (Summer, Short)

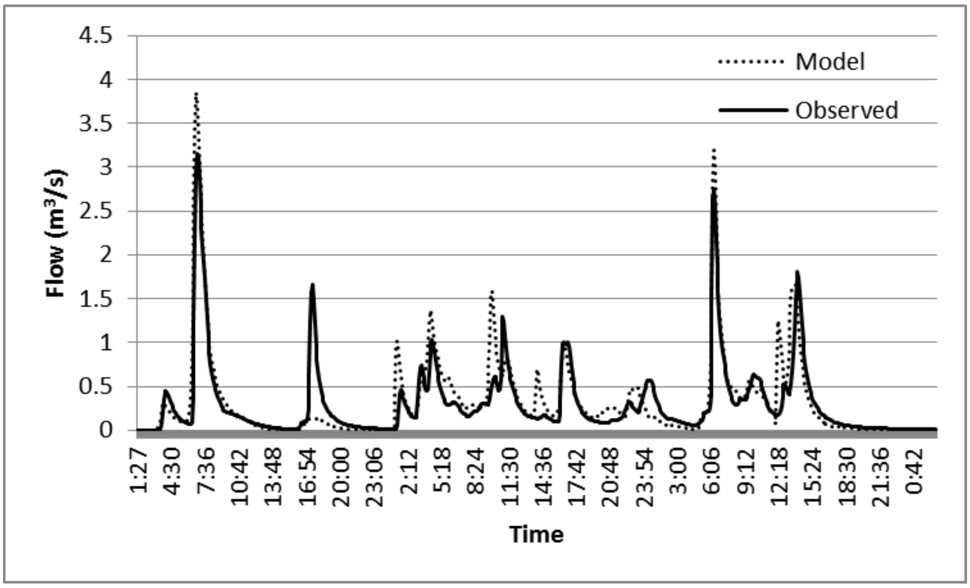


Figure 3-5 - Calibration for Event 16: 4 November, 2004 (Summer, Long)

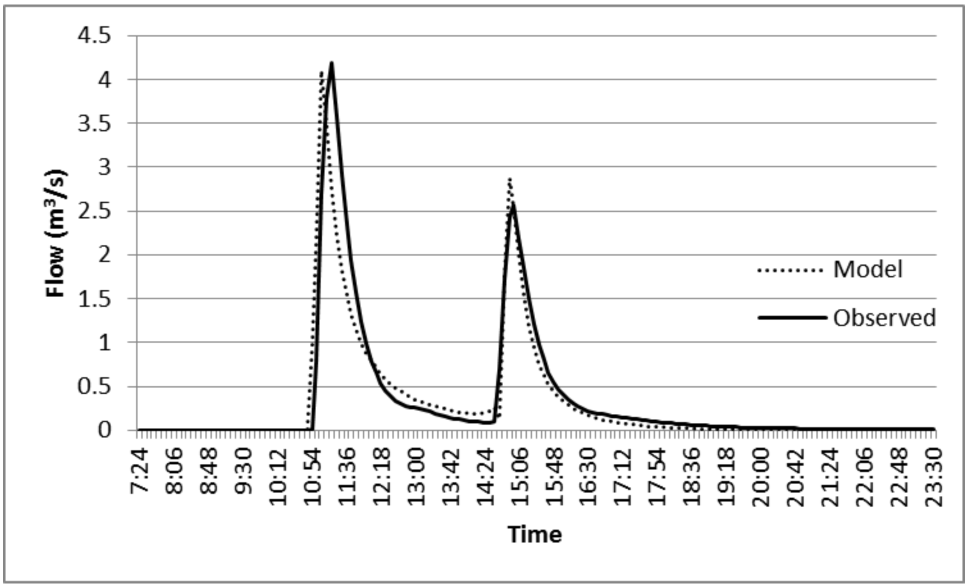


Figure 3-6 - Calibration for Event 19: 3 January, 2005 (Summer, Short)

3.5 Sources of Error

There are several sources of error that are suggested to influence the quality of the model calibration. These include the nature of rainfall data, parameter variation and gauge error. These are discussed further in the following sections.

3.5.1 Rainfall data

The collection and application of rainfall data is a possible source of error in this study. Some authors acknowledge that it is 'virtually impossible to obtain a precise and accurate measure of rainfall' (Ladson, 2008 citing Sumner 1988) and the measurement of rainfall has a significant impact on runoff.

Some rainfall measurement errors can occur in rainfall measurement process. For example, sources of error in rainfall measurement in the field can be attributed to losses due to wind deformation above the gauge orifice, as well as gauge wetting and splashing. Pluviograph rainfall is known to under-record low rainfall intensities (< 0.25 mm/h) (Ladson, 2008).

Other and potentially greater sources of error in the rainfall data may be in the attribution of rainfall to various points in the catchment on the basis of elevation and nearest gauge using data from Parafield (023013) and Little Para (A5040528) (Section 3.5). This method was adopted in an attempt to capture the spatial variation of rainfall which may occur across the catchment due to elevation. This was considered to be the best possible solution given the availability of short timestep rain gauge data.

For example, when only the data for Parafield Airport (023013) are used uniformly across the catchment, there is a reduced quantity of runoff produced by the model. To illustrate, using Event 20 as a case study, the predicted flow is reduced from 143800 m^3 to 136500 m^3 when the Little Para (A5040528) gauge is excluded. This is because the gauge at Little Para typically records higher rainfall than the gauge at Parafield for most events. This indicates the importance of attributing rainfall data correctly across the catchment when using multiple gauges. In this study, rainfall data was attributed based on elevation of the sub-catchment (Section 2.4.5), and there were only two gauges with appropriate quality data to select from. In other studies, rainfall data may be collected within or close to the catchment and it should be closely examined to attribute rainfall to catchments in the most appropriate manner. A further degree of accuracy may be possible through the processing and application of radar rainfall data which was not available from the Bureau of Meteorology for the period of this study.

3.5.2 Model Parameter Variation

There may be some variation in the model parameters throughout the catchment that was not captured in this study. For example, there may be variations in the

- percentage of connected impervious area (due to ongoing greenfield and infill development)
- Manning's 'n' value of the catchment pervious and impervious surfaces
- Manning's 'n' value of the conduits in the stormwater network
- Soil properties (infiltration, available storage depth)

These variations were not explored because with only a single flow gauge at the catchment outlet (point of harvest) there was little basis on which to examine the effect of parameter variation across the catchment. A distribution of flow gauges throughout the catchment would allow for different portions of the catchment to be calibrated on a more accurately based on observed flows, however with a single gauge it was considered more appropriate to uniformly adopt calibrated data within the ranges recommended for SWMM modelling from reference material (such as Rossman, 2010) which may be adjusted to suit the single site of observed data.

Wagener *et al.* (2004) also indicate that 'hydrological models are not typically capable of fitting all system response modes with a single parameter set due to the presence of structural errors'. Examples of such structural errors include parameters or variation in parameters that are not captured by a model. Variation of soil parameters over seasons, for example, are one such parameter not accounted for in SWMM which may inhibit calibration.

Changes in the percentage of connected impervious area may also have impacted on the study. For example, catchment impervious data was adopted from the mini-catchments data from Cardno-Willing NSW (2008). According to the accompanying report for this data, the imperviousness was attributed to the mini-catchment cells based on representative analysis of key areas within the broader catchment. Minor variations in this data across the catchment may have influenced the timing and quantity of observed runoff flows compared to those predicted using the PCSWMM model.

It should also be noted that over time, changes in the catchment landuse may have an impact on the calibration of this model. For example, in the 30 year plan for greater Adelaide (SA DPLG, 2010), there are 15 040 new dwellings planned for the Greater Adelaide region, with an increasing proportion of 50 to 70% of these new dwellings planned to be infill development. While large scale new development on the rural, pervious fringe of the catchment is not identified in the plan, it should be noted that infill of rural living and residential portions of the catchment may lead to an increasing portion of impervious area, and hence runoff, in the catchment.

3.5.3 Errors in Observed Flow Data

It is possible that successful calibration of some events was affected by errors in observed flow data. The source of this error is unknown, but it would appear that there is a significant discrepancy between the simulated and observed data for Event 21 (Figure 3-7, 6 December 2004), which was not found to occur to such an extent for the other storm events. Note that the peak flow in Figure 3-7 appears to be over-predicted and time-shifted from the observed flow.

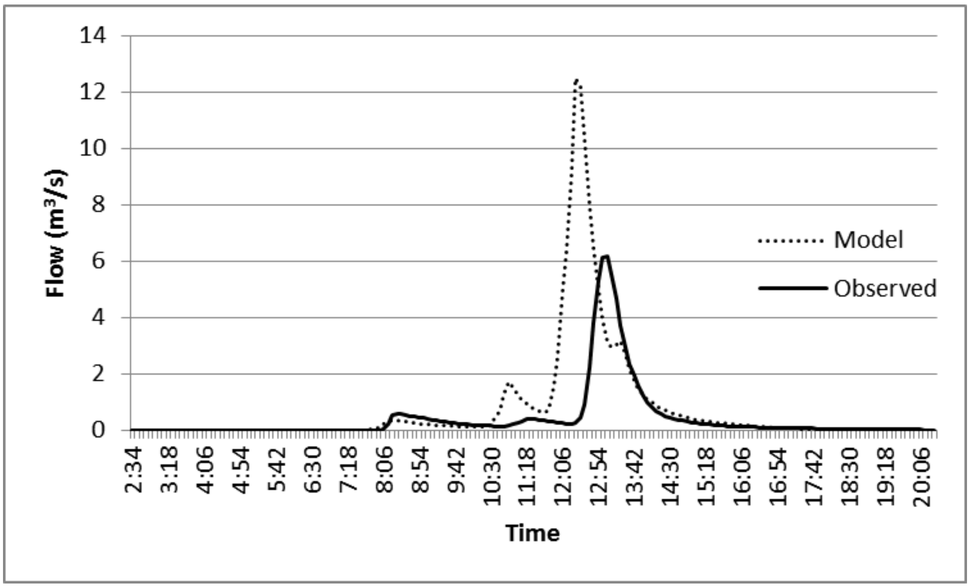


Figure 3-7 - Calibration for Event 21: 6 December 2004

The reason for the apparent time shift is unknown, but may be due to discrepancies between gauged rainfall and the character of rainfall over the catchment. However, a significant source of error in Figure 3-7 is the over prediction of flow. The model consistently over-predicted the peak flow of high flow events (Such as Event 20, 21 and 22). This may be due to an inappropriate relationship between flow depth and flow rate for the rating table used to determine flow rate at the Parafield drain gauge during high flow events. This problem may be attributable to the overflow of the Parafield drain during high events, where the depth of flow is not adequately representing the volume of flow based on channel geometry. It is suggested that the depth/flow rate relationship be further investigated to confirm the validity of this relationship for high flows (those in excess of approximately 6 m^3/s).

3.6 Application of the Model to Recent Storm Events

The flow data for this project was collected between 2001 and 2006. The possibility of landuse change occurring since this period was examined through a comparison of catchment orthophoto images from September 2002 and February 2010 (see Appendix H). The comparison indicated some landuse change had occurred in this time. A majority of landuse change had occurred in the Eastern (Cobbler Creek) area of the Parafield and Cobbler Creek catchment areas. This was mainly attributable to continuing residential development proceeding in the vicinity of Golden Grove. It should be noted that this catchment area only contributes flow to the Parafield drain via pumped flow to the Cobbler Creek dam at a fixed flow rate. There has been limited development within the

Parafield catchment, most of which includes some industrial development in the south west and residential development in the north east of (See Figure H2).

During mid to late 2011 the flow monitoring gauge at the harvesting location was reinstated. Recent flow data from this gauge (A5041049) was made available together with the Parafield Airport rain gauge (023013) and the Little Para rain gauge managed by SA Water (A5040528). Using this data is was possible to undertake a comparison of the calibrated model with more current data from the catchment. Figures 3-8 to 3-10 show the fit of the model to three more recent events.

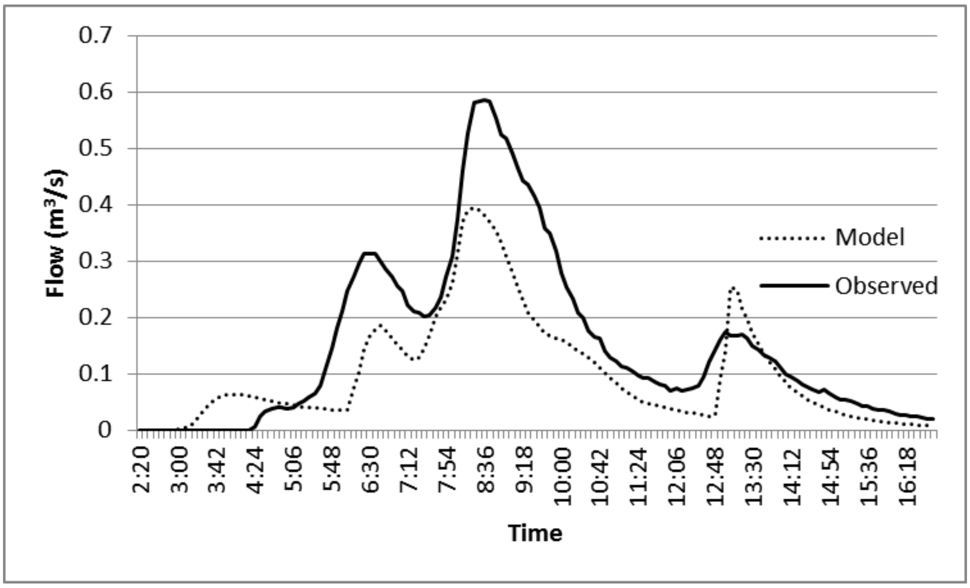


Figure 3-8 – Fit of the model to the event of 25 November 2011 (Nash Sutcliffe r^2 = 0.64)

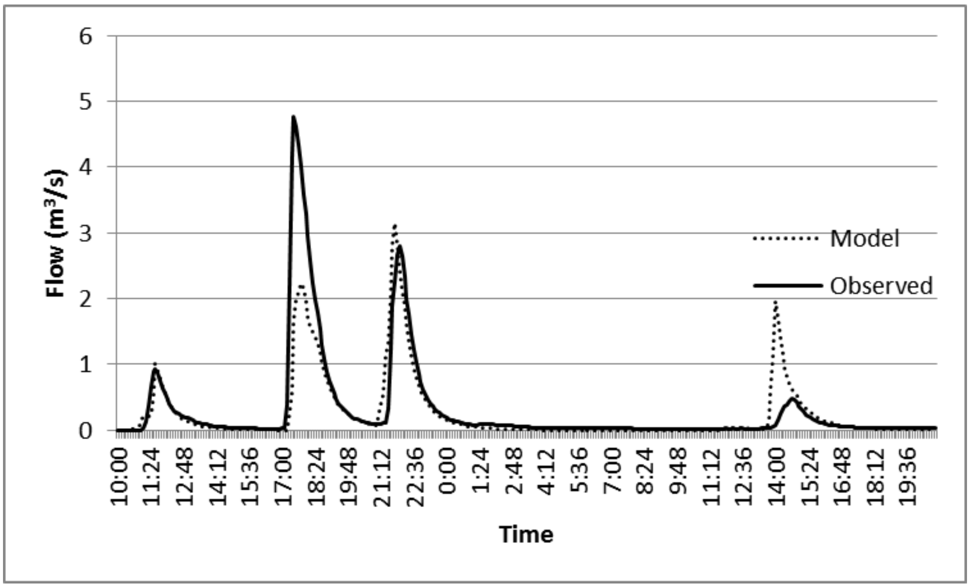


Figure 3-9 - Fit of the model to the event of 17 December 2011 (Nash Sutcliffe r^2 = 0.68)

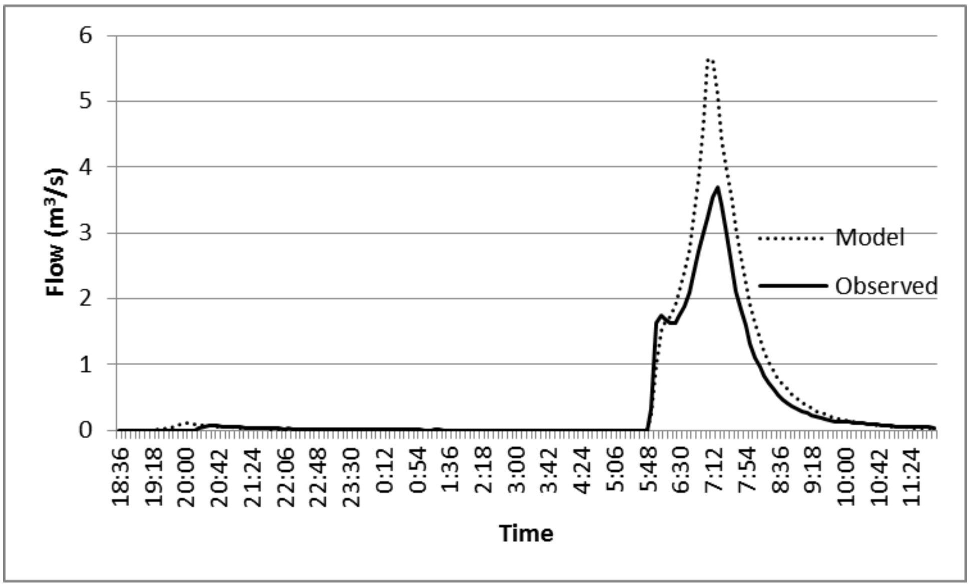


Figure 3-10 - Fit of the model to the event of 4 February 2012 (Nash Sutcliffe r^2 = 0.78)

The figures indicate that the model still predicts the flow from the catchment relatively well. Fitting of the model during this period was more difficult because of the relatively short period of monitoring data as well as some missing rainfall records from the Parafield gauge in both November and December 2011. Events were also selected to be isolated from Cobbler Creek pump flows for which the start-up and shut-off time were not available. Importantly for this study, based on the events above, the model appears to produce a reliable prediction of when runoff flow is expected (travel time and duration) to be present in the Parafield drain following storm events.

3.7 Further Improvements to the Parafield Model

There are several suggested measures that may lead to improvements in the calibration of the Parafield harvesting scheme model.

- The first, and perhaps most important improvement would be the installation of a gauging station at the upstream inlet to the Cobbler Creek storage basin which can measure inflows during storm events. The presence of other gauging stations to calibrate smaller sub catchment areas would further improve the accuracy of the model.
- Evaluation of total storage volume and event discharge relationships for storages which are present throughout the catchment
- Evaluation of the relationship between depth and flow records, particularly for high flows such as those in excess of 6 m^3/s.
- 'Ground truthing' (verification) of the location and invert of pit and pipe data as recorded on in GIS maps. Although this data was available for most stormwater pits and junctions in City of Salisbury, some data was assumed. Furthermore, data on pipe and pit inverts was not available for any pits or junctions located within the City of Tea Tree Gully. There was also little information available on the nature of natural channels within the Parafield or Cobbler Creek catchments such as width, depth and slope.
- Although there are records kept indicating whether the Cobbler Creek pump is switched on or off, this data was not available to the project due to problems with the City of Salisbury

SCADA system. Resolution of these problems and the availability of pump station activity would be of benefit to flow modelling and risk management in general.

- Use of radar rainfall data may improve the variation in rainfall data across the catchment. This was not available from the Bureau of Meteorology for the purposes of this project. It should be noted that the use of radar rainfall data should only be undertaken where the available historic records are of sufficient quality (by comparison to pluviograph records), of a suitable timestep and subjected to appropriate filtering using the methods such as those of Jordan and Hill (2006) to account for atmospheric influences occurring between the study site and the Adelaide (Buckland Park) weather radar.
- Use of a longer period of observed flow and rainfall data from will allow a comprehensive update of the model in future years where infill and further greenfield development may lead to higher flows from the Parafield and Cobbler Creek catchments.

3.8 Summary

Based on a review of appropriate modelling tools, the PCSWMM model (a commercial variant of the EPA SWMM model) was selected as an appropriate tool to assess the transport characteristics of pollutant spills for the Parafield scheme based on the ability to model at a short timestep and for a well-developed flow routing capability. A large amount of spatial information was used to develop a model of the Parafield and Cobbler Creek catchment. A field study was also undertaken to measure the time of travel of a flow from the furthest reach of the catchment (Cobbler Creek pump station) to the flow gauge at the end of the catchment. The model was then calibrated using observed flow data (for surface runoff properties) and the measured catchment time of travel to calibrate conduit roughness). The main sources of error in the model included: the inability to calibrate predicted runoff flows in the Cobbler Creek catchment because it was ungauged; the limited ability to calibrate flows in the Parafield catchment because only a single flow measurement weir was available to provide observed flow data, and; rainfall data may not accurately represent rainfall in the catchment because only two rain gauges were available which were both near to but outside the catchment boundaries.

4 Fate and Transport of Hazardous Spills – Dry Weather

The travel time for flows pumped from the Cobbler Creek pump station to the Parafield drain at a rate of 50 L/s is approximately 90 minutes, based on field observation (Section 3.3). This information was used during calibration to calibrate the travel time of the stormwater network, which was largely dependent on pipe roughness because other factors, such as pipe length and slope remained constant (Appendix A). Assuming properties such as pipe roughness remain constant throughout the catchment, this allows an investigation into the 'time of travel' for spills to travel from various sections of the catchment considered to be high risk locations for spill events, sewer overflows or other forms of flow contamination. In this section of the report, risk locations are identified where spills or contamination were found to be most likely to occur (Section 4.1), followed by an investigation into the time of travel for spills from these locations in dry weather (Section 4.3). Throughout this chapter, it should be noted that unless otherwise specified, travel times and pollutant transport characteristics refer to spills simulated in the PCSWMM model of the catchment (described in Section 3).

4.1 Selection of Risk Locations

The locations considered to present a high risk for spills or flow contamination were adopted for this study based on the Parafield and Cobbler Creek catchment hazard analysis reported by Page *et al.* (2011). The spill analysis in this report investigates locations categorised as 'Very high risk' by Page *et al.* (2011). Categories of contaminant at the very high risk locations were pathogens, inorganic chemicals, salinity and sodicity, nutrients, organic chemicals and turbidity. In some circumstances, medium risk locations were also considered.

The risk locations considered in this study are presented on a map of the catchment in Figure 4-1 with further detail on each location provided in Table 4-1. The very high risk contaminants at each location are summarised in Table 4-2. The following sections provide further detail on the selection criteria for each risk location with respect to contaminant category.

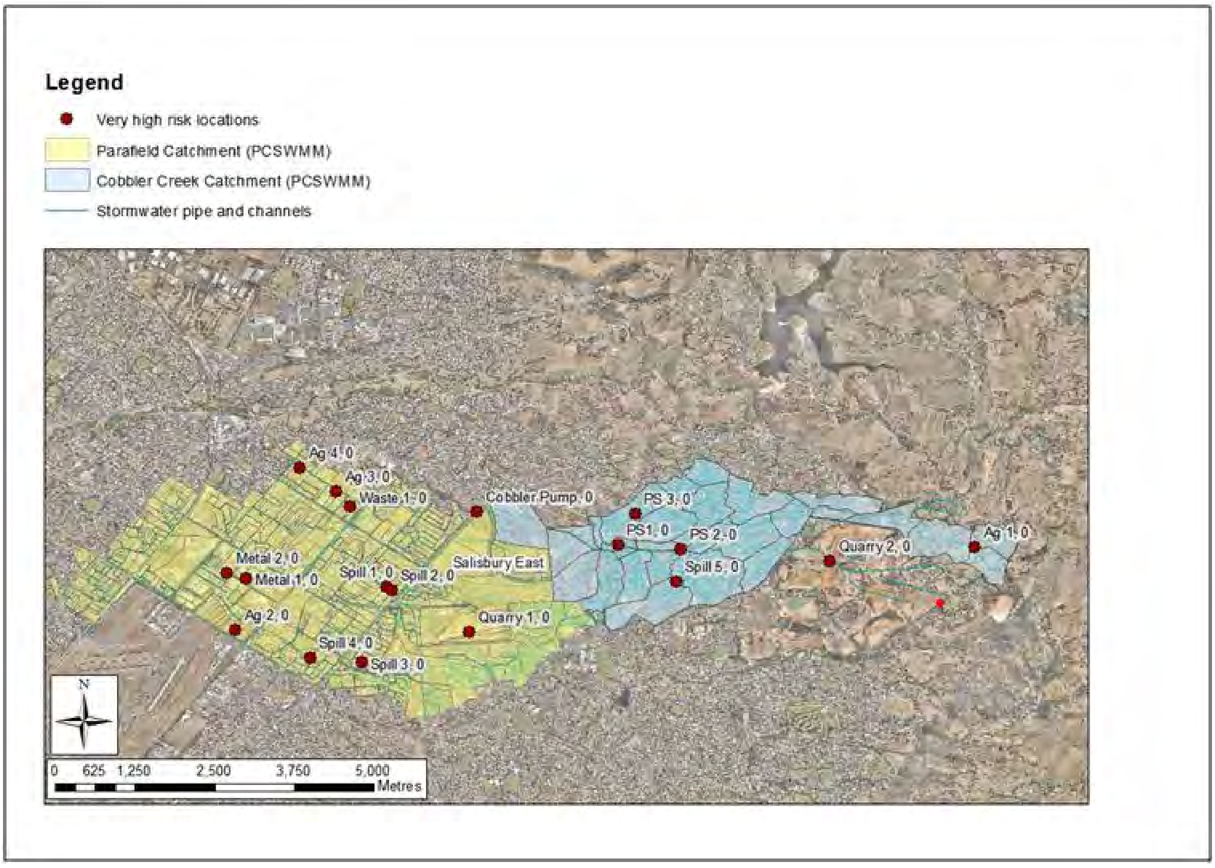


Figure 4-1 – Location of sites considered 'Very high risk' in the Parafield catchment (adapted from Page *et al.*, 2011)

Table 4-1 – Locations considered to be at high risk for spills and stormflow contamination (adapted from Page *et al.*, 2011)

#	Name	Description / Location
1	Spill 1	Intersection of Opal Court and Marquisite Drive, Salisbury East
2	Spill 2	Intersection of Amber Court and Marquisite Drive, Salisbury East
3	Spill 3	Intersection of Brabham Crescent and Laver Avenue, Salisbury Heights
4	Spill 4	Intersection of Keller Road and Andrew Avenue, Salisbury East
5	Spill 5	Valhalla Drive, Golden Grove
6	PS 1	Middleton Crescent, Golden Grove
7	PS 2	St Buryan Crescent, Golden Grove
8	PS 3	Bushmills Street, Greenwith
9	Ag 1	Yatala Vale
10	Ag 2	Salisbury South
11	Ag 3	Salisbury Plain, Stanbel Road 1
12	Ag 4	Salisbury Plain, Stanbel Road 2
13	Metal 1	Metal and electronics manufacturing - Climate Systems, Nylex Avenue, Salisbury South
14	Metal 2	Metal wheel manufacture and distribution (Mullins Wheels, Cheviot Road, Salisbury South)
15	Quarry 1	Ridge Road, Salisbury East
16	Quarry 2	Ross Road, Golden Grove
17	Waste 1	Industrial waste disposal, Arcoona Road, Salisbury Plain
18	Cobbler	Cobbler Creek pump station (pump into Parafield catchment)

Table 4-2 – Nature of hazards identified at the high risk locations from Table 4-1

#	Name	Risks					
		Pathogens	Inorganic	Organic	Salinity	Nutrients	Turbidity
1	Spill 1	✓					
2	Spill 2	✓					
3	Spill 3	✓					
4	Spill 4	✓					
5	Spill 5	✓					
6	PS 1	✓					
7	PS 2	✓					
8	PS 3	✓					
9	Ag 1	✓		✓		✓	
10	Ag 2	✓				✓	
11	Ag 3			✓		✓	
12	Ag 4			✓		✓	
13	Metal 1		✓				
14	Metal 2		✓				
15	Quarry 1		✓		✓		✓
16	Quarry 2		✓		✓		✓
17	Waste 1			✓			
18	Cobbler						✓

4.1.1 Pathogens

There are three main sources of pathogens considered to produce a very high risk in the Parafield and Cobbler Creek catchments. These risks are considered to be present in areas with one or more of the following characteristics:

- Spills – a high frequency of reported sewer overflow occurrence
- Pump stations – presence of a sewer pump station located adjacent or near a natural stream
- Agriculture – presence of livestock cultivation

Areas of high exposure to sewer overflows, locations Spill 1 to Spill 5, were identified by Page *et al.* (2011) using records of known sewer overflow occurrence from United Water operations records between May 2003 and February 2011. High risk areas were those where more than 5 events were reported in the immediate area. Generally speaking, sewer overflows have occurred throughout the catchment, but are particularly prevalent in residential areas at the bottom of hills in the east of the Parafield catchment and the south west of the Cobbler Creek catchment. Overflows are more sporadically reported in the low lying areas of the Parafield catchment.

Pump station locations have been included in the analysis as a conservative measure. There were few overflows reported to have occurred near pump stations. For the three pump station locations identified on or near creeks, PS1 to PS3, a single overflow event had been reported at both PS 1 and PS 3.

Risk areas containing livestock, Ag 1 and Ag 2, were also included as a very high pathogen risk in this analysis. This is considered a conservative assumption; livestock cultivation has been a constant presence in the catchment and has therefore been contributing runoff during high rainfall events since the harvesting scheme started operating. A comparison of mean urban stormwater runoff quality data for *E. coli*, thermotolerant coliforms, Enterococci and faecal streptococci presented by NRMMC, EPHC and NHMRC (2009) and limited stormwater quality data presented for the Parafield catchment (Swierc et al, 2005; Page *et al.*, 2010) has shown pathogens have been detected in raw stormwater at Parafield. This is despite the continuing presence of livestock cultivation (and domestic pets) in the catchment. Monitoring data for the Parafield catchment is limited however, and little information was available on the nature (flow volume, duration) of stormwater events sampled. It is therefore recommended that field monitoring for pathogens continues to be undertaken at or near the entry to the instream basin during storm events, especially where runoff from pervious areas is likely to occur, as it is possible that monitoring has not captured runoff from impervious areas where livestock are present. Operators should also be aware of any expansion in livestock cultivation within the catchment.

Spills from locations Spill 1 to Spill 4 are all located near stormwater pits which may be conservatively assumed to lead directly to underground stormwater pipes. Because of this, there are limited opportunities to intercept or capture flows prior to reaching the instream basin when a spill is identified. As sewer overflows often occur during storm events, some dilution may be assumed to occur during transport, but the most obvious means of managing overflows from locations Spill1 to Spill 4 is to divert stormwater flows past the instream basin when a spill is identified (i.e. stop harvesting stormwater from the stormwater event). This is, however, contingent on having sufficient time to both learn of and react to an overflow event.

The high risk locations at Spill 5 and PS 1 to PS 3 each drain to the Cobbler Creek flood detention basin. As such, risk management options are more numerous. Firstly, the spill concentration may be expected to be diluted by stormwater as it approaches the Cobbler Creek flood detention basin. Advection and dispersion processes will also occur in the Cobbler Creek storage before the flow reaches the outlet. If there is adequate notification, there is an opportunity to disable the pump at Cobbler Creek, preventing the transfer of contaminated flows from Cobbler Creek into the Parafield catchment whilst continuing to harvest from the Parafield catchment. Failing this, all flow can be arranged to bypass the Parafield airport diversion weir if the isolation of Cobbler Creek is not conducted quickly enough. In the event of failure to divert flows from ultimately entering the instream basin, the surface water detention time, wetland treatment processes and aquifer storage, transfer and recovery processes may also contribute to reduce the risk to human health caused by sewer overflows and other forms of pathogen contamination.

4.1.2 Inorganic chemicals

There were several locations identified by Page *et al.* (2011) to present a high risk of inorganic chemical spills. These included Metal 1, Metal 2, Quarry 1 and Quarry 2 (Table 4-1)

These locations are considered to present a low risk to the stormwater scheme because it is assumed that each listed activity operates under a South Australian EPA licence (where required) and has spill management procedures in place. These procedures are considered beyond the scope of this report, however, and for the purposes of this analysis, spills are assumed to enter the stormwater network at the nearest stormwater pit. In the case of quarries, spills are assumed to enter the catchment at the stormwater pit nearest to the detention basin capturing stormwater runoff from the quarry.

The risk assessment by Page et al (2011) also indicated a high risk location due to rubber tyre manufacturing. This facility was the Bridgestone tyre manufacturing facility which was decommissioned in April 2011 and was thus excluded from this analysis. Future landuse at this location is uncertain however, and the possibility of future manufacturing activity or residential development should be considered in future studies. Such activities may contribute to water quality hazards in the catchments, particularly during the construction phase.

4.1.3 Organic chemicals

There are four locations indicated to present a high risk source for organic chemicals. This includes a horticulture facility near location Ag 1 and several smaller horticulture sites located along Stanbel Road, Salisbury Plain, represented by locations Ag 3 and Ag 4 (Table 4-1). A further risk was considered to be the waste management facility in Salisbury Plain, location Waste 1. It is known to accept bottle and can waste for recycling, but may also contain waste oil and grease products and has been included as a precautionary measure.

4.1.4 Salinity and Sodicity

There are two high risk locations for salinity and sodicity identified in the risk assessment by Page *et al.* (2011). These include locations Quarry 1 and Quarry 2 (Table 4-1). Runoff from the quarry sites should be minimal due to the presence of detention basins for managing stormwater at these sites. City of Tea Tree Gully (pers. comm.) indicated that runoff from Quarry 2 is highly unlikely to occur at any event. However, increased salinity has been observed from wet weather flow monitoring results collected by CSIRO staff downstream of these locations (Page *et al.*, 2011) and was attributed to increased salinity of baseflow in cuttings becoming present in surface water.

4.1.5 Nutrients

Locations considered to be very high risk for nutrient contamination consist of the agricultural landuse at Yatala, Salisbury Plain and Salisbury South represented by locations Ag 1 to Ag 4. The contribution of these locations to nutrients in stormwater runoff should be examined by field monitoring. These locations are largely impervious and do not contribute greatly to runoff in small storm events. Furthermore, some locations such as Ag 1 collect runoff into on-site dams which are visible on aerial maps but not registered in the digital elevation map used for this report.

4.1.6 Turbidity

Locations at high risk of turbidity included the two quarry locations (Quarry 1 and Quarry 2). Turbidity has also been found to be higher in flows pumped into the Parafield catchment from Cobbler Creek (Site 18, Cobbler). The quarry locations should not contribute largely to turbidity due to implemented management practices, as discussed in Section 2.3. However, monitoring has indicated increased salinity downstream of quarry locations (Page *et al.*, 2011) and the travel time of these locations was therefore considered.

4.2 Water Quality Modelling Analysis

There were a number of assumptions made in the modelling of pollutant spill transport. These assumptions are outlined in the following sections. These assumptions relate both to the nature of spills, and include some modelling details relevant to the representation of spills in the PCSWMM (and EPA SWMM) model.

4.2.1 Pollutant Spill Type

A pollutant spill can be simulated in one of two ways in PCSWMM. The first is by assuming an inflow of water representing the spill to a stormwater pit, specifying that the water has a pollutant concentration (mg/L). This is termed the 'CONCENTRATION' pollutant assumption. The second is by assuming the addition of a dry pollutant mass (mg) which is added to a stormwater pit and is carried downstream by stormflow or baseflow running through this point in the stormwater drainage network. This is termed the 'MASS' pollutant assumption.

To determine if one of the methods produced a more conservative result, an analysis was carried out using the PCSWMM model. The analysis involved the simulation of a four day period with no rainfall and a constant baseflow at Pit-23238 (Site 19, Cobbler) equal to 5 L/s, 50 L/s or 100 L/s. A 20 Ton pollutant (MASS) or 20 kL fluid pollutant (CONCENTRATION) was also added to the model immediately downstream of the Cobbler pump at Pit-23137. The results were examined to determine travel time to the flow gauge at the end of the catchment (A5041049), the time for the peak concentration to occur and the duration of the pollutant plume in the outflow. Table 4-3 shows the results of the analysis. In these results, the pollutograph was bounded by the appearance of pollutant in excess of 1 mg/L at the catchment outlet.

Table 4-3 – Pollutant characteristics of MASS and CONCENTRATION based pollutant loads at Pit-23137 with continuous baseflow

	CONCENTRATION			MASS		
Baseflow (L/s)	Travel time (mins)	Time to peak concentration (mins)	Duration (hrs)	Travel time (mins)	Time to peak concentration (mins)	Duration (hrs)
5	78	204	8.89	97	228	9.05
50	44	102	3.53	48	108	3.52
100	36	84	2.68	36	84	2.7

The results indicate that the CONCENTRATION input method provides a slightly more conservative estimate of travel time (i.e. the flow travels faster to the catchment outlet), especially when there is little flow to carry the pollutant (i.e. in conditions of low flow). This is attributed to the extra momentum provided by the injection of additional fluids in the CONCENTRATION case compared to low flows in the MASS input case. Based on these findings, the travel time analysis for pollutant spills in high risk areas was undertaken by assuming a pollutant CONCENTRATION.

4.2.2 Continuity Routing Error Assessment

Continuity analysis is undertaken at the end of each model run in the PCSWMM model. The analysis examined the conservation of mass for water and pollutants generated by the model. The analysis considered conservation of mass at the catchment surface (the runoff generation model) and conservation of mass in the routing of flows through the stormwater drainage network of pipes and channels (the transport component to the model). Minor errors may be anticipated to occur in every model run for both model components.

During simulated spill event trials it was found that very high continuity errors were occurring in the transport component of the model routing water quality through the stormwater pipes and drains of the Parafield catchment model. According to Rossman (2010), the most common cause of continuity errors of the transport component of the model occur when the modelling timestep is too long or when conduit (pipe and channel) lengths are too short. Similar issues with continuity errors were also reported in previous spill event simulation using EPA SWMM modelling, and in this case it was suggested that a background flow was required to move the simulated spill downstream in the EPA SWMM model (City of Novi, 2000).

To investigate the cause of routing errors in the Parafield model further, the timestep was varied between one second and six minutes in a series of model runs to investigate the flow routing error, quality routing error, travel time of flow and travel time of pollutant. Model runs were undertaken over four simulated days with a 12 minute, 20 kL simulated spill event occurring at midday (12:00) on day one of the simulation, situated at Pit-23138 (Site 18, Cobbler). The simulation had zero rainfall. To avoid problems associated with initial storage losses in the pit and pipe network, a constant baseflow was also simulated during the model period at Pit-23138. The results of this analysis are shown in Table 4-4. The results indicate that flows were routed similarly at longer and shorter timesteps and with a similar level of flow continuity error.

Table 4-4 - Effects of selected PCSWMM routing timestep on model output

Routing Timestep	Baseflow (L/s)	Variable	Travel time (mins)	Time to peak (mins)	Duration (hours)	Continuity Error (%)
1 Second	5	Flow	100	114	1.04	-0.413
		Quality	78	210	9	16.64
	10	Flow	55	66	0.55	-0.225
		Quality	43	108	3.77	3.375
	100	Flow	50	60	0.35	-0.188
		Quality	36	84	2.92	-0.335
60 Seconds	5	Flow	98	114	1.1	-0.5
		Quality	144	294	9.01	73.115
	10	Flow	55	66	0.53	-0.118
		Quality	97	174	4.13	28.579
	100	Flow	49	60	0.35	-0.096
		Quality	90	150	3.32	2.595
360 Seconds	5	Flow	98	120	1.21	-0.44
		Quality	447	636	10.28	81.114
	10	Flow	49	72	0.74	0.049
		Quality	398	510	5.61	26.723
	100	Flow	59	-	-	0.035
		Quality	392	486	4.77	1.439

The continuity errors for flow routing were all well below the 10% level considered acceptable by Rossman (2010). However, water quality routing results produced high continuity errors and unrealistically long travel times when a low baseflow rate was applied. Further investigation of the pollutant transport in the Parafield model indicated that at long routing timesteps the pollutant plume was travelling through a single conduit at each 6 minute modelling timestep, regardless of conduit length. The error in continuity of pollutant mass transport in the stormwater model was found to occur to a greater extent when low or zero baseflows were simulated. In these conditions, the pollutant mass tended to increase with each timestep as the pollutant plume was routed through the catchment. When the routing timestep of the model was reduced to one second, however, the continuity error of the pollutant plume was decreased, and the travel time and pollutograph were better suited to the hydrograph. In fact, the quality routing tended to occur faster than the flow routing, introducing a degree of conservatism in the travel time analysis. As a result of the findings of this analysis, the PCSWMM results reported throughout this document were based on model simulations with a one-second routing timestep. It should also be noted that the shortest travel times occurred with the highest baseflow rates, which were considered of key importance to this study.

4.2.3 Benchmarking of the Model

For comparison of the PCSWMM simulation data with model results, a theoretical approximation of the catchment travel time has been provided using the Kirpich equation. The Kirpich equation (Equation 4) was developed based on flow in rural stormwater basins in the United States with well-defined channels and steep slopes. It has been widely used in the United States for estimating the time of concentration of catchments and channel flows for design purposes (Texas department of Transportation, 2011), however Pilgrim and Cordery (1993) advise that this equation tends to provide low values in Australia.

$$t_c = 0.0078L^{0.77}S^{-0.385} \qquad \text{- Equation 4}$$

Where t_c represents the time of concentration, L represents the length of the longest overland flow path or channel (length of conduit and open channel for the purposes of approximation in this report) and S represents the average slope along length L. Chow (1988) advises that when estimating the time of concentration for flow in concrete channels the value of t_c should be multiplied by 0.2. It should be noted that the equation only accounts for length and slope and does not explicitly account for other parameters which can influence time of concentration such as changes in slope along the length of the flow path, roughness of the flow path and the rate of flow. Based on application of the Kirpich equation, a theoretical travel time from each high risk location to the catchment outlet is provided in Table 4-5 (Page 42) for comparison with the PCSWMM modelling results.

4.3 Fate and Transport of Hazardous Spills in Dry Weather

4.3.1 Dry Weather Flow - Method of Determination

In this report, the dry weather travel time refers to the time taken for flows to travel from the risk locations in Table 4-1 to either the Parafield drain or the detention basin at Cobbler Creek. The travel time was calculated using the following two simulated 'spill' events draining directly into stormwater pits at the risk locations:

- Spillage of 20, 000 L of liquid contaminant over a 12 minute period (100 '44 gallon drum')

- Spillage of 40,000 L of liquid contaminant over a 12 minute period (200 '44 gallon drums', or the volume of a small tanker)

The travel time was determined by running the PCSWMM model for a period of 4 days with zero rainfall over the modelling period. Pollutant spills were simulated to occur over 12 minutes with a peak at 12:00 (midday) on the first day of the simulation. The spill consisted of a liquid contaminant with a generic pollutant concentration of 1 000 000 mg/L, representing a pure liquid pollutant with a density of 1 kg/L. The characteristics of the assumed spills are shown by the spill hydrographs in Figure 4-2.

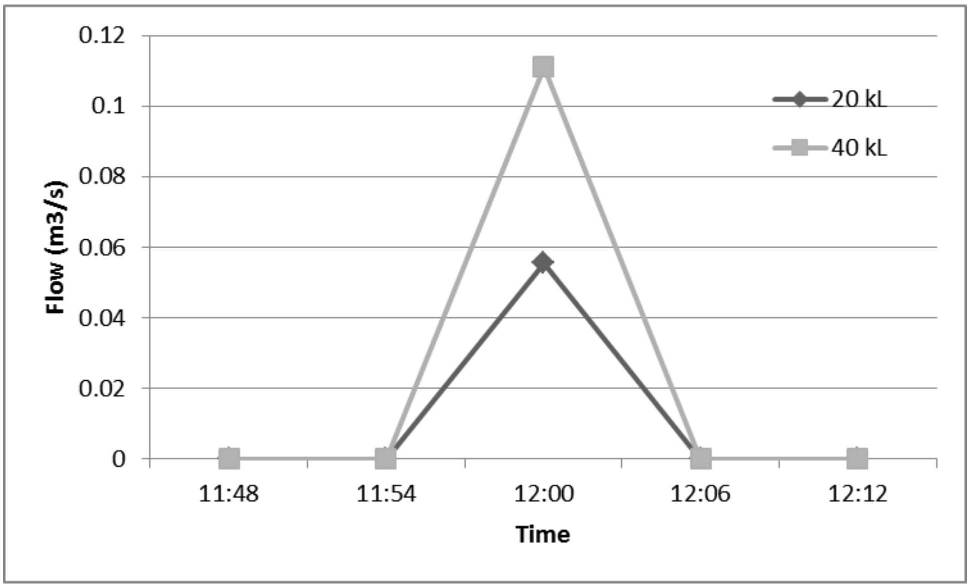


Figure 4-2 - Assumed hydrograph of the 20 kL and 40 kL pollutant spills

In this analysis, PCSWMM was run using kinematic wave routing. Kinematic wave routing solves the continuity equation (conservation of mass) and a simplified form of momentum conservation (Saint Venant) equations as outlined by Rossman (2010). Factors considered when determining the time of travel included the slope of pipes, Manning's 'n' (or roughness) coefficient, flow rate and depth of flow in pipes. Initial modelling results showed high errors in the routing of flow volume and water quality in the PCSWMM model due to instability in the flow routing with low flow volumes occurring over a short duration (see Section 4.2.2). These errors have been shown to occur in previous studies examining the travel time of pollutants under very low flow conditions (City of Novi, 2000). To overcome this issue, flows were simulated with a small baseflow of 5 L/s occurring at the point of contaminant spill, which produced a more acceptable routing error for both flow and pollutants. The 5 L/s baseflow was considered appropriate because it reduces the time of travel of pollutants and therefore provides a degree of conservatism in the resulting travel time estimation.

It was assumed that pollutant spills occurred at the nearest stormwater pit to the hazardous location. This was considered an appropriate assumption because it represented a scenario where the total volume of a spill will enter the stormwater network (rather than some portion being detained/retained on the catchment surface). It also assumes that the spill travels immediately to the point of harvest (rather than allowing for a time of travel across the catchment surface). This assumption was also considered appropriate because spills and sewer overflows may be considered likely to occur on or adjacent to roadways, which is also where the majority of stormwater collection pits are distributed in the catchment.

The travel time of the pollutant spill to the Parafield airport flow gauge (A5041049) was determined by the time difference between the commencement of the spill (11:54 am, day one) and the time that the flow of pollutant reached the catchment gauge location in the PCSWMM model.

4.3.2 Catchment Travel Time Results – Dry Weather

The time of travel for a spill at each risk location during dry weather was determined using PCSWMM with the results shown in Table 4-5. For the purposes of a conservative risk assessment of dry weather contamination events, it is recommended that the travel time for a 40,000 L spill is considered to represent the time to respond to dry weather spill events. Results are shown for a 20 kL and 40 kL contaminant spill at the location indicated, each occurring in the presence of a 5 L/s baseflow.

Table 4-5 – Travel time of spills during dry weather in the Parafield and Cobbler Creek catchments based on simulation in PCSWMM

Site	Pit-ID	Distance (m)[1]	Elevation start (m)	Elevation end (m)	Mean slope (%)	PCSWMM travel time (mins)[2]		Manually estimated travel time (mins)	
						20 kL (+ 5L/s)	40 kL (+ 5L/s)	Kirpich equation *EST*^{2} (mins)	Chow estimate (*EST*^{2} x 0.2)
1	Pit-25331	5100	51.63	9.81	0.01	78	67	89	18
2	Pit-25639	5180	54.23	9.81	0.01	78	67	88	18
3	Pit-20061	4090	53.86	9.81	0.01	60	54	67	13
4	Pit-25356	3390	27.29	9.81	0.01	55	54	77	15
5	J6219	3380*	168.79	107.40	0.02	49[CC]	44[CC]	48[CC]	10[CC]
6	J16055	1630*	134.11	107.40	0.02	24[CC]	18[CC]	28[CC]	6[CC]
7	J9321	2508*	145.06	107.40	0.02	36[CC]	31[CC]	41[CC]	8[CC]
8	J6995	2100*	175.00	107.40	0.03	24[CC]	18[CC]	26[CC]	5[CC]
9	J23710	6660*	248.97	107.40	0.02	155[CC]	127[CC]	76[CC]	15[CC]
10	Pit-18621	2060	18.80	9.81	0.00	30	30	56	11
11	Pit-18250	4280	38.12	9.81	0.01	55	48	84	17
12	Pit-18251	3795	35.34	9.81	0.01	48	42	76	15
13	Pit-24527	2480	21.34	9.81	0.00	30	30	63	13
14	Pit-19692	2160	19.76	9.81	0.00	30	27	57	11
15	Pit-20089	5865	67.29	9.81	0.01	79	72	92	18
16	J6479	4360*	182.59	107.40	0.02	76[CC]	72[CC]	59[CC]	12[CC]
17	Pit-16017	4395	38.89	9.81	0.01	60	50	86	17
18	Pit-23138	6800	80.68	9.81	0.01	84	73	101	20

[1] Approximate distance from spill location to the Parafield Airport flow gauge (A5041049), based on length of stormwater pipe. Measurements marked (*) indicate the distance to Cobbler Creek spillway, to which 6800 m must be added to account for distance from Cobbler Creek pump station to the instream basin
[2] Time for pollutant plume to travel from spill location to flow measurement weir A5041049
[CC] Values indicate travel time to Cobbler Creek detention basin only

The travel time of pollutant spills to the catchment outlet was longer when zero baseflow was assumed. For example, the travel time of a 20 kL spill at Site 18 was approximately 114 minutes with zero baseflow, compared to 84 minutes when a 5 L/s baseflow was assumed (See Appendix C). However, results in the absence of a baseflow were compromised by high levels of flow routing error. Without a baseflow, the volume of flow was magnified by between 80% and 264% from point of spill to the catchment outlet. Assuming a small baseflow, however, reduced this error to between 0.3% and 1.5% (for the 20 kL spill) or between 0.2 and 0.9 % (for the 40 kL spill). As such, the results in Table 4-5 are proposed as the most reliable estimate of pollutant spill travel time from high risk locations. The assumption of a baseflow also has the benefit of introducing a degree of conservatism. The associated flow routing error and other details these results, and the travel time of spills without the presence of a baseflow are presented in Appendix C.

The results indicate that the spill volume has an impact on time of travel in the catchment. For example, to travel from the Cobbler Creek Pump station (Site 18) to the end of the catchment was 84 minutes for a 20,000 L spill over 12 minutes duration. However, this travel time was reduced to 73 minutes when a 40 kL spill occurred. This attributed to a greater a greater kinetic energy for larger flows (such as a bigger spill volume) effectively forcing water through the stormwater network faster, compared to lower flows over the same duration. Increased depth also reduces the net effect of pipe and wall roughness on the flow of water through the pipe network, leading to increased flow velocities.

Overall, the modelling shows that during dry weather the travel time of spills in the catchment provides a short response time for any remedial actions. The travel time can be a matter of minutes at some locations close to the catchment outlet, and up to approximately 90 minutes at the furthest reaches of the Parafield catchment. In fact, key areas of dry weather spill risk, such as those in the industrial zone in the South West of the Parafield catchment, are closer to the Parafield wetland diversion weir and spills may reach the outlet in a little over half an hour. It should be noted, however, that this assumes that the spill volume is not retained by wetting and storage as it travels through the catchment.

Travel times are of varying length in the Cobbler Creek catchment. Like the Parafield catchment, the travel time of spills near the Cobbler Creek flood retention basin may be only a matter of minutes due to high slopes and close proximity to the basin. However, the travel time from the agricultural area to the east of Cobbler Creek catchment is approximately 155 minutes because slope and the natural channel roughness produce large flow duration. In most cases nearer the Cobbler Creek retention basin, the travel time of simulated pollutant flows were faster (per unit of distance) than those in the Parafield catchment. This is because the mean gradient of pipelines and channels to the catchment of the Cobbler Creek flood detention basin were typically between 1 to 2%, compared to pipelines in the Parafield catchment where the typical gradients range between 0.4 to 1%. A higher gradient leads to higher flow velocity and hence a more rapid catchment travel time. It should be noted however that the simulations of flow in the Cobbler Creek catchment are presented with less certainty than in Parafield because there were no flow data available to calibrate the routing of flows through the drainage corridors in Cobbler Creek catchment. In addition, flow routing is more difficult to predict in this catchment because the main drainage lines are dominated by natural channels such as Cobbler Creek and Slate Creek which may be more variable in roughness and slope than the subsurface pipes which dominate the Parafield catchment. Channel roughness has been conservatively assumed in Cobbler Creek based on the range representing natural channels presented by Rossman (2010) (Appendix A) however it is strongly recommended that for any future modelling works, flow should be monitored at the inlet to the Cobbler Creek detention basin.

4.4 Summary

Based on an existing catchment hazard identification procedure (Page *et al.*, 2011), 18 locations in the Parafield and Cobbler Creek catchment were identified which presented a highest risks for pollutant spills and overflows. Pollutants of concern included pathogens, inorganic chemicals, organic chemicals, salinity and sodicity, nutrients and turbidity.

Preliminary modelling was then undertaken to determine the most effective approach to simulating dry weather pollutant spills using PCSWMM. This process revealed that the most effective way to simulate pollutant spills was by assuming a liquid pollutant spill (CONCENTRATION) rather than assuming a dry pollutant mass (MASS). This process also revealed that high levels of water quality and flow routing errors were produced when a spill was simulated to occur without some constant baseflow to carry the pollutant downstream. Based on these findings, pollutant spills were assumed to occur as a liquid pollutant with a small baseflow, which produced more acceptable model routing errors and more conservative (shorter) predicted travel times.

Dry weather pollutant spill simulation indicated that the travel time was largely dependent on distance from the catchment outlet. In the Parafield catchment, the travel time of a pollutant spill was found to be from 30 to 84 minutes, indicating a limited time to respond to spills in dry weather. For Cobbler Creek catchment, the travel time of pollutants from the point of the spill to the Cobbler Creek retention basin was from 8 to 155 minutes. While spills in this region may still reach Cobbler Creek dam, there remains the possibility to disable the pump which diverts water from Cobbler Creek into the Parafield catchment in the event of a rainfall event following a pollutant spill. It should be noted however that flows in this catchment are determined with less certainty, but are likely to be conservative – that is, travel times are likely to be under-estimated.

5 Fate and Transport of Hazardous Spills - Wet Weather

The previous section investigated the travel time of pollutants in dry weather, which is an assumption suitable for a majority of days in the catchment (rain is typically only observed on 61 days per year by the gauge at Parafield Airport, 023013). However, pollutant spills and overflows in wet weather may be expected to travel at a faster rate, and travel times for pollutant spills in wet weather will thus provide a more conservative estimate of travel time throughout the catchment. Furthermore, the occurrence of sewer overflows are often associated with wet weather, and rainfall can also produce dangerous conditions in transport corridors and on sites storing chemicals and waste, necessitating the study of wet weather pollutant flows.

The characteristics of pollutant flows in wet weather may be expected to vary depending on the nature of the storm event, the spill location and the spill duration. To undertake a deliberately conservative analysis of wet weather pollutant transport, key assumptions were first analysed to determine the characteristics of flow which were subsequently assumed for wet weather spill modelling in PCSWMM. These findings are reported in Section 5.1. Following this, the methodology to determine wet weather travel times from high risk locations using PCSWMM is presented in Section 5.2 followed by the presentation of modelling results and discussion in Section 5.3.

It is acknowledged that diffuse pollutants are also a concern from urban developments during wet weather. However diffuse pollutants have not been considered in this report due to a lack of water quality data. Consideration of diffuse pollutants may be undertaken in future works by referring to the large areas of high and very high risk for parameters such as salinity and turbidity and exploring the build-up and wash-off characteristics of salt and turbidity from these areas. In the current study, the time of travel from these locations is examined in the form of a pollutant spill (not build-up and wash-off from the catchment surface).

5.1 Wet Weather Spill Event Attenuation Characteristics

The nature of pollutant transport in throughout the catchment can be influenced by a variety of factors including storm intensity (mm/hr), duration , spill location, spill duration and time of occurrence. Before undertaking analysis of pollutant travel times, it was considered important to determine circumstances which were likely to produce the worst case (i.e. lowest) travel time for conducting a risk assessment. The following sections describe the approach and results of an analysis to determine this worst case set of assumptions.

5.1.1 Approach to Determining Wet Weather Spill Event Attenuation Characteristics

To examine the impact of key storm event and pollutant spill characteristics on travel time, a series of analyses were undertaken to compare the effect of assumed variables on the timing and nature of the pollutant plume at the catchment outlet (the Parafield drain). These variables included:

- *Stormwater event type*: the 3 month ARI and 10 year ARI storms for Adelaide of the half hour and 12-hour duration were used for the assessment. Each of the four storms were simulated at midday on day 1 of a 5 day simulation period consisting of otherwise dry weather.
- *Spill location*: there were two locations simulated to receive each stormwater event type and spill type; the first is at the Cobbler Creek pump station (Pit-23138) and the second

approximately half way to the catchment outlet of this main stormwater line (J102). These locations are illustrated in Figure 5-1.

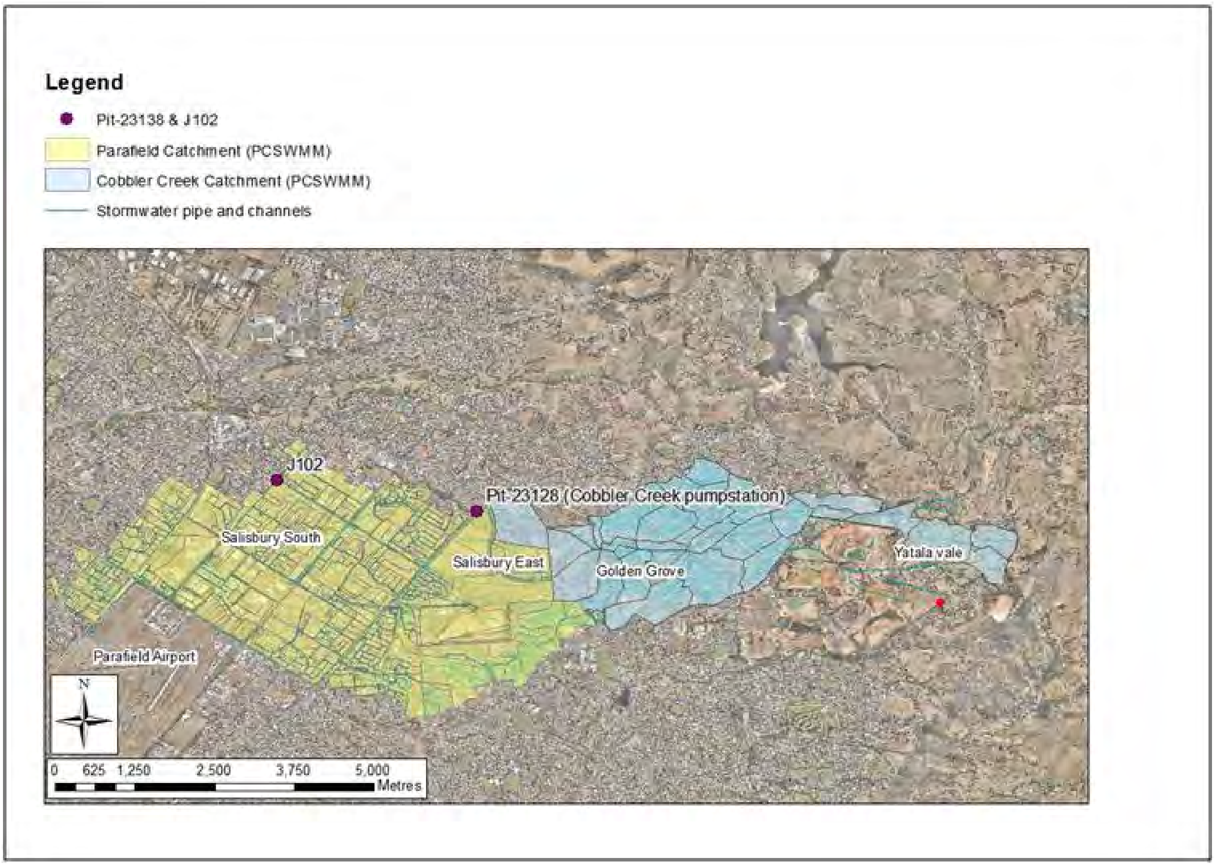


Figure 5-1 – Location of PCSWMM simulated spills at Pit-23138 and J102 in the study of spill attenuation characteristics

- *Spill duration*: there were three spill types simulated for each stormwater event type and spill location. Each spill consisted of 20 kL volume, over a 6 minute, 30 minute or 2 hour duration. The pollutant was assumed to consist of an aqueous generic contaminant with a concentration of 1 000 000 mg/L (i.e. the pollutant was assumed to be an injection of flow consisting of 1 kg/L of generic pollutant). Degradation and diffuse pollutant sources were not considered, and any change in concentration is attributed only to dilution with stormwater runoff flows.
- *Spill timing*: Each spill was assumed to occur at one of two time points, namely at the beginning of the half hour or 12 hour storm (at t = 0, or midday on day one) or at the midpoint of the half hour or 12 hour storm (at t = 12 minutes and six hours, or 12:12 and 18:00 on day one, respectively).

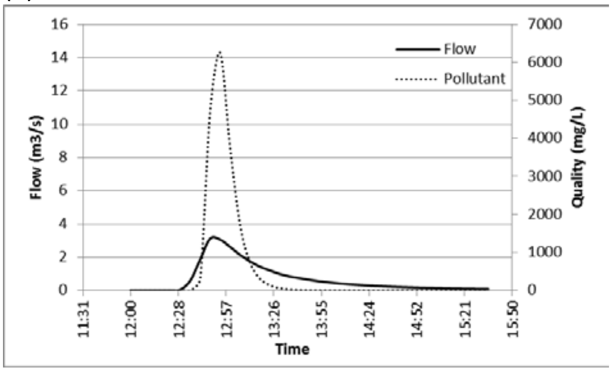
To compare the variable outlined above, the hydrograph and pollutograph characteristics were examined at the catchment outlet. The travel time of the pollutant was determined based on the time between the beginning of the spill and the appearance of the pollutant at the catchment outlet (as defined by the appearance of a 1 mg/L concentration of pollutant in flow at the catchment outlet). The total period of the pollutograph refers to the time over which the pollutant was in excess of 1 mg/L at the catchment outlet. The event mean concentration of the generic pollutant refers to the mean concentration of pollutant in flows over the entire stormwater event, where a stormwater event was defined as the period over which flow was in excess of 1 L/s at the catchment outlet.

The results of the modelling scenarios presented above are presented in full in Appendix D. A summary of key findings from a comparison of this data is provided in the following sections.

5.1.2 Effect of Stormwater Event Size

The effect of stormwater event size was assessed by comparing the results for simulated spills of the same duration and occurring at the same time with different stormwater events sizes (ARIs) in Appendix D. An example of the effect of stormwater effect size is illustrated using the 3 month, 30 minute storm and the 10 year, 30 minute storm in Figure 5-2. Increasing the size of the storm event (by increasing the ARI) produced a lower simulated maximum and mean concentration, a reduced time of travel and a reduced total simulated pollutograph period (less total time of contaminant flow observed at outlet).

(a)



(b)

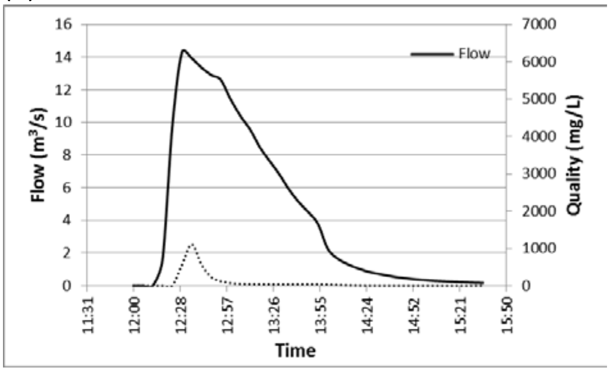


Figure 5-2 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a 5 minute duration spill occurring at the top of the catchment [Pit-23138] at the beginning of the (a) 3 month, 30 minute storm and the (b) 10 year, 30 minute storm

5.1.3 Effect of Spill Timing

The effect of spill timing is an important consideration. At the beginning of a storm event, pollutant spills prior to the storm may be washed immediately into the stormwater drainage system. In the case of sewer overflows, the occurrence of spills at the middle of stormwater events are a realistic consideration as sewer overflows tend to occur during periods of high flow.

The effect of spill timing was assessed by comparing the results in Appendix D for identical storm event ARIs, identical spill durations but different spill timings (start of storm or middle of storm). At the furthest point in the Cobbler Creek catchment (Pit-23128), the simulated maximum and mean pollutant concentration at the catchment outlet was typically higher when the spill was simulated at the middle of the storm event compared to the beginning. The time of travel tended to be quicker when the spill occurred at the middle of the event compared to the beginning when the 12 hour storm was considered (see Figure 5-3). There was little effect of spill timing in the 30 minute storm. The total time of the simulated pollutograph was generally longer at the outlet when the spill was simulated at the middle of stormwater events compared to the beginning.

(a) (b)

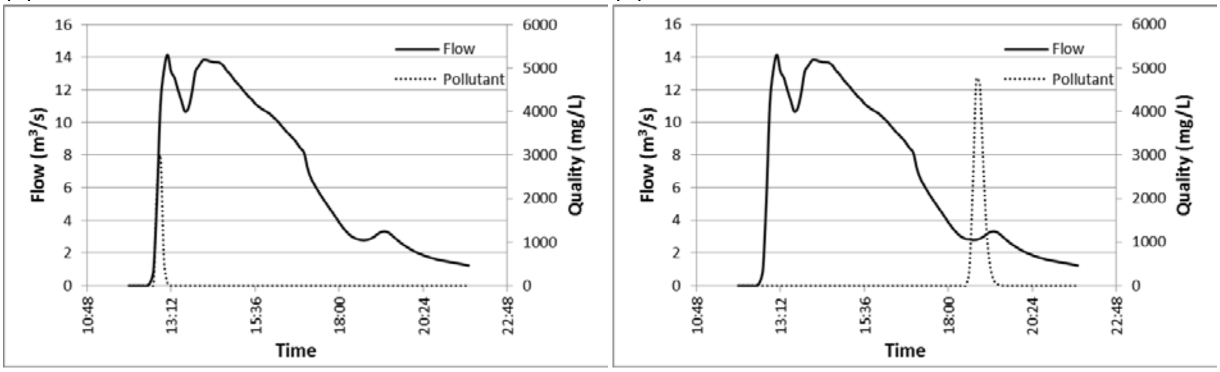


Figure 5-3 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a 5 minute duration spill occurring at the top of the catchment [Pit-23138] and at (a) the beginning of the 10 year, 12 hour storm and (b) the middle of the 10 year, 12 hour storm

This was somewhat different to the effect of spill timing on simulated outflows when the spill occurred at the middle of the catchment (J102). When a spill was simulated at this location at the beginning and middle of an event, the simulated maximum and mean pollutant concentration at the catchment outlet was found to be lower when the spill occurred during the middle of a spill compared to the beginning, as illustrated in Figure 5-4. This is because the reduced distance (and hence travel time) of the spill at the beginning of the event coincides with low flows, and hence a high concentration of pollutant at the catchment outlet (harvest point). At the middle of the event flows dilute the pollutant to a greater extent.

(a) (b)

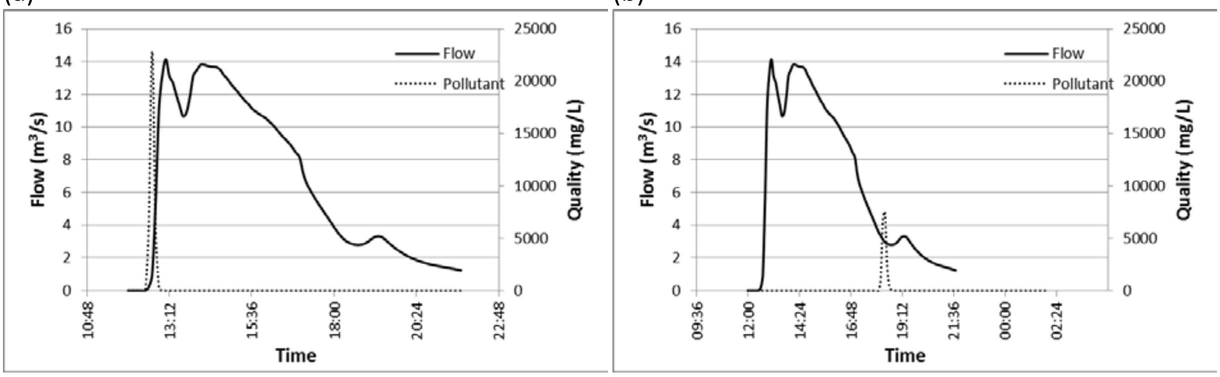


Figure 5-4 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a 5 minute duration spill occurring at the middle of the catchment [J102] and at (a) the beginning of the 10 year, 12 hour storm and (b) the middle of the 10 year, 12 hour storm

Similar to a spill at the top of the catchment, however, the time of pollutant travel was also generally quicker for a mid-event spill compared to one at the start of the event, which is also attributable to the presence of existing flows at the time of spill to transport the contaminant. In addition, the period over which the pollutograph was observed at the outlet of the model was generally longer when the spill occurred at the middle of the storm event.

5.1.4 Effect of Spill Duration

To examine the effect of spill duration, outlet pollutographs were compared for the 6 minute, 30 minute and 2 hour spill duration at each point of the catchment with identical storm ARI and duration (using the data in Appendix D). The results showed that longer simulated spill durations produced lower simulated event mean concentration and lower maximum concentrations. An example of this effect is shown in Figure 5-5.

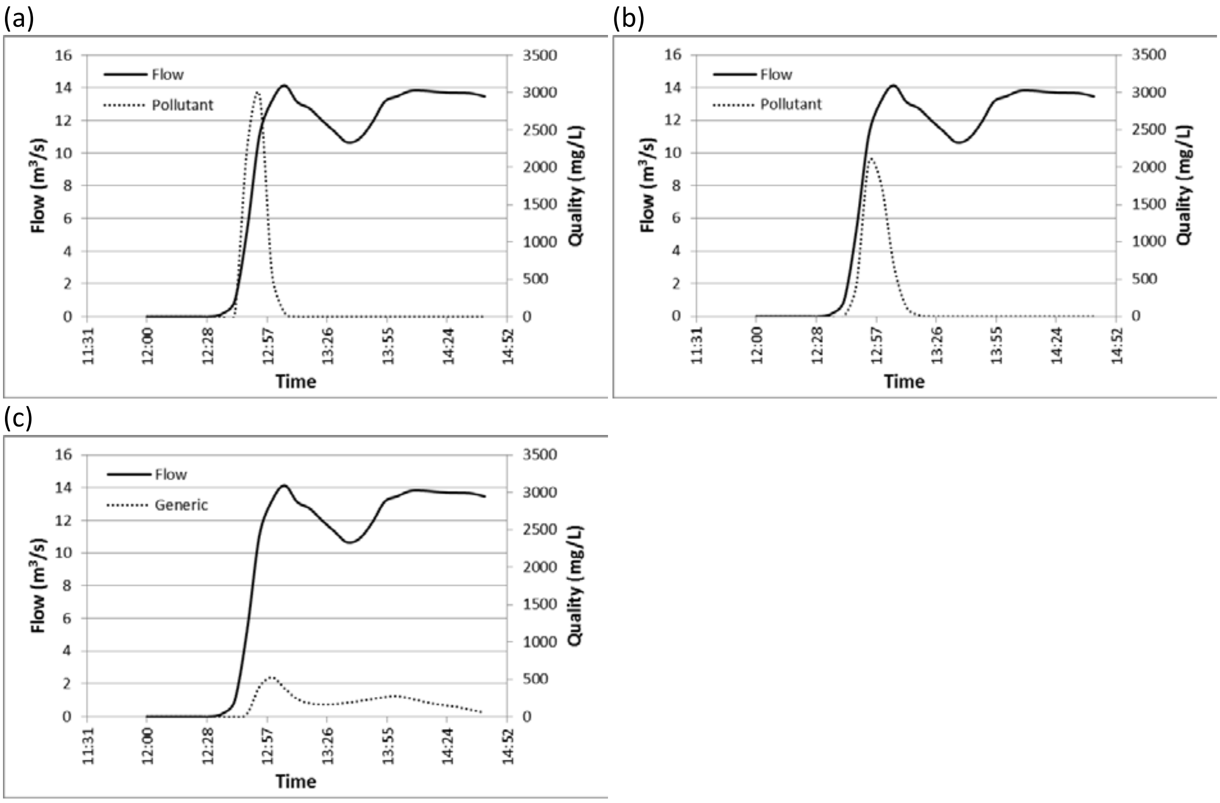


Figure 5-5 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a pollutant spill occurring at the top of the catchment [Pit-23138] at the beginning of the 10 year, 12 hour storm and with a pollutant spill duration of (a) 5 minutes (b) 30 minutes and (c) 2 hours

Exceptions to this observation occurred when the spill duration exceeded the event duration because outflows were produced from a higher ratio of pollutant spill volume and runoff volume. Longer spill durations also produced longer simulated pollutographs at the Parafield drain. There was little effect found on the travel time of the spill when the spill duration increased.

5.1.5 Effect of Storm Duration

To examine the effect of storm duration, the outlet pollutographs were compared for the half hour and two hour storms of same ARI, considering identical spills at each spill location at identical times using the data in Appendix D. It was found that longer storm durations tended to produce a longer travel time with a higher maximum and higher mean pollutant concentration at the outlet. This is because the longer duration storm tended to have a lower rainfall intensity than short duration storms, which leads to an extended hydrograph but less runoff at the time of spill during the spill event to dilute flows. An example of the effect of storm duration is shown for the 10 year, 30 minute storm and 10 year 12 hour storm in Figure 5-6.

(a) (b)

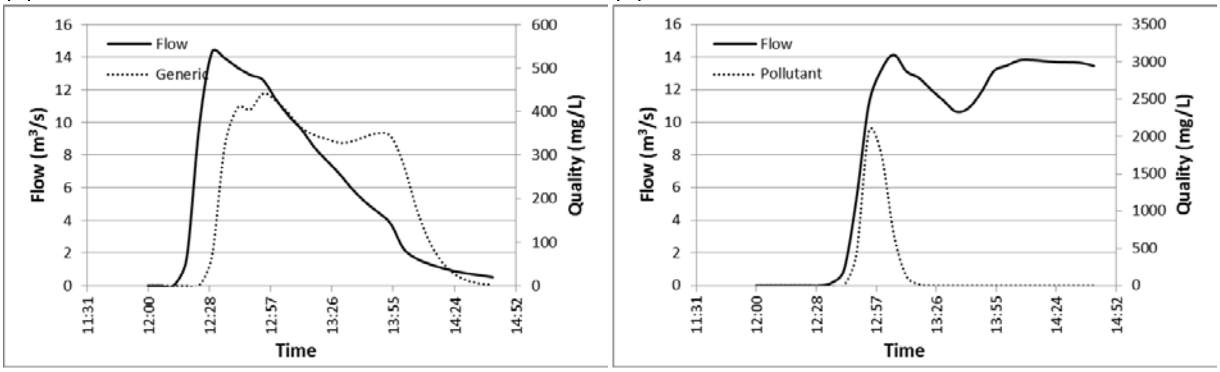


Figure 5-6 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a 30 minute pollutant spill occurring at the top of the catchment [Pit-23138] at the beginning of the (a) 10 year, 30 minute storm and (b) 10 year, 12 hour storm

5.1.6 Effect of Spill Location

To examine the effect of spill location, outlet pollutographs were compared for identical spills in two locations using the data in Appendix D, assuming equal storm events and storm duration. The PCSWMM model indicated that when a spill occurs at the farthest point in the catchment (Pit-23138) compared to an identical spill at the middle of the catchment (J102) there was a longer simulated travel time, lower simulated peak and mean concentration at the catchment outlet and a longer pollutograph period compared to the catchment outlet.

(a) (b)

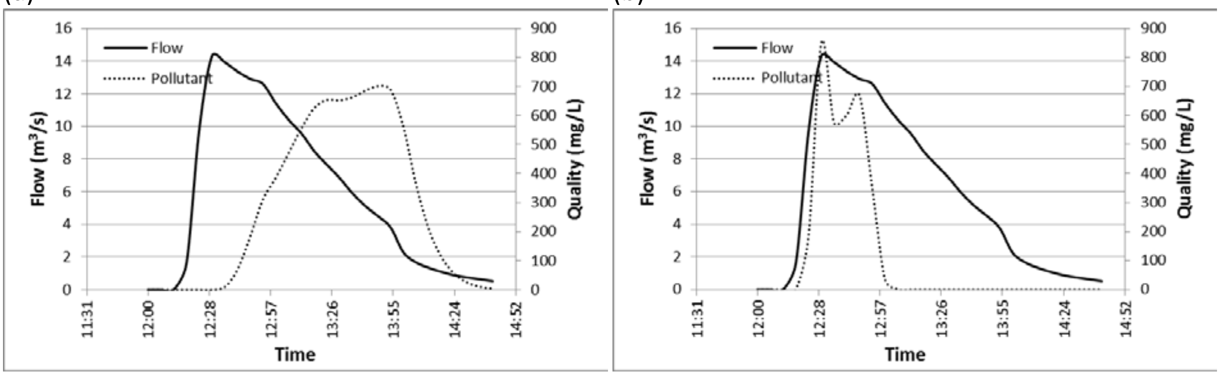


Figure 5-7 – Comparison of the flow and pollutant concentration characteristics at the Parafield drain with a 30 minute pollutant spill occurring at the middle of the 10 year, 30 minute storm (a) at the top of the catchment [Pit-23138] and (b) at the middle of the catchment [J102]

5.1.7 Effect of Spill Pollutant Concentration

To examine the effect of spill pollutant concentration, the travel time, maximum and mean concentration at the outlet were compared when the assumed pollutant concentration was halved or divided by ten in identical storms and identical spills. The initial pollutant concentration on the injected flow had a ‘linear’ impact on the analysis results – by halving the assumed pollutant concentration, the outlet peak and mean concentration was also halved. When the initial concentration was divided by ten, the outlet peak and mean concentrations were also divided by ten. The assumed pollutant travel time, peak and mean concentration were not otherwise affected.

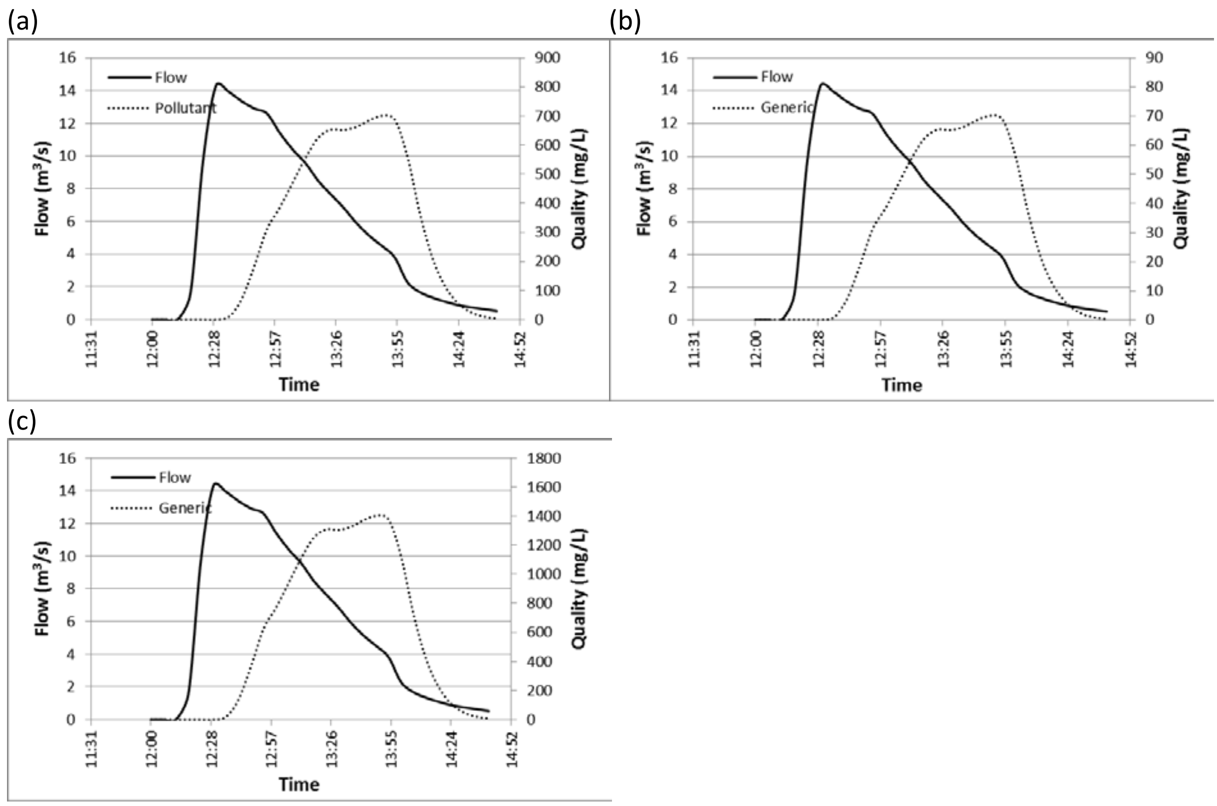


Figure 5-8 – Comparison of the flow and pollutant concentration characteristics with a pollutant spill occurring at the top of the catchment [Pit-23138] at the beginning of the 10 year, 30 minute storm and with a pollutant spill duration of 30 minutes when the assumed pollutant concentration was multiplied by (a) 0.1 (b) 1 and (c) 2 [note scale of x-axis]

5.1.8 Discussion of Wet Weather Event Spill Attenuation

The results indicated that the worst case spill characteristics, in terms of catchment travel time, were when a spill occurred at the middle of a high intensity, short duration storm event. In all of the simulated scenarios, it is important to consider that each consisted of the same volume of spill (20 kL) of a generic contaminant, and that despite fluctuations in the simulated mean and peak concentration at the outlet, each case represents the transport of the same contaminant mass.

Catchment managers should also note that while the catchment travel time is a valuable reference for determining the time of response for avoiding any contamination of stored stormwater, this time was brief in many simulations. A review of simulated pollutographs at the catchment outlet however indicated that any measures undertaken which can intercept the peak concentration portion may still play an important role in risk management as capture of the peak will be likely to capture a majority of pollutant mass. For this reason, it is recommended that future studies into pollutant transport indicate the time of travel and the time to peak concentration on the simulated pollutograph.

5.2 Wet Weather Flow – Method of Determination

Wet weather travel time refers to the time taken for flows to travel from the risk locations identified in Table 4-1 to the Parafield drain gauge (A5041049) or the detention basin at Cobbler Creek in a wet weather event. The travel time was calculated using spillage of 20 kL of liquid contaminant over a 12 minute period (ten ‘44 gallon drums’ or the volume of a small tanker truck) directly into the nearest stormwater pit to the high risk location.

Spills were assumed to coincide with two separate simulated storm events, namely the ten year ARI, half hour duration storm and the three month ARI, half hour duration storm, each determined based on a storm over Adelaide using the methods of Australian Rainfall and Runoff (Pilgrim, 1987). The spill was assumed to occur at the middle of the event, in accordance with the worst case travel time conditions found for wet weather flows in Section 5.1.

Simulation was conducted using an assumed four day period, where the selected design storm event occurred uniformly over the catchment at 12:00 (midday) on the first day. The pollutant spill was assumed to be a spill of a pure liquid pollutant (represented using a flow of water with a ‘generic’ contamination of 1 000 000 mg/L, corresponding to a pure liquid pollutant with a density of 1 kg/L). The spill was assumed to begin at 12:06, increasingly linearly to a peak spill rate at 12:12, then reducing linearly to zero spill at 12:18. Spill rate was dependent on the assumed spill volume. At the completion of the simulation, the hydrograph and pollutograph characteristics were examined at the catchment outlet. The travel time of the pollutant was determined based on the time between the beginning of the spill (12:06 pm) and the appearance of the pollutant at the catchment outlet (as defined by the appearance of a 1 mg/L concentration of pollutant in flow at the catchment outlet). The total period of the pollutograph refers to the time over which the pollutant was in excess of 1 mg/L at the catchment outlet. The event mean concentration of the generic pollutant refers to the mean concentration of pollutant in flows over the entire stormwater event, where a stormwater event was defined as the period over which flow was in excess of 1 L/s at the catchment outlet.

5.3 Catchment Travel Time – Wet Weather

The travel time for a 20 kL spill from each risk location during a 10 year, half hour storm at the middle of the event is shown in Table 5-1. A more comprehensive version of this table, and data for the 3 month storm event are provided in Appendix E.

Table 5-1 – Travel time and nature of simulated spills in PCSWMM during wet weather in the Parafield and Cobbler Creek (10 year ARI, 0.5 hour storm, 20 kL spill over 12 minutes)

Site	Model ID	Distance (m)[1]	Time of travel (mins)[2]	Time to peak conc. (mins)[3]	Peak conc. (mg/L)[4]
Site 1	Pit-25331	5,100	24	42	297
Site 2	Pit-25639	5,180	24	42	282
Site 3	Pit-20061	4,090	18	36	375
Site 4	Pit-25356	3,390	18	36	590
Site 5	J6219	3,380*	21[CC]	36	342
Site 6	J16055	1,630*	12[CC]	24	9694
Site 7	J9321	2,508*	18[CC]	30	4041
Site 8	J6995	2,100*	12[CC]	24	6625
Site 9	J23710	6,660*	36[CC]	78	1399
Site 10	Pit-18621	2,060	18	24	1835
Site 11	Pit-18250	4,280	18	36	998
Site 12	Pit-18251	3,795	18	36	2278
Site 13	Pit-24527	2,480	12	24	2533
Site 14	Pit-19692	2,160	12	24	2614
Site 15	Pit-20089	5,865	24	42	240
Site 16	J6479	4,360*	43[CC]	78	216
Site 17	Pit-16017	4,395	18	30	1669
Site 18	Pit-23138	6,800	31	54	72

[1] Approximate distance from spill location to the Parafield Airport flow gauge (A5041049), based on length of stormwater pipe. Measurements marked (*) indicate the distance to Cobbler Creek spillway, to which 6800 m must be added to account for distance from Cobbler Creek pump station to end of Parafield catchment.
[2] Time for pollutant plume to travel from spill location to flow measurement weir A5041049
[3] Time for peak of pollutant plume to travel from spill location to flow measurement weir A5041049
[4] Peak concentration at the flow measurement weir A5041049
[CC] Values indicate travel time to Cobbler Creek detention basin only

A significant finding from the wet weather flow study is the speed with which water and associated contaminants may travel during a high flow event (such as that represented by a 10 year ARI, half hour storm or greater). For the Parafield catchment, the simulated catchment travel time was found to be between 18 and 31 minutes at the closest high risk location and furthest reach of the catchment, respectively. Notably, the peak pollutant concentration at the outlet was reached between 36 and 54 minutes after the spill, respectively. In the Cobbler Creek catchment, the travel time was between 12 to 43 minutes for the high risk locations closest to and furthest from the catchment outlet, respectively, and the peak concentration reached the Cobbler Creek detention basin within 24 to 78 minutes.

In each case, the travel time of the pollutant flow (including the time to peak concentration) was significantly less than the predicted travel time of lower ARI events (see Appendix E) and less than dry weather pollutant spill flows (Section 4.3). More importantly, these travel periods leave little time for catchment managers to respond to any immediate knowledge of spills or overflows in the catchment.

The results in Table 5-1 reveal that the catchment travel time predicted by PCSWMM is typically closer to that travel time predicted by the Kirpich equation (Section 4.2.3, Page 39) following adjustment by the modification factor recommended by Chow (1988). This finding is considered appropriate because the theoretical travel time is intended for design purposes, and thus should provide a prediction of the time of concentration (in this case, travel time of pollutant to outlet) during high flow conditions.

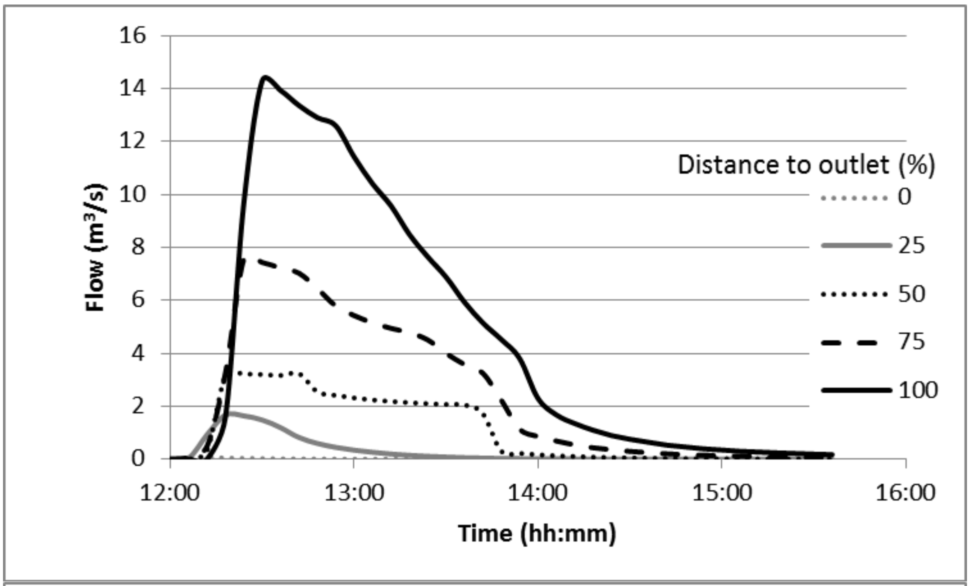
The short duration travel times indicated in this analysis indicate that there is little time to respond to the occurrence of a spill event in most sections of the catchment during wet periods. There is therefore a need to focus on identification of potential hazards and consideration of actions which can reduce them. For example, while the time is limited to respond to knowledge of a hazardous spill or overflow event, the hazard identification process revealed high risk locations which may be considered a priority for risk management. For example, where sanitary sewer overflow has been problematic in the past, existing infrastructure should be investigated and potentially upgraded to reduce the risk of future occurrence.

The short travel times in Table 5-1 also indicate a need to focus on online monitoring at point of harvest which can identify hazards which affect infrastructure, treated water quality or the risk of storing and using harvested water. The presence of such systems should also allow for immediate rejection and/or flushing of waters from point of harvest in the event of a contaminant plume being detected, subject to environmental requirements downstream.

5.4 The Impact of Dilution in Wet Weather

The impact of dilution during wet weather events was analysed with respect to distance from the point of the assumed spill (Sites 1 to 18 in Section 4.1). This was conducted by examining the hydrograph (flow rate over time at a fixed point) and pollutograph (pollutant concentration over time at a fixed point) at the 0% of the distance to outlet (point of spill), 25 % of this distance, 50% of this distance, 75% of this distance and 100% of this distance (at the catchment outlet). The results of this analysis are presented for the Parafield catchment in Appendix G. An example of the findings is presented in Figure 5-9. This analysis was not conducted for spill locations in the Cobbler Creek catchment because the stormwater pipe network was simplified due to the limited data on pipe and pit elevations in the region.

(a)



(b)

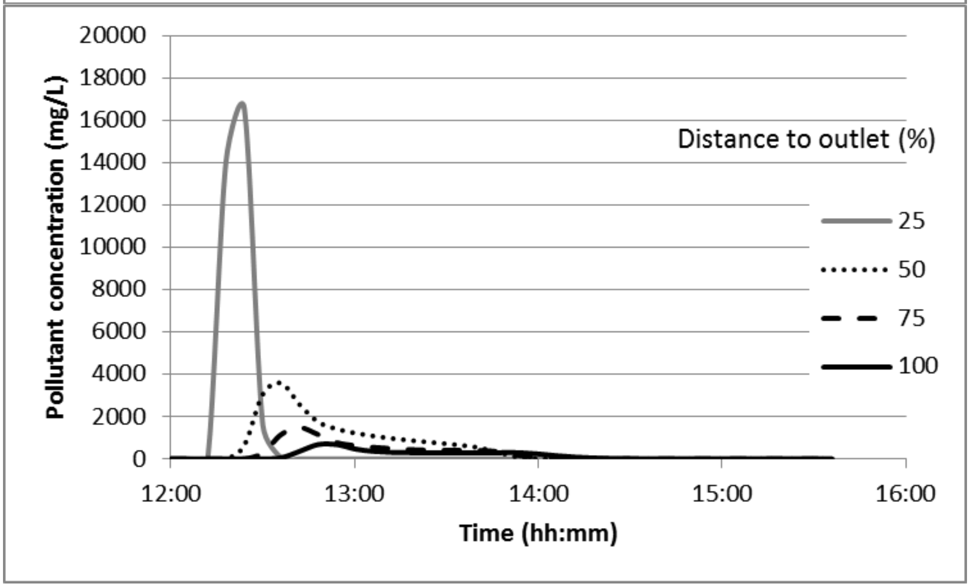


Figure 5-9 – The (a) Hydrograph and (b) pollutograph with distance along the flow path from Pit-23138 to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 6800 m.

The analysis indicated that with increasing distance along the spill travel pathway, the flow rate increased because of the additional flow entering the pathway from the number of interconnecting conduits and their subcatchments. Conversely, the effect of this increasing flow causes the pollutant concentration to decrease with distance as the pollutant concentration was diluted with water not yet affected by the spill. It is also clear that longer pathways tend to result in additional dilution, a matter which is also shown in the peak concentration data in Figure 5-1. It should be noted that this may not be the case for all systems. For example, in other water catchments, a conduit may travel a long distance and not receive any inflow from other subcatchments, while a relatively short distance in the Parafield catchment may receive runoff from a large number of impervious surface catchments in a heavily developed urban location.

5.5 Summary

This section has described the travel time of flows during wet weather spill events. Before undertaking wet weather spill simulations, preliminary modelling was undertaken to determine the most conservative assumptions for the assumed rainfall characteristics, timing of spill and duration of spill. It was determined that larger events tended to carry a pollutant faster to the catchment outlet, but with higher levels of dilution. Faster travel times also result when a spill was assumed to occur in the middle of a storm compared to the beginning, and that high intensity, short duration storms produced a faster travel time than longer duration storms. Based on these results, two short duration storms (the half hour duration, three month and ten year ARI design storm for the Adelaide region) were selected to represent wet weather conditions for spill analysis with pollutant spills occurring at high risk locations in the middle of the event.

The simulated catchment travel time for wet weather spills was found to be even faster than the dry weather spills, with a travel time of between 18 and 31 minutes at the closest high risk location and furthest reach of the catchment, respectively. The peak pollutant concentration at the outlet in each case was reached between 36 and 54 minutes after the spill, respectively. For the Cobbler Creek catchment, the travel time was between 12 to 43 minutes for the high risk locations closest to and furthest from the catchment outlet, respectively, and the peak concentration reached the detention basin within 24 to 78 minutes. These results indicate a very limited period of time to allow human intervention to respond to divert all or the majority of the pollutant spill from reaching the instream basin of the Parafield harvesting scheme. While pollutant dilution was shown to occur to a large extent in the catchment, there was limited time for it to occur sufficiently and peak pollutant flows occurred in relatively short periods of time after the pollutant spill first became apparent at the catchment outlet.

6 Recommendations for Risk Management

6.1 Operations and Control Measures

The following paragraphs address operations and control measures for risk management in the Parafield and Cobbler Creek catchments.

The flow measurement and modelling simulations undertaken for this report have indicated that there is a limited time available to respond to pollutant spills, overflows or other events which can affect water quality in the instream basin of the Parafield harvesting scheme. This indicates that human intervention into the harvesting program after spills occur may not be appropriate as a *sole* mechanism for managing water quality hazards within the catchment. As such, key opportunities for risk management in addition to human intervention are considered to be:

- The completion and adherence to a strict catchment hazard analysis and critical control points plan which identifies risks and attempts to control them to prevent spillage, overflow or continuous pollution to occur
- The installation of online water quality monitoring infrastructure which can automatically detect significant variations in water quality parameters, overcoming the need for human communication and action such as manual intervention in automatically controlled harvesting procedures. It should be noted that the use of online monitoring may provide limited benefit at the present time. While online monitoring tools will be effective for detecting unacceptable levels of physical parameters (such as pH, colour, turbidity and salinity), there are no known online monitoring devices which can detect fluctuations in other pollutants such as pathogens, herbicides, pesticides, nor for dissolved hydrocarbons such as fuel, oil and grease. Research is ongoing into the provision of tools for online monitoring of pollutants and their surrogates, and this should be considered in the development stormwater harvesting schemes.

It is known that there is no mechanism for diverting flow away from the 48 ML instream basin of the Parafield harvesting scheme (Section 2.4.3) as all water flows into the instream basin (except overflows) and are subsequently pumped out. Although there is a limited time available for human intervention to respond to spills in the catchment, it is suggested that the ability to divert flows away from the instream basin in the event of a spill may be of benefit, especially if this mechanism is linked to online monitoring tools which can reject flows automatically based on high pollutant concentrations. This may improve the overall quality of harvested water rather than harvesting all runoff flows.

Further to the previous point however, under the current arrangements, the 48 ML instream basin, in the absence of diversion infrastructure, may be considered to be a final water quality control at the end of the Parafield catchment which prevents hazardous flows from proceeding to the Barker Inlet wetlands. If hazardous flows are intercepted, water may be detained within the basin and disposed of back into the Parafield drain downstream of the inlet works once dilution or treatment processes have been allowed to take place. However, this will require human intervention to override the automatic pump which diverts water from the instream basin to the second 49 ML holding basin. In cases where flow is intercepted by the basin, the potential contamination of basin surfaces should be investigated prior to recommencing stormwater harvest processes.

It is recommended that the implications of flow rejection are further researched to explore the responsibility of scheme operators (such as City of Salisbury) for flows which are rejected on water quality grounds. Using the Parafield system as an example, if flow is intercepted by the instream basin and found to be polluted due to contamination events upstream, is it appropriate for this water to be knowingly pumped out of the basin and sent downstream to enter Dry Creek and the Barker Inlet wetlands? What kind of treatment, if any, may be applied in the basin should a hazardous flow be intercepted? Who are the stakeholders when determining whether water polluted with hazardous substances should be rejected downstream, with or without treatment? And which authorities should be informed prior to doing so? Similar questions may also be raised should the Cobbler Creek detention basin intercept flows polluted with hazardous substances, with the additional complication that this basin provides an important function in flood management downstream.

It is also recommended that field monitoring for pathogens continues to be undertaken at or near the entry to the instream basin during storm events, especially where runoff from pervious areas is likely to occur (for long duration events). This is because it is possible that monitoring has not captured runoff from pervious areas where livestock are noted to be present. The harvesting scheme operators should also be aware of any expansion in livestock operations or cultivation within the catchment

The potential for future industrial or residential development at the former Bridgestone site at the south west of the catchment should be monitored by catchment managers. This is a particularly high risk location because it is a short distance to the outlet of the catchment. Development will also be likely to increase the portion of impervious area in this section of the catchment which will increase runoff volumes in the Parafield catchment, particularly in the early stages of storm events.

Limited catchment controls exist in some residential, industrial and quarrying areas in the form of stormwater detention basins. The presence of these basins can be considered of benefit for risk management purposes because they will intercept spills and overflows before they reach the catchment outlet. However, these basins will need to be emptied or otherwise treated before all, or the bulk of, their contents is allowed to proceed downstream. The most obvious structural measures within the catchment consist of stormwater basins in residential areas and for runoff management of quarry sites. The location of these basins was presented in Section 2.4.8 and Figure 2-7. While these basins may intercept spills and overflows which occur within their respective catchment area, it should be noted that they ultimately flow into the Parafield catchment in conditions of overflow, or through slow drainage after rainfall. In the event of a hazardous event causing polluted runoff in the catchment of these basins, catchment managers should take measures to ensure that subsequent runoff events do not lead to polluted basin outflows.

City of tea Tree Gully (pers. comm.) suggest that the majority of the turbidity observed from the Cobbler Creek catchment is sourced from residential development construction, particularly that proceeding north of Crouch Road and Slate Creek, Golden Grove. It is therefore recommended that large scale greenfield and brownfield development in the catchment of stormwater harvesting and reuse facilities be more effectively controlled with effective sediment control plans which recognise the fate of stormwater runoff downstream of the developing catchment.

A review of sewer overflows in the Parafield and Cobbler Creek catchment indicated that while overflows occurred in a variety of locations across the catchment over the eight year period considered, there was a distinct pattern of sewer overflows occurring at the bottom of steep slopes. To reduce the occurrence of sewer overflows in a harvesting scheme catchment, it is recommended that catchment managers review the available data on sewer overflows and, where possible, upgrade infrastructure in high risk locations to cope with higher flows.

The background research for this report indicated several structural and non-structural measures for risk management already in place within the catchment. A summary of non-structural measures was provided by Swierc *et al.* (2005), including general information of several licensing and education measures within the Parafield catchment.

6.2 Key Risk Locations

The key risk locations assumed in this study were sourced from a previous report summarising catchment hazards (Page et al, 2011). Overall, however, the risk locations of greatest concern would generally be identified as those which provide the highest risk for contamination events and the shortest travel time to the harvest point at Parafield airport. The findings of this report however have indicated that the travel time from even the furthest point in the Parafield catchment was relatively short. Short travel times are also present in the Cobbler Creek catchment. Furthermore, it is difficult to intercept Cobbler Creek flows due to the automatic operation of the Cobbler Creek pump which draws a portion of Cobbler Creek retention basin flows into the Parafield catchment. As such, the hazard locations identified by Page et al. (2011) are all considered to allow transport to the harvesting location in a very short time.

6.3 General Recommendations for Simulating Harvesting Scheme Operations

This investigation has led to an improved understanding of how hazard event simulation should be approached for stormwater harvesting schemes sourcing water from developed and undeveloped catchments. The following general advice is provided for conducting future analysis of stormwater harvesting scheme operations for hazard analysis and control.

1. Communication with current and previous operators, researchers and designers

The first and most important step in any analysis of a stormwater harvesting and reuse scheme is considered to be communication with people who have previously worked on the scheme design and operation. This may include local authorities, state authorities, catchment managers, scheme designers, land developers, researchers and professional consultants known to have undertaken works within the catchment. It is also considered important to establish a contact person in local governments where the project is located. These lines of communication may be used to make immediate queries when they arise, to arrange site visits and to find out about the existence of and arrange access to data which will aid in the process of simulating the system.

2. Compile a library of existing reports and other literature relating to the scheme

It is beneficial to be aware of previous studies and key assumptions they have used before undertaking a new modelling approach. Existing studies may provide significant historical information and design revisions which may or may not be included on existing digital maps or project briefs. Existing literature may also indicate the presence of unknown sources of data such as flows and rainfall, or identify issues with these data which will save time in analysis of the data and prevent calibration of the model with problematic data.

3. Compile and assess a library of data relevant to the catchment

Relevant data for describing the catchment includes any information on the catchment layout, including spatial information and elevations, the layout of any infrastructure (especially stormwater pits, pipes and pumps) as well as any measured rainfall and flow information. Climate data, such as rainfall and evapotranspiration should also be sourced from the relevant authorities including the Bureau of Meteorology, Department for Water (or local equivalent), natural resource management boards and local water authorities, where appropriate.

4. Examine key land uses

It is important to determine key land uses for any consideration of runoff volume and water quality. It is also important to cross check land use zoning with existing development via site visits and review of recent orthophotomaps. Areas zoned with high levels of industrial development in spatial data, for example, may be assumed to have a high runoff and pollutant load due to characteristically high levels of imperviousness and pollutant load generation. However, should the industrially zoned area consist of disused factories or large tracts of undeveloped land, there is effectively no process based pollutant load being generated, and any assumptions regarding stormflow or water quality hazards may not be justified.

5. Identify the location of any storages which may (a) have an impact on flows and (b) present opportunities for stormwater runoff detention.

Detention basins are an important consideration because they may be considered both a control point for polluted flows and a hazard. Detention basins provide the benefit of spill interception and/or attenuation, which can be important in the prevention of a pollutant affecting flows at the point of harvest. However, should a spill be intercepted by a detention basin, the bulk of the contaminated water in the basin must be disposed of or treated before it is allowed to flow through to the point of harvest. Detention basins can present a hazard that needs to be managed to reduce risks to public health.

6. Determine key travel times with input from observed measurements

The travel time of water through the catchment should be determined via field experimentation during model calibration. This is because the travel time is a key variable which indicates the time for catchment managers to respond to known hazards which have reached the stormwater network. key factors which can influence travel time are distance and slope (which may be obtained from construction records, where available) and Manning's ‘n’ roughness coefficient. Assessing catchment travel times using real water flows where it is possible to do so allows the calibration of the often difficult to determine Manning's 'n' value or other routing parameters which affect time of travel through pipes and engineered as well as natural channels.

7. Establish an understanding of operations management in the catchment

It is important to establish an understanding of key operations rules used in the catchment which may not be documented in available literature. For example, a series of general operations management rules for the Parafield catchment were presented in Section 2.4.3 which were not explicitly stated in existing literature on the Parafield scheme.

6.4 Summary

Previous sections of the report indicated that the travel time of spills and overflows in dry and wet weather were relatively short. It was considered unlikely that human intervention would be appropriate for the holistic management of pollutant spills. A series of recommendations for improving catchment management practices were provided in this section. Key recommendations include the suggestion that the main avenues for managing spills are by conducting appropriate and up-to-date hazard analysis within the catchment boundary, and the installation of online monitoring systems which can automate the procedure for rejecting flow from the harvesting scheme, should this be acceptable downstream, and for detention basins to be included within the drainage system where practical.

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Appendix A – PCSWMM Model Parameters

PCSWMM model parameters

There are three main components to the Parafield catchment model. These are:

1. Subcatchment areas (sub-areas of the catchment which drain to a stormwater pit or stream, of which 1463 were delineated)
2. Conduits (streams or stormwater pipes, in 1826 individual segments)
3. Nodes (stormwater pits and defined flow junctions, of which 1823 were determined).

Each of these main features has a series of characteristics or 'model parameters' which influence how catchment features impact the generation and/or transport of rainfall runoff. The following sections outline these model parameters and how they were determined for the Parafield catchment model.

Subcatchment Properties

Name

The name of a subcatchment does not influence the model results. In most cases, catchment names have been adopted based on the mini-catchments GIS data. Newly developed catchments are named automatically in order of their construction.

X and Y Coordinate

The X and Y coordinate values refer to the location of mapping data in the PCSWMM GIS interface. The X and Y coordinate data is spatial only and does not affect the outcome of the model, except where it specified length and other properties. However, it should be noted by other users that all data imported into the PCSWMM model was from GIS files projected to **GDA94 MGA Zone 54**. X and Y coordinates were not adjusted during model calibration.

Description

The description data box is a general information box that does not affect the model results. It is used for the input of general information during modelling. In some cases notes have been made during modelling for the benefit of other users.

Tag

The catchment tag does not have an influence on the model results. It is generally used to insert an alternate name for a model element. In this case, the catchment tags have been imported as the alternate catchment names from the mini-catchments GIS data.

Rain Gauge

The catchment 'rain gauge' parameter indicates the rainfall station corresponding to the catchment. Rainfall data from the relevant file representing data acquired from this gauge will be applied to the catchment when the model is run. Many rain gauges can be represented in the one model. Further information on the selection and designation of appropriate rain gauges used in the PCSWMM model of the Parafield catchment was presented in Section 2.4.5. The assignment of rain gauges to each catchment was not adjusted during the calibration process.

Outlet

The subcatchment outlet refers to the point (usually a stormwater pit) in any subcatchment to which stormwater runoff drains to. The subcatchment outlet for each catchment in the model was assigned based on the outlet pit indicated in the mini-catchments GIS data. In some cases, the subcatchment outlet was manually assigned based on proximity to other known pits where:

- There was no known pit corresponding to that referenced in the mini-catchments GIS data (i.e. the referenced pit did not exist in the model)
- The outlet pit was deleted due to inadequate data on invert and surface elevation
- The outlet pit was deleted due to errors caused by very short conduits
- The outlet was not connected to any known stormwater pipe (in which case the outlet was assigned as the nearest pit with a pipe).

Area (Ha)

The area parameter refers to the plan area of the subcatchment. The area of each subcatchment was extracted from the mini-catchments GIS data. Where catchments were drawn into the model, drawing was conducted to scale and area was automatically determined. The subcatchments ranged in size from approximately 10 m^2 up to 60 Ha. Smaller catchments were generally located in the City of Salisbury area and were based on the mini catchments GIS data delineation (Cardno-Willing NSW, 2008) while larger catchments were rural areas which were delineated manually for this project.

Width (m)

In accordance with the methods described by Rossman (2010), catchment width was estimated using Equation A1:

$$\frac{Catchment\ area\ (m^2)}{Max.overland\ flow\ Length\ (m)} \qquad \text{- Equation A1}$$

The maximum overland flow length represents the distance from the catchment outlet to the furthest drainage point in the subcatchment. In this analysis, flow length was assumed to be equal to the mini-catchment “length” in the mini catchment GIS data (Cardno-Willing NSW, 2008) supplied by City of Salisbury.

The width parameter is commonly adjusted during the model calibration process in the SWMM model. To achieve an appropriate calibration, the width of all catchments was adopted as **10%** of the value determined using the relationship in Equation A1. This was determined by varying the width parameter uniformly across all catchments to achieve the most appropriate model fit during calibration. Variation of individual values was not considered in this project because only one flow gauge was available for calibration at the end of the catchment.

Flow Length (m)

The flow length was derived from the mini-catchments GIS model (Cardno-Willing NSW, 2008), importing values of 'Shape_leng' data. Other values were determined by PCSWMM when shapes were imported from the GIS data file used to delineate extra catchments. The flow length is not used in the current version of the SWMM model, but has been included as a carry-over feature from previous versions. It ranges from 0.6 to 3250 m, and was proportional to catchment size.

Slope (%)

For catchments in the City of Salisbury delineated by Cardno Willing NSW (2008) and additional catchments outside these boundaries, the slope of the catchments was estimated using ArcMap 10. Using a layer of point elevation data provided by City of Salisbury, the average slope within each mini-catchment was estimated using the spatial analyst toolbox. Percentage slope between each point in the layer was determined using the slope tool (Spatial analyst – Surface – Slope) and the average slope value within each subcatchment of the PCSWMM model was determined using the mean slope (Spatial Analyst tools – Zonal – Zonal statistics as table). The following provides a step-by-step guide to the approach:

1. Export PCSWMM mini-catchment data as a shape file (Use export tool)
2. Convert this to a raster data set in ArcCatalog (Conversion tools – to raster – feature to raster, number of catchments picked up in raster is dependent on 'Output cell size')
3. Convert elevation points to slope raster data (Spatial analyst – Surface – Slope)
4. Use zonal statistics as table (Spatial Analyst tools – Zonal – Zonal statistics as table) to determine the mean slope values of the point data with respect to the PCSWMM catchments

The slope ranged from 0.01 to 18%, with higher slopes in the hill face region, roughly central to the modelled area.

Imperviousness (%)

The catchment imperviousness was imported into PCSWMM with the mini-catchments GIS data. The imperviousness of the catchments was reviewed in PCSWMM using an orthophotomap of the region, indicating some catchments with obvious discrepancies from the imported values. This includes several open space catchments labelled 50% impervious, particularly in undeveloped industrial zones. In such cases, impervious areas were estimated based on visual analysis of the orthophotomap (usually 0% impervious).

At the conclusion of the analysis, the catchment impervious value was adjusted to produce greater runoff volumes during the runoff calibration procedure. As such, **all catchments with an existing imperviousness greater than 20% were considered to have an impervious value 5% greater** than that provided by the mini catchments GIS data. This measure was adopted during calibration to increase the quantity of runoff to more adequately match observed runoff. This may also be adjusted by changing other values, such as the depression storage, however this focusses the extra runoff at the beginning of a storm, thus influencing flow rate at the beginning of a storm event. By changing this value, runoff quantity was increased across the rainfall event as opposed to an immediate boost at the beginning.

As a final measure of imperviousness, it was found that the Parafield catchment had a total imperviousness equal to 22.4% (349 of 1552 Ha) and the Cobbler Creek catchment was approximately 18% impervious (129 of 697 Ha).

N Imperv

The N Imperv value refers to the Manning's 'n' coefficient of the impervious surface of the catchment. Using the typical N Perv values provided by Rossman (2010), this value was altered during the calibration process and found to be approximately 0.04 across the catchment.

N Perv

The N perv value refers to the Manning's 'n' coefficient of the pervious surface of the catchment. Using the typical N Perv values provided by Rossman (2010), this value was altered during the calibration process and found to be approximately 0.15 across the catchment.

D store Imperv (mm)

The *D store Imperv* value refers to the mean depth of the depression storage across a sub-catchment impervious surface. Based on the typical depression storage data for impervious surfaces presented by Rossman (2010), this value was altered during the calibration procedure and was found to be equal to approximately 2 mm across the catchment.

D store Perv (mm)

The *D store Perv* value refers to the mean depth of the depression storage across a sub-catchment pervious surface. Based on the typical depression storage data for pervious surfaces presented by Rossman (2010), this value was altered during the calibration process and found to equal approximately 3.8 mm across the catchment (representing a high range value for lawns).

Although areas of open space exist in the eastern portion of the catchment which may correspond to higher pervious area storage values (such as values relevant to forest litter and pasture) these areas are the steepest area of the catchment and as such a value more typical of lawns was adopted. Furthermore, there was no flow data which allowed for more discrete assumptions to be examined.

Zero Imperv (%)

The *Zero Imperv* value refers to the percentage of the impervious catchment area that has no depression storage. There was little guidance on the selection of a value for *Zero Imperv*, and as such the value was altered during the calibration process and set to 50%.

Subarea routing

Subarea routing refers to the nature of runoff routing within a subcatchment, between the pervious and impervious areas. There are three options to select:

1. IMPERV, where runoff from the pervious area flows to the impervious area
2. PERV, where runoff from the impervious area flows to the impervious area
3. OUTLET, where runoff from both pervious and impervious areas flows directly to the outlet

For the Parafield model, the OUTLET option was specified.

Percent routed

The percent routed figure refers to the percentage of sub-catchment runoff routed between areas in the sub-catchment areas (in accordance with the selected subarea routing routine). The percent routed value used in the model was 100%

Curb Length

The 'curb' (kerb) length refers to the length of formal kerb and gutter drainage in the catchment area. Note that this value is only a consideration when determining pollutant build-up based on kerb length. There was no diffuse pollutant build-up considered in the Parafield catchment model.

Snow pack

There was no allowance for snow conditions considered in the model.

LID controls

There were no Low-Impact Development (LID, used similarly in this context to the Australian term Water Sensitive Urban Design, WSUD) controls assumed to be present at the model calibration stage. Such controls would include infiltration systems, wetlands and bioretention systems.

Groundwater

There was no groundwater component assumed to be present in the model.

Infiltration

There are three options for modelling the infiltration of water into the unsaturated surface soil in the SWMM model:

1. Horton infiltration
2. Green-Ampt infiltration
3. SCS curve number infiltration

The Horton infiltration model was assumed to be appropriate for this modelling exercise. Research has shown that there is little difference between the results of the Green-Ampt and Horton equation in the modelling of infiltration (Hsu *et al.*, 2002).

Horton Infiltration Parameters

The Horton infiltration model determines the rate at which the infiltration rate declines during a storm as soil moisture increases. The Horton infiltration model can be described using the following equation:

$$f_t = f_c + (f_0 - f_c)e^{-kt}$$

Where

f_t	=	The infiltration rate at time *t* (mm/hr)
f_c	=	The constant 'equilibrium' infiltration rate for the soil in a saturated condition (mm/hr)
f_0	=	The initial infiltration rate of the soil (mm/hr)
k	=	The decay rate of the soil infiltration rate (hour^{-1})
t	=	Time (hours)

These and other key parameters of the Horton infiltration model applied in PCSWMM (drying time and maximum volume) are further described in the following paragraphs. A diagram representing these parameters in a storm event is presented in Figure A 1.

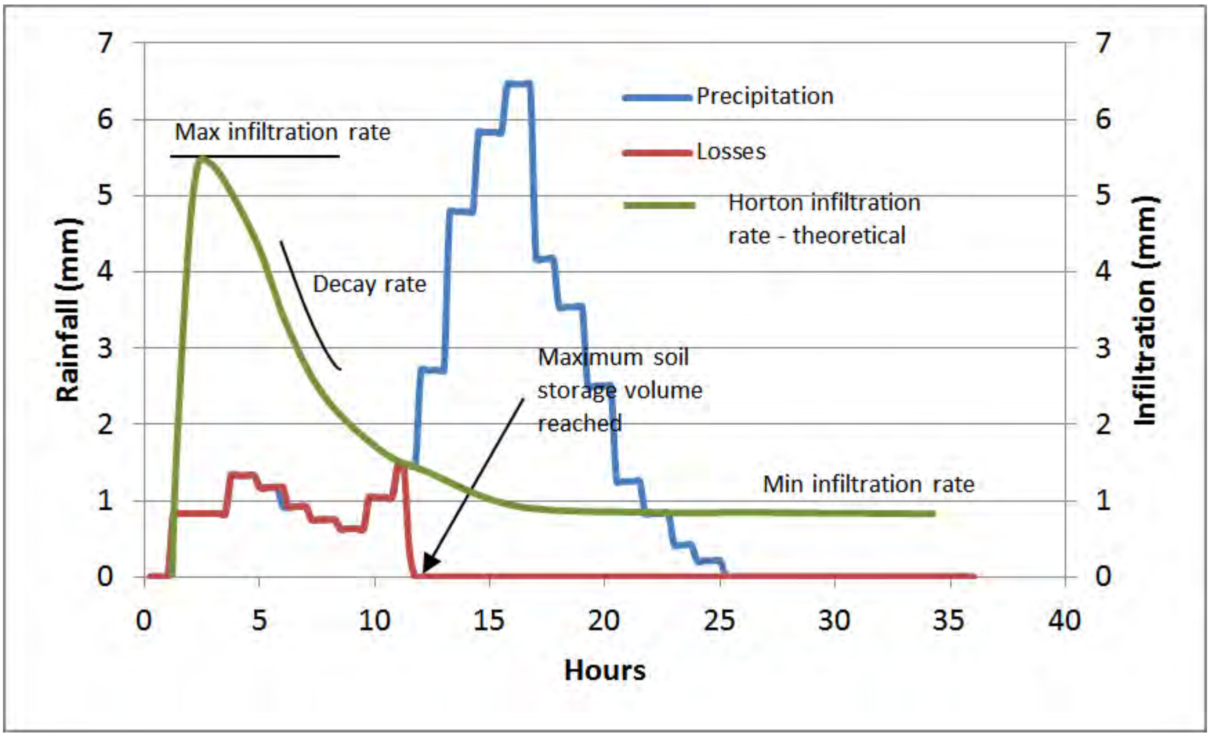


Figure A 1 – Diagram indicating how the Horton Infiltration model proceeds to calculate infiltration during a storm

Maximum Infiltration Rate (f_0, mm/hr)

According to the digital atlas of Australian soils, the catchment area consists of 'hard alkaline red soils'. Extrapolating from spatial data supplied by PIRSA, the region is characterised by clay-loam soils. Based on this information and the values recommended by Rossman (2010), the maximum infiltration rate was assumed and altered slightly during the calibration process where it was found to be 6 mm/hr across the catchment (a rate between that of a clay and loam soil). This value also corresponds to soil with little to no vegetation, which appears to be the case based on aerial photographs of the catchment area – the area is typically urban, with some open space dominated by grass and other small vegetation.

Minimum Infiltration Rate (f_c, mm/hr)

The minimum infiltration rate of all subcatchments is equivalent to the saturated hydraulic conductivity of a soil. This was assumed to be equivalent to a clay-loam soil for all sub-catchments, which according to Rossman (2010) is approximately 1 mm/hr. This value was altered slightly during calibration and a value of 3 mm/hr was adopted.

Decay rate (k, 1/hr)

The decay rate is the inverse of the time it takes for the infiltration rate of the soil to decay from its maximum value to its minimum value. According to Rossman (2010) this value typically ranges from 2 to 7 hours^{-1}. Based on alteration of this value during model calibration, a value of 0.1 was adopted in this model. This indicates that the decay of the infiltration rate in the catchment was slow (i.e. the decline in the infiltration rate during the progress of a storm was lower than that typically assumed).

Drying time (Days)

The drying time refers to the time it takes for the infiltration zone to dry out completely (during dry weather); after this time, in the event of rainfall, the soil will begin infiltration at the maximum infiltration rate. The drying time adopted for the Parafield catchment model was determined during calibration to be 3 days.

Maximum Volume (mm)

According to Rossman (2010), the maximum volume can be estimated as the difference between a soil's porosity and its wilting point times the depth of the infiltration zone. Based on recommended values for porosity and wilting point provided for clay-loam soils by Rossman (2010), this value was assumed to be approximately 30 mm. After calibration, the value was adopted as 90 mm. It should be noted that this parameter represents the storage capacity of soil, while the 'D store perv' value represents the storage of the pervious surface environment due to ponding (i.e. D store perv does not represent the subsurface)

Conduit properties

Conduit properties were adopted based largely on the properties in the stormwater pipe information provided by the South Australian Department for Water (Stormwater Pipe GIS model)[8]

Name

The conduit name has no impact on the model results. The name of each conduit was imported from the 'OBJECTID' field in the attribute table of the stormwater pipe GIS model.

Inlet node

The inlet node for each conduit was typically assigned by cross referencing the information in the stormwater pipe GIS model with the stormwater pit GIS model. In some cases, there was no inlet node specified. This included circumstances such as:

- Conduits split into several sub components, which were removed and replaced by a single equivalent conduit
- Conduits which did not correspond to any visible inlet nodes. In cases where there was no clear source of flow, these conduits were deleted from the model. In other cases, a pit was manually assigned based on proximity.

[8] Available from https://www.waterconnect.sa.gov.au/WaterResources/SurfaceWater/Pages/default.aspx

Outlet node

The outlet node for each conduit was typically assigned by cross referencing the information in the stormwater pipe GIS model with the stormwater pit GIS model. In some cases, there was no outlet node specified. This included circumstances such as those outlined for inlet nodes, above.

Description

The conduit description has no influence on the model results. This data field was used to make notes about conduit properties when required.

Tag

The conduit tag has no influence on the model results. The conduit tag was used to indicate the pipe material using the MATERIAL field from the stormwater pipes GIS model. In most cases, however, a material was not specified.

Length (m)

Pipe length (m) refers to the length of each conduit section. The pipe length was determined in two ways:

- For pipes which required no alteration, length was imported into PCSWMM based on the 'Shape_len' column in the stormwater pipes GIS model attribute table
- For pipes which required alteration, length was determined using the PCSWMM auto-length feature – this feature allows any object drawn on a scale model to be automatically determined.

Roughness

Pipe roughness refers to the Manning's 'n' value of the pipe surface material. Recommendations for Manning's 'n' for various pipe materials were provided by Rossman (2010). For the purposes of this study, there were three values adopted

- A value of 0.01 was assumed for pipes, based on a high end value for clay and concrete pipes
- A value of 0.01 was used for open concrete channels, including the Parafield drain at the end of the catchment
- A value of 0.03 was used for natural or unlined channels

The value of pipe roughness for pipes and open concrete channels was calibrated using the flow measurement procedure described in Section 3.3.1.

Inlet offset (m)

The inlet offset refers to the height (based on a datum) or depth (from invert of the inlet node) of the conduit inlet. It is used to define where a conduit is connected to the conduit inlet, and to determine a conduit slope. An illustration of this value is indicated in Figure A 2 below.

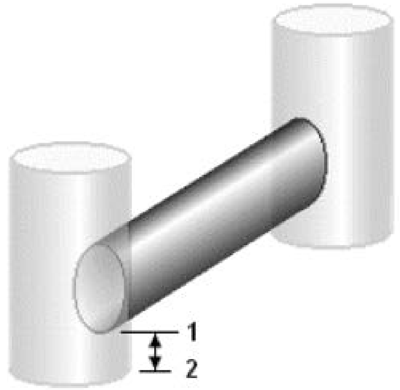


Figure A 2 – Demonstration of the Inlet offset value where elevation is represented by depth from surface

For the Parafield model, the conduit inlet offset was adopted as an elevation, a fixed height above Australian Height Datum (AHD), which was imported from the 'Inv_in' column in the stormwater pipes GIS model attribute table. In some cases data was manually entered for inlet offset. This was because

- The inlet offset value was not provided, in which case the inlet offset was determined to be at the very base of the inlet pit or, in the case of open channels, the height of the channel base at the inlet
- The inlet offset value was not sensible (i.e. the inlet offset was higher or lower than the range of the inlet pit depth). In this case, the value was adjusted to be at the base of the inlet pit or, in the case of open channels, the height of the channel base at the inlet
- The inlet offset value provided a 'negative slope', where flows would be expected to go upwards through the stormwater network. In these cases, the inlet offset was manually adjusted to provide an even grade to the nearest pit with a known, sensible invert.

Outlet offset (m)

The inlet offset refers to the height (based on a datum) or depth (from invert of the inlet node) of the conduit inlet. It is used to define where a conduit is connected to the conduit inlet, and to determine a conduit slope. The conduit inlet offset was adopted as a height above AHD, which was imported from the 'Inv_out' column in the stormwater pipes GIS model attribute table. In some cases data was manually entered for inlet offset. This was due carried out in a similar manner to those situations outlined above for inlet offset.

Initial flow (m^3/s)

The initial flow refers to the flows in a conduit which were present at the beginning of the model run period. There were no initial flows assumed to be present in the Parafield catchment model.

Maximum flow allowed

The maximum flow allowed refers to the maximum flow allowable in a conduit. Maximum flow criteria were not considered applicable to the Parafield catchment model, and the value was left at zero (not applicable).

Entry loss coefficient

Head loss coefficient for flows entering pipes. This value was assumed to be zero. Pipe friction was assumed to account for all losses in the stormwater distribution network.

Exit loss coefficient

Head loss coefficient for flows exiting pipes. This value was assumed to be zero. Pipe friction was assumed to account for all losses in the stormwater distribution network.

Flap gate

The flap gate function refers to the presence of a gate preventing backflow from downstream conduits. It is indicated with a 'Yes' or 'No' value. There were no flap gates assumed in the Parafield catchment.

Cross section

The cross section refers to the shape of the conduit. All drainage links known to be pipe conduits were assumed to be 'circular' (i.e. cylindrical) with a single dimension, the diameter.

The Parafield drain (the concrete open channel at the end of the catchment) was represented by a trapezoidal channel as shown in Figure A 3.

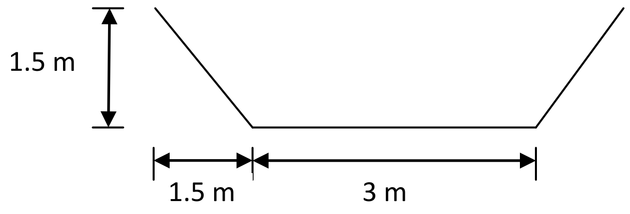


Figure A 3 – Geometry of the Parafield drain

Natural open drainage channels were identified by aerial photography and by descriptions in the stormwater GIS data from the SA Department for Water. These channels were assumed to be represented by a trapezoidal channel with the characteristics in Figure A 4.

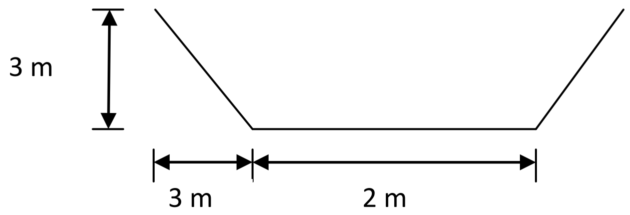


Figure A 4 – Assumed geometry of open drainage channels in the Parafield catchment (excluding the Parafield drain)

Geometry 1 (m) - Geometry 2 (m) - Geometry 3 (m) - Geometry 4 (m)

The geometry functions are used to designate the dimensions of standard shapes in PCSWMM. For circular (cylindrical) conduits, this was imported from the 'Dimension1' column in the stormwater pipes GIS model attribute table. Dimensions of the Parafield drain were derived from site measurement.

Barrels

Barrels refers to the number of parallel pipes of equal size, slope and roughness which are represented by the conduit in the SWMM model. In all cases, this was equal to 1 for the Parafield catchment.

Transect

The transect option allows the user to enter the name of a model transect. The transect model can thus be used to represent an irregular cross section as a conduit, for example a natural stream. There are no transects used in the Parafield model.

Shape curve

The shape curve refers to the name of a custom shape where the standard shapes do not apply to the conduit. Only standard shapes were adopted in the Parafield model.

Culvert code

The culvert code refers to a number representing inlet geometry if conduit is a culvert. This function was not used for the Parafield model.

Summary of Calibration Approach

To calibrate the model, all parameters were fixed except those parameters which are summarised in Table A 1. The nature of the parameter adjustments is also summarised based on the preceding sections of Appendix A.

Table A 1 – Annotation of parameter adjustment during the calibration process

Parameter	**Changes**
Sub-catchment Properties	
Width (m)	Determined using Rossman (2010) and adjusted
Imperviousness (%)	Determined from GIS mini-catchment data where possible; adjusted to increase runoff volume and peak flow
N-Imperv	Adopted based on Rossman (2010) and adjusted to simulate travel time according to observed storm flow data
N-Perv	Adopted based on Rossman (2010) and adjusted to simulate travel time according to observed storm flow data
D Store Imperv (mm)	Adopted based on Rossman (2010) and adjusted to produce flow according to observed storm flow data
D Store Perv (mm)	Adopted based on Rossman (2010) and adjusted to produce flow according to observed storm flow data
Zero Imperv	Little guidance on the selection of this value, but the best results were achieved when this was set to 50% (changing this value will lead to changes in other impervious area values)
Horton Infiltration Parameters	
Maximum infiltration rate (mm/hr)	Adopted based on Rossman (2010) and adjusted to simulate storm flow data for events where flow was in excess of 3 m^3/s
Minimum infiltration rate (mm/hr)	Adopted based on Rossman (2010) and adjusted to simulate storm flow data for events where flow was in excess of 3 m^3/s
Decay rate (1/hr)	Adopted based on Rossman (2010) and adjusted to simulate storm flow data for events where flow was in excess of 3 m^3/s
Drying time (days)	Adopted based on Rossman (2010) and adjusted to simulate storm flow data for events where flow was in excess of 3 m^3/s
Maximum volume (mm)	Adopted based on Rossman (2010) and adjusted to simulate storm flow data for events where flow was in excess of 3 m^3/s
Conduit properties	
Roughness	Adopted based on Rossman (2010) and adjusted to simulate travel time according to data in Section 3.3.1

Appendix B – Calibration Plots

The following plots display the fit of the modelled to the observed data for all 22 events selected for model calibration. Further details on each plot including the peak flow rate, total volume of the observed and estimated flow and several statistical assessments of the fit are provided in Section 3.4.3.

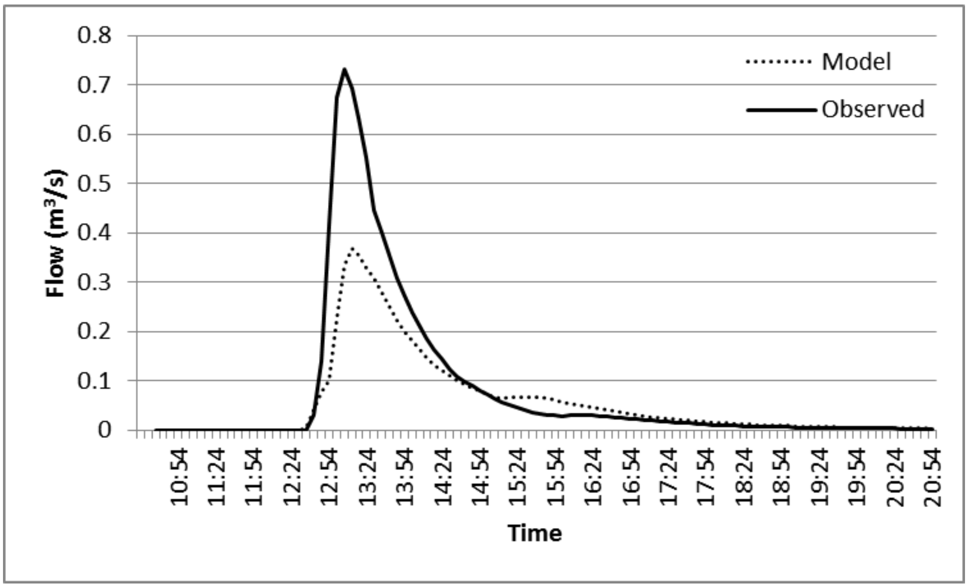


Figure C 1 – Comparison of observed and simulated flow for Event 1: 8 July 2003

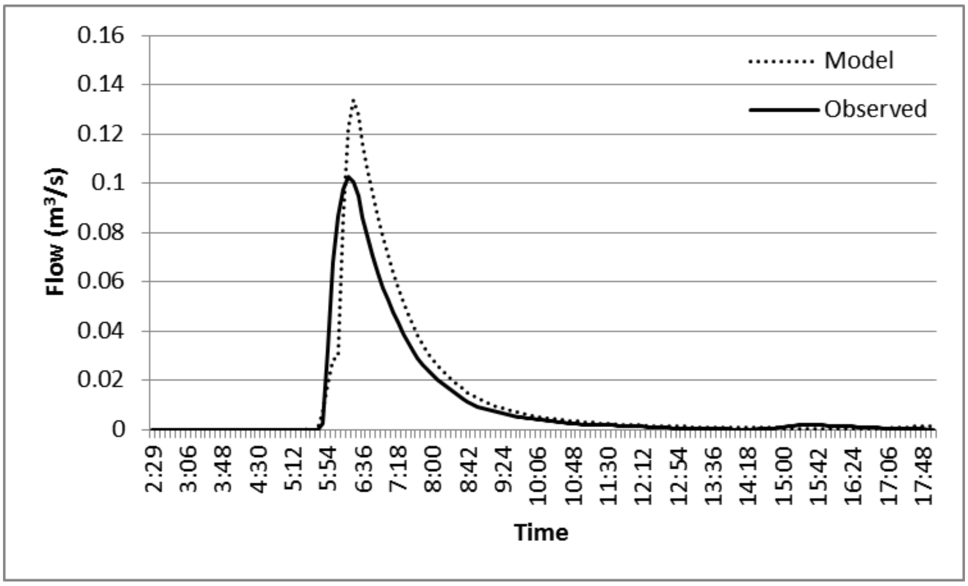


Figure C 2 – Comparison of observed and simulated flow for Event 2: 29 January 2004

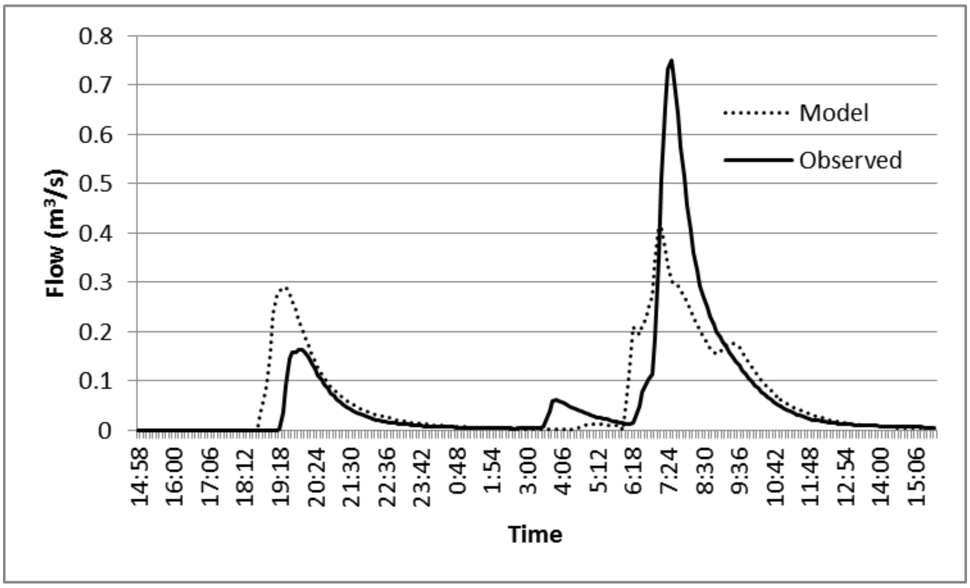


Figure C 3 – Comparison of observed and simulated flow for Event 3: 15 May 2004

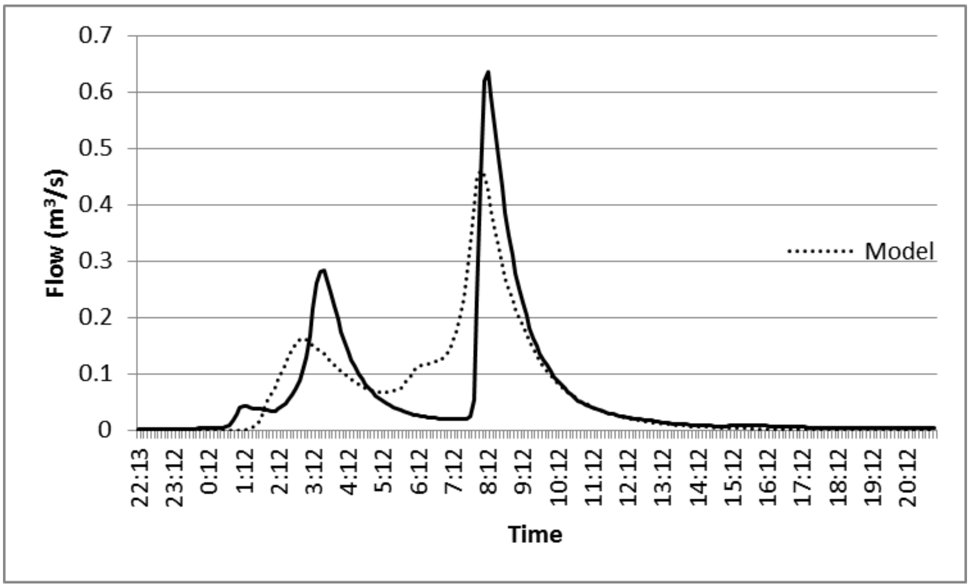


Figure C 4 – Comparison of observed and simulated flow for Event 4: 22 May 2004

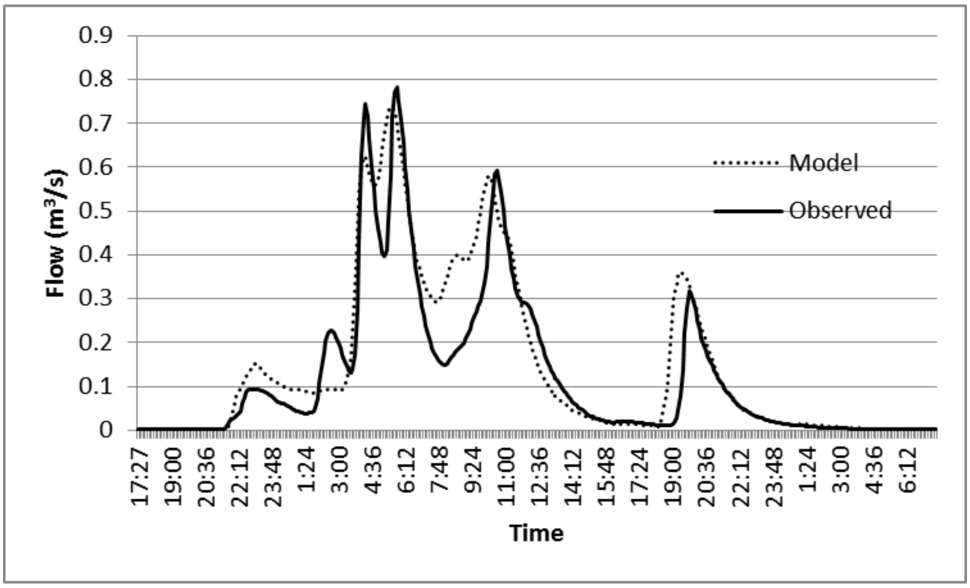


Figure C 5 – Comparison of observed and simulated flow for Event 5: 1 June 2004

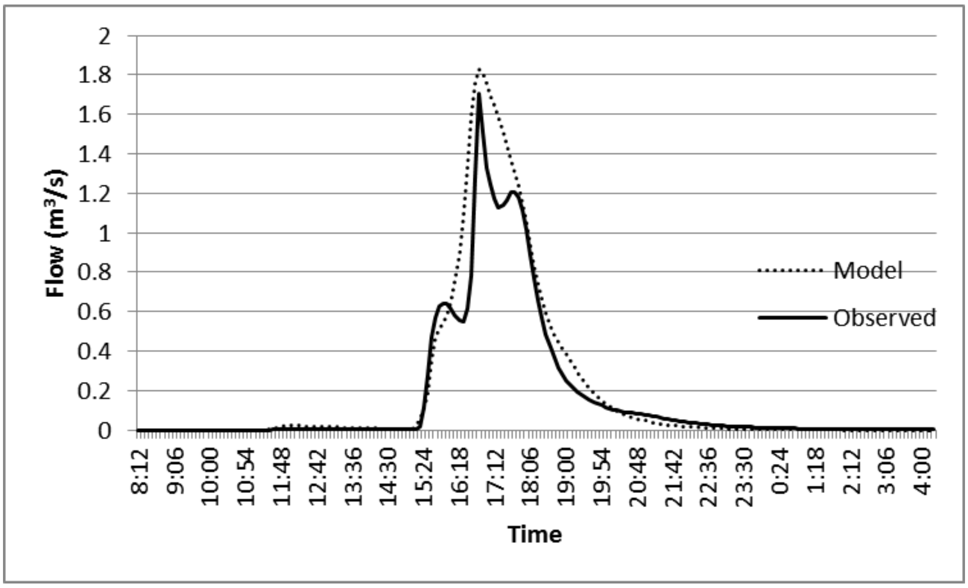


Figure C 6 – Comparison of observed and simulated flow for Event 6: 21 February 2004

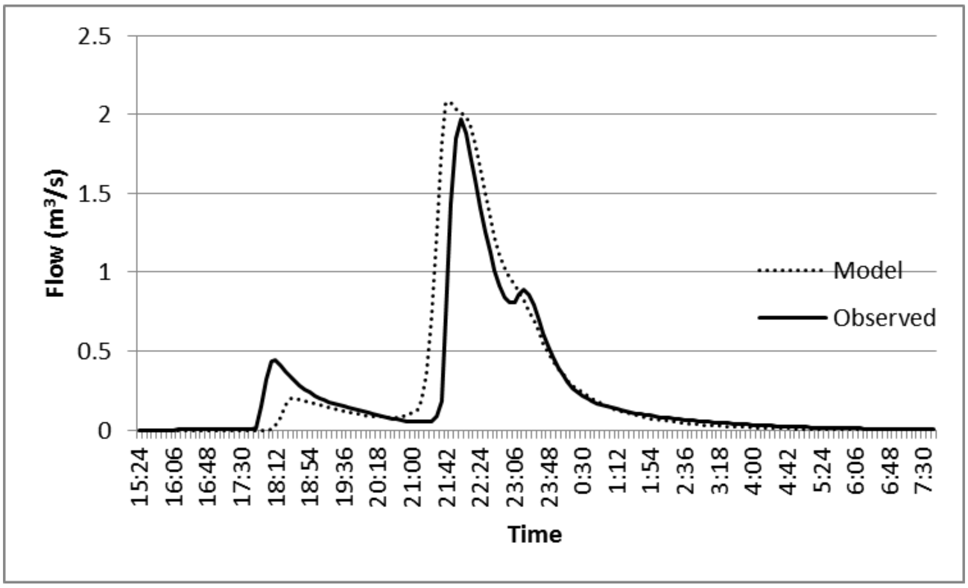


Figure C 7 – Comparison of observed and simulated flow for Event 7: 22 April 2004

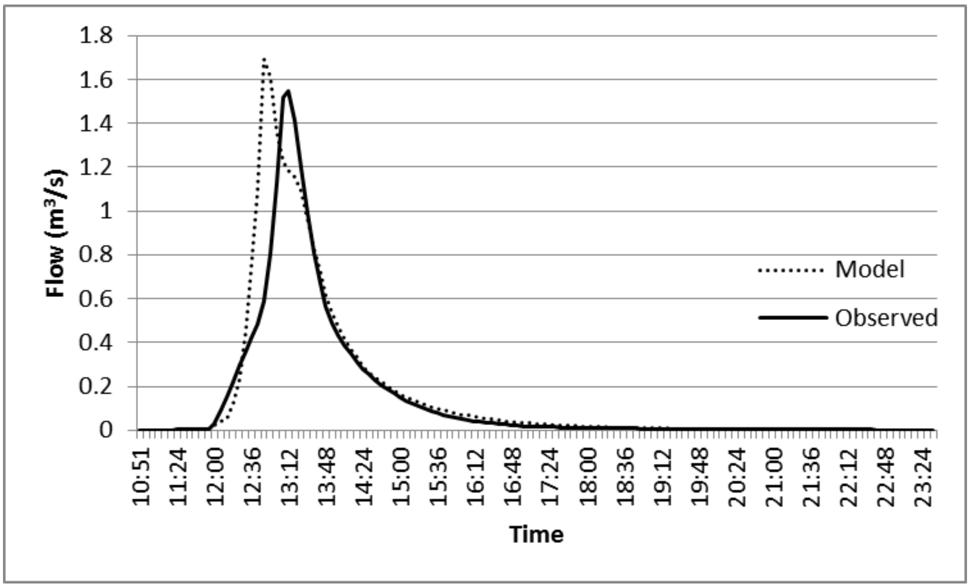


Figure C 8 – Comparison of observed and simulated flow for Event 8: 29 April 2004

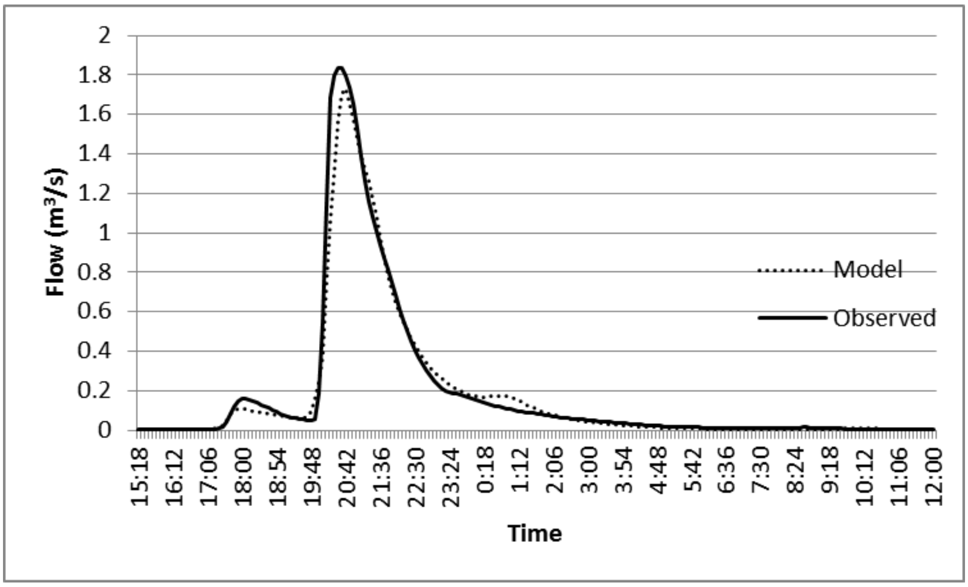


Figure C 9 – Comparison of observed and simulated flow for Event 9: 8 July 2004

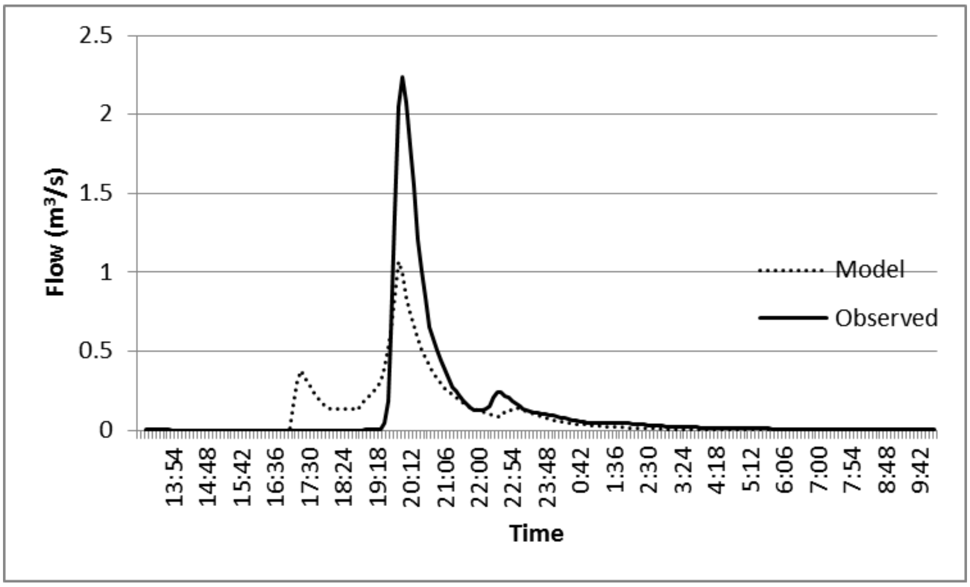


Figure C 10 – Comparison of observed and simulated flow for Event 10: 3 July 2003

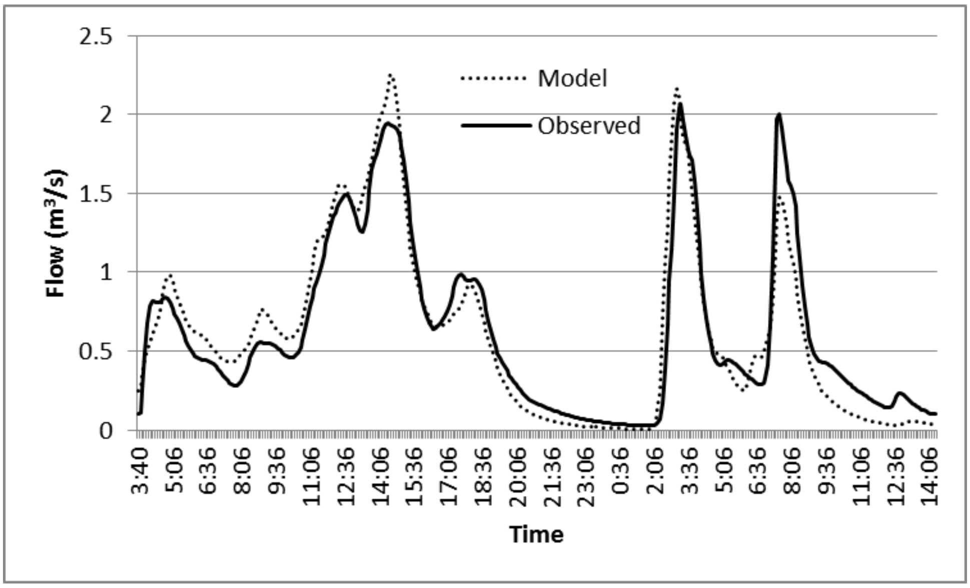


Figure C 11 – Comparison of observed and simulated flow for Event 11: 23 July 2004

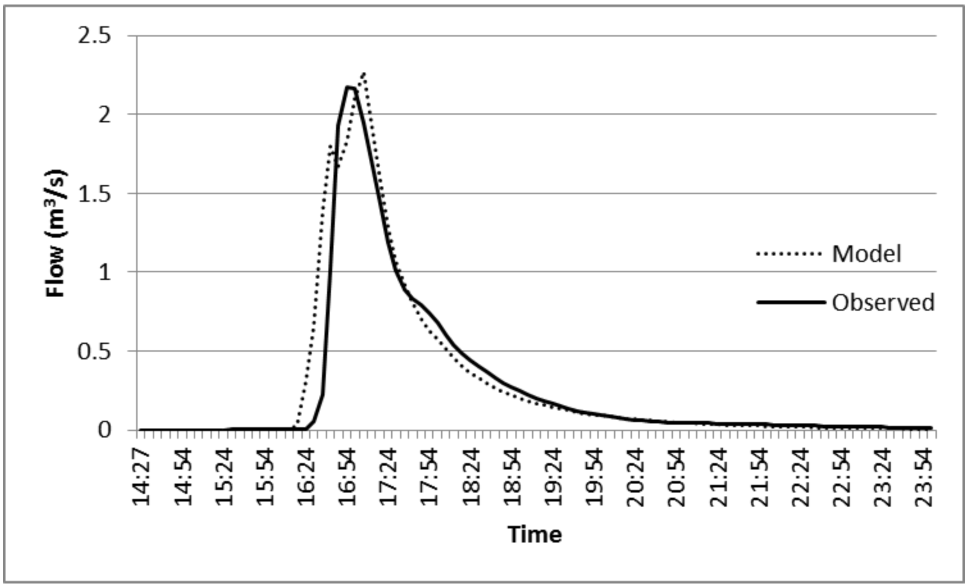


Figure C 12 – Comparison of observed and simulated flow for Event 12: 28 May 2004

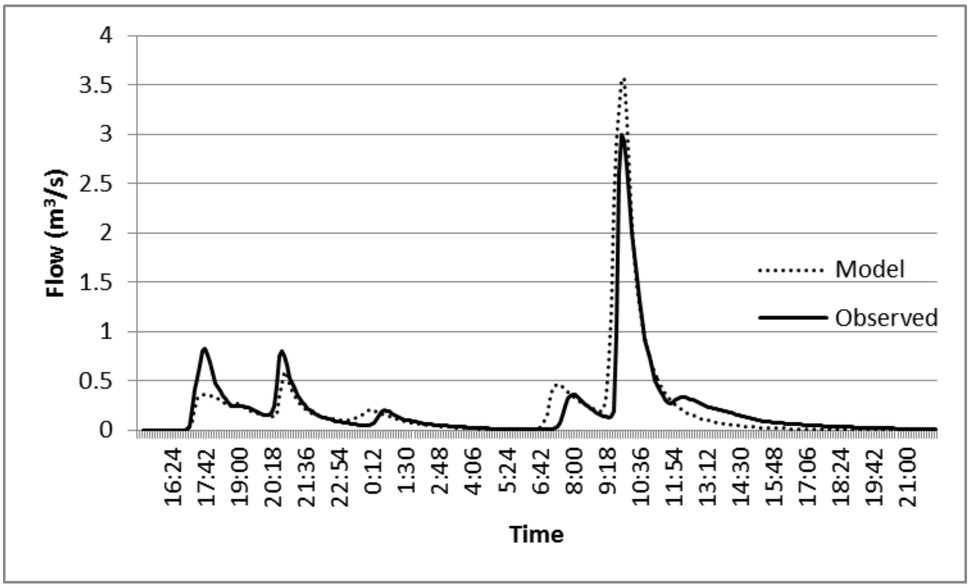


Figure C 13 – Comparison of observed and simulated flow for Event 13: 11 July 2003

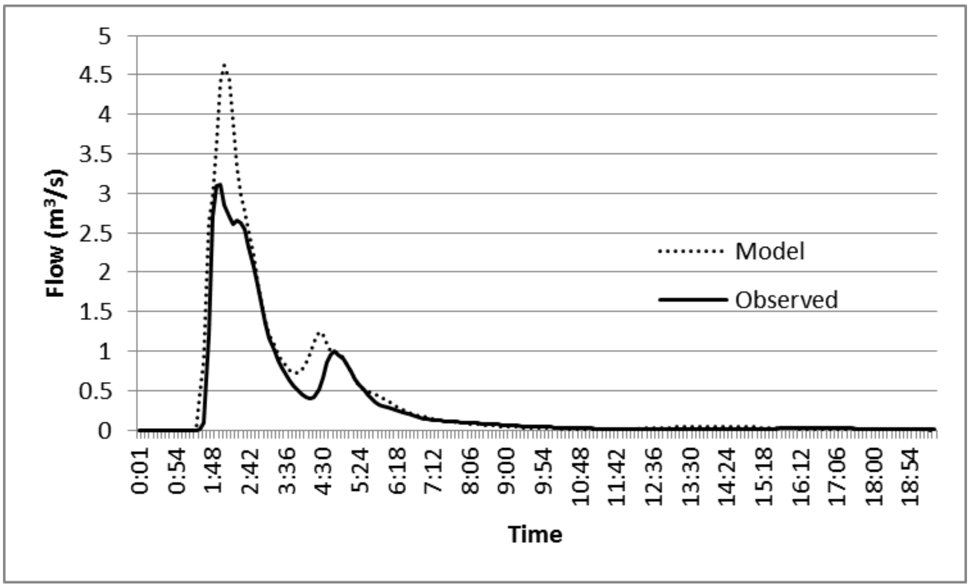


Figure C 14 – Comparison of observed and simulated flow for Event 14: 9 June 2004

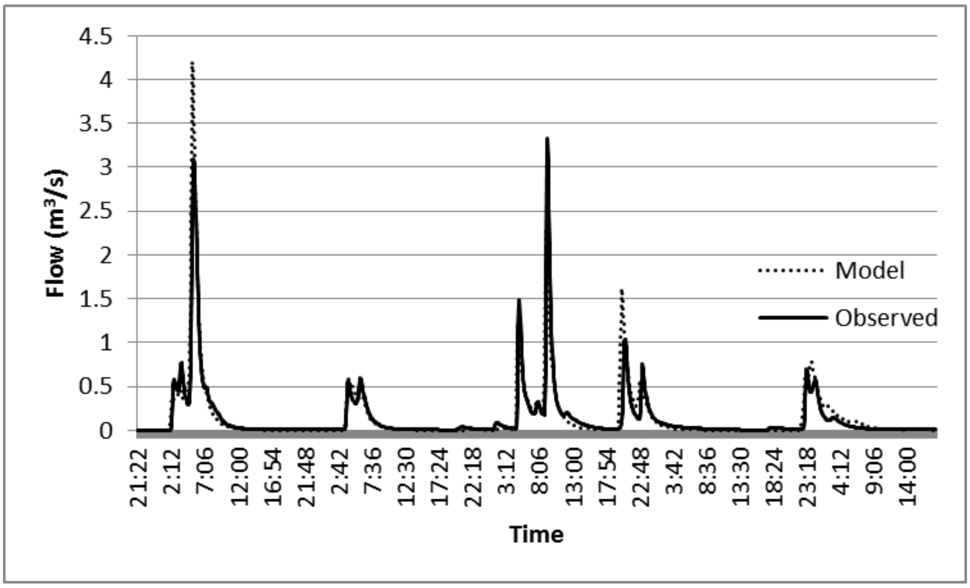


Figure C 15 – Comparison of observed and simulated flow for Event 15: 11 June 2004

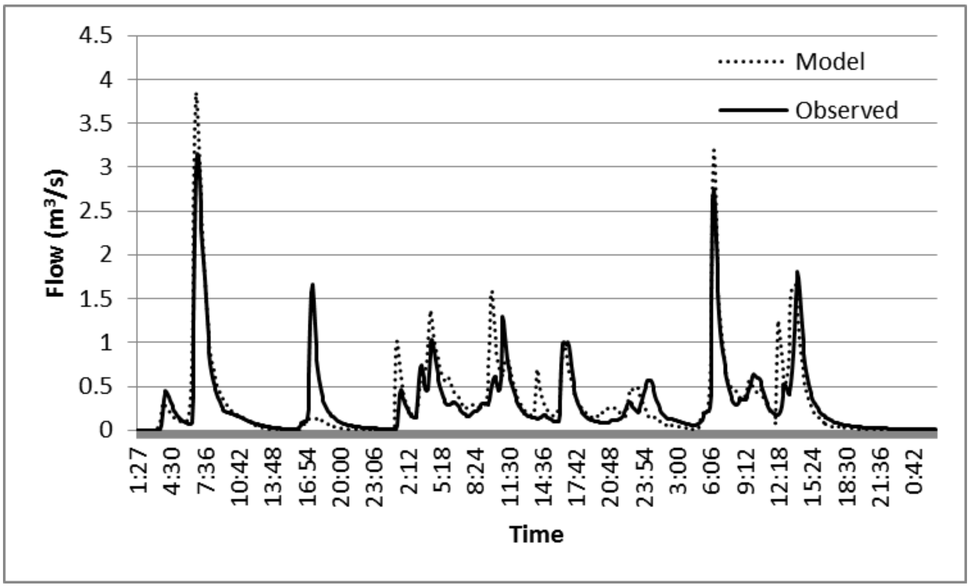


Figure C 16 – Comparison of observed and simulated flow for Event 16: 4 November 2004

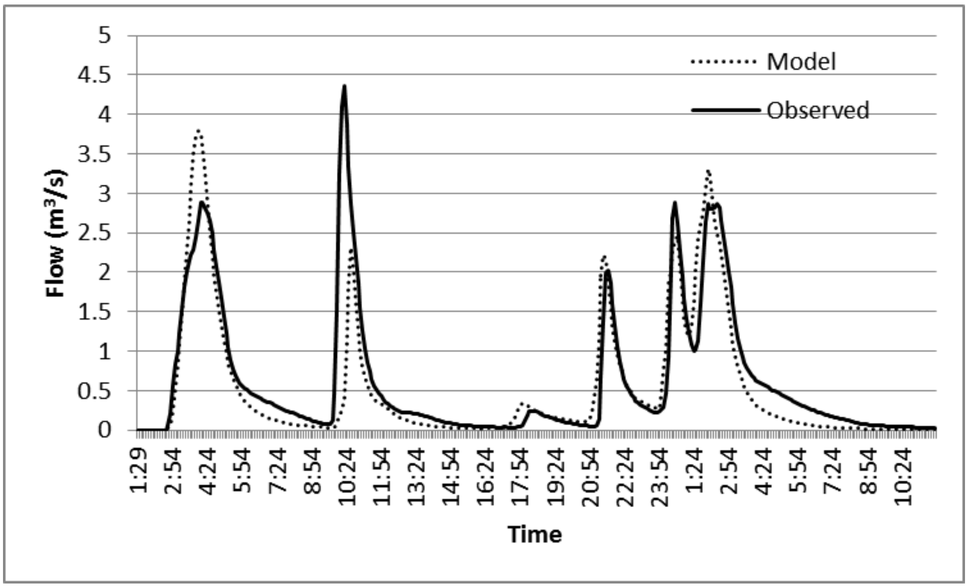


Figure C 17 – Comparison of observed and simulated flow for Event 17: 18 June 2004

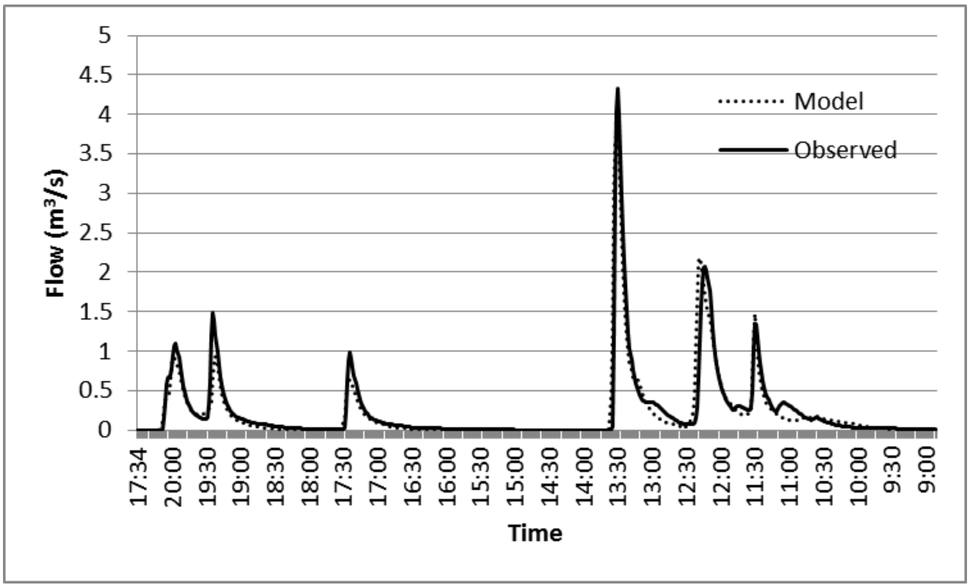


Figure C 18 – Comparison of observed and simulated flow for Event 18: 23 June 2004

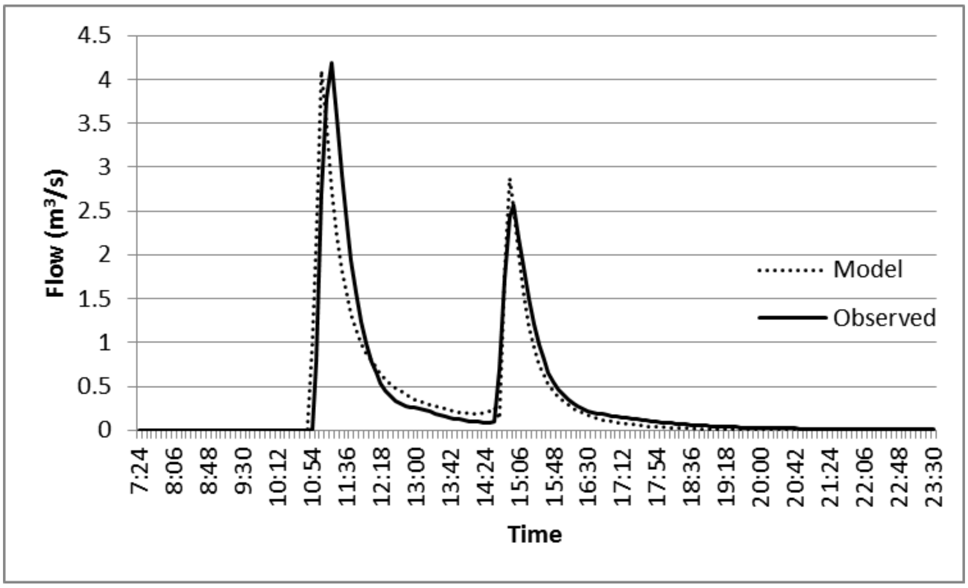


Figure C 19 – Comparison of observed and simulated flow for Event 19: 3 January 2005

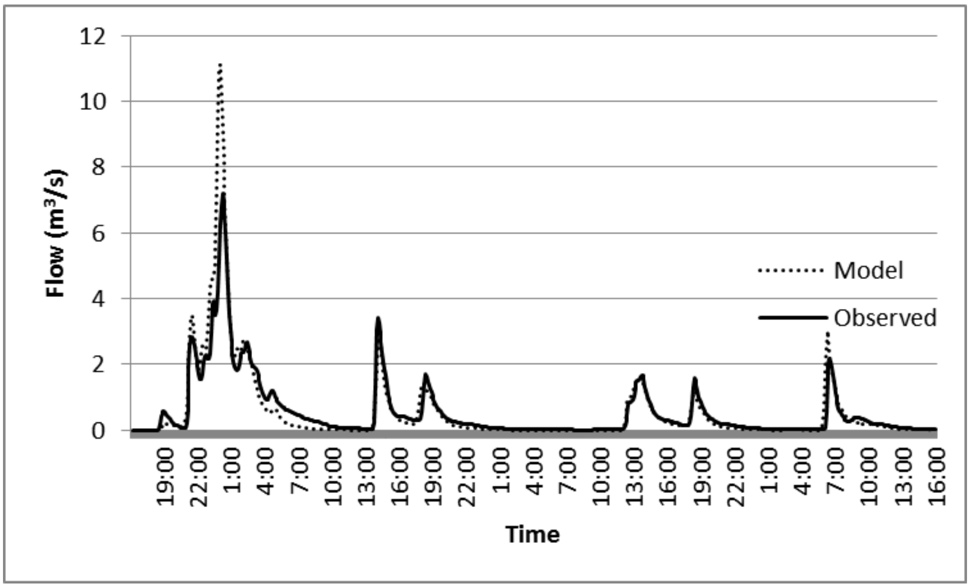


Figure C 20 – Comparison of observed and simulated flow for Event 20: 26 June 2003

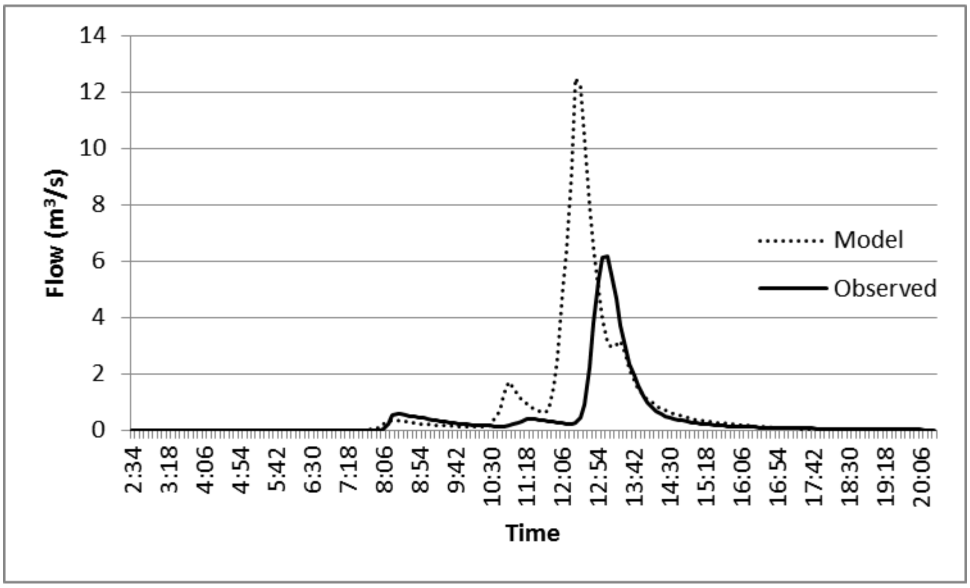


Figure C 21 – Comparison of observed and simulated flow for Event 21: 6 December 2004

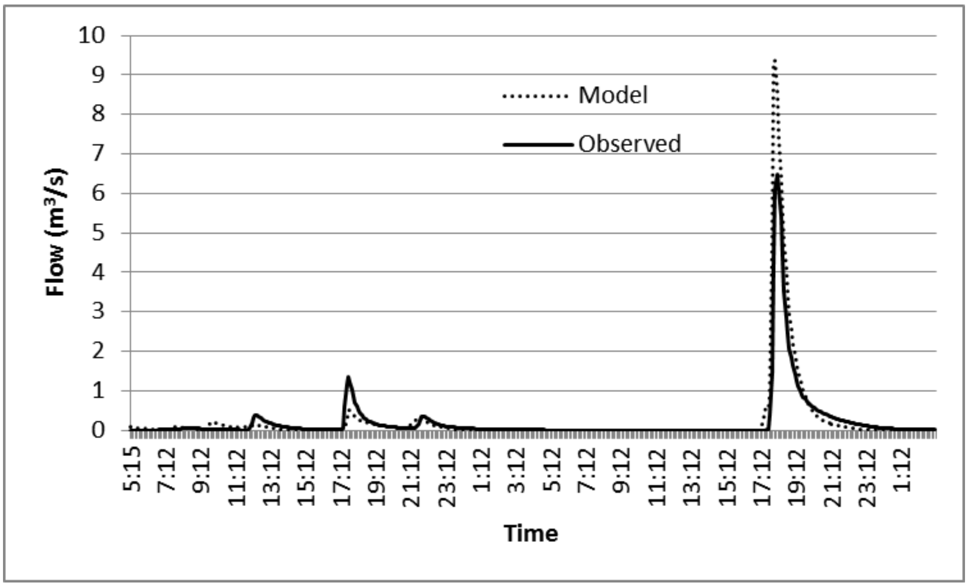


Figure C 22 – Comparison of observed and simulated flow for Event 22: 8 December 2004

Appendix C – Dry Weather Flow Spill Event Results

The statistics of dry weather spill simulations are presented in Tables C1 to C4. In each case, the tables indicate the site (based on the high risk locations identified in Section 4.1), the SWMM model pit ID, distance from catchment outlet, time of travel, duration of contaminant flow, time to peak flow and the corresponding quality and flow routing errors.

Table C 1 – Dry weather travel time statistics for a 20 kL spill with zero baseflow

Site	Drain/Pit (Model ID)	Distance (m)	Time of travel (mins)	Duration (hours)	Time to peak (mins)	Quality routing error (%)	Volume routing error (%)
Site 1	Pit-25331	5100	102	10.5	138	-36.5	-118.8
Site 2	Pit-25639	5180	102	10.5	138	-43.9	-119.2
Site 3	Pit-20061	4090	84	9	114	-29.2	-106.6
Site 4	Pit-25356	3390	78	8.7	108	-56.0	-100.8
Site 5	J6219	3380	84	8.4	138	-93.0	-118.2
Site 6	J16055	1630	24	4.8	54	-81.0	-81.0
Site 7	J9321	2508	60	7.4	108	-109.6	-109.6
Site 8	J6995	2100	24	4.7	54	-74.6	-83.2
Site 9	J23710	6660	234	16.7	300	-264.2	-264.2
Site 10	Pit-18621	2060	48	7.1	78	-75.0	-87.6
Site 11	Pit-18250	4280	78	8.6	108	-37.8	-92.6
Site 12	Pit-18251	3795	66	8.1	96	-59.0	-95.1
Site 13	Pit-24527	2480	42	7.3	72	-105.4	-105.4
Site 14	Pit-19692	2160	42	7.3	72	-32.7	-104.3
Site 15	Pit-20089	5865	108	10.6	144	-49.8	-120.7
Site 16	J6479	4360	138	10	198	-124.0	-124.0
Site 17	Pit-16017	4395	78	8.8	108	-34.0	-98.8
Site 18	Pit-23138	6800	114	10.5	150	-20.9	-87.5

Table C 2 - Dry weather travel time statistics for a 20 kL spill with a 5 L/s baseflow

Site	Drain/Pit (Model ID)	Distance (m)	Time of travel (mins)	Duration (hours)	Time to peak (mins)	Quality routing error (%)	Volume routing error (%)
Site 1	Pit-25331	5100	78	9.21	114	10.7	-0.6
Site 2	Pit-25639	5180	78	9.22	1114	10.4	-0.6
Site 3	Pit-20061	4090	60	8.68	96	9.9	-0.5
Site 4	Pit-25356	3390	55	8.61	90	10.4	-0.5
Site 5	J6219	3380	49	11.35	108	9.8	-0.6
Site 6	J16055	1630	24	9.09	48	9.6	-0.4
Site 7	J9321	2508	36	10.73	90	11.1	-0.6
Site 8	J6995	2100	12	9.06	108	9.3	-0.3
Site 9	J23710	6660	155	26.52	240	15.1	-1.5
Site 10	Pit-18621	2060	30	8.23	54	10.1	-0.5
Site 11	Pit-18250	4280	55	8.55	90	10.1	-0.5
Site 12	Pit-18251	3795	48	8.42	78	8.9	-0.4
Site 13	Pit-24527	2480	30	8.38	60	11.0	-0.5
Site 14	Pit-19692	2160	30	8.37	60	11.1	-0.5
Site 15	Pit-20089	5865	79	9.228	120	10.5	-0.6
Site 16	J6479	4360	76	12.65	138	9.8	-0.8
Site 17	Pit-16017	4395	60	8.58	84	9.3	-0.5
Site 18	Pit-23138	6800	84	9	114	16.6	-0.4

Table C 3 - Dry weather travel time statistics for a 40 kL spill with zero baseflow

Site	Drain/Pit (Model ID)	Distance (m)	Time of travel (mins)	Duration (hours)	Time to peak (mins)	Quality routing error (%)	Volume routing error (%)
Site 1	Pit-25331	5100	78	10.9	108	-98.5	-63.4
Site 2	Pit-25639	5180	78	10.9	108	-100.2	-37.5
Site 3	Pit-20061	4090	60	9.4	84	-87.9	-87.9
Site 4	Pit-25356	3390	60	9	78	-86.1	-67.6
Site 5	J6219	3380	44	11.51	96	-83.4	-103.5
Site 6	J16055	1630	18	4.9	42	-67.7	-67.7
Site 7	J9321	2508	48	7.6	78	-92.1	-92.1
Site 8	J6995	2100	24	4.7	54	-74.6	-83.2
Site 9	J23710	6660	262	18	204	-229.2	-229.3
Site 10	Pit-18621	2060	36	7.36	54	-56.4	-75.4
Site 11	Pit-18250	4280	60	8.9	84	-47.2	-64.8
Site 12	Pit-18251	3795	54	8.3	72	-50.7	-76.1
Site 13	Pit-24527	2480	36	7.5	54	-71.1	-85.8
Site 14	Pit-19692	2160	36	7.4	54	-84.7	-84.7
Site 15	Pit-20089	5865	84	11	102	-15.9	-96.2
Site 16	J6479	4360	102	10.7	144	-113.6	-113.6
Site 17	Pit-16017	4395	60	9.1	78	-85.4	-85.4
Site 18	Pit-23138	6800	84	11	114	-21.5	-80.0

Table C 4 - Dry weather travel time statistics for a 40 kL spill with a 5 L/s baseflow

Site	Model ID	Distance	Time of travel	Duration	Peak	Quality Routing Error	Volume Routing Error
Site 1	Pit-25331	5100	67	9.45	96	13.9	-0.7
Site 2	Pit-25639	5180	67	9.45	96	14.3	-0.7
Site 3	Pit-20061	4090	54	8.85	78	12.4	-0.6
Site 4	Pit-25356	3390	48	8.79	78	12.1	-0.6
Site 5	J6219	3380	44	11.51	90	10.9	-0.9
Site 6	J16055	1630	18	4.13	42	10.3	-0.5
Site 7	J9321	2508	31	10.89	72	11.6	-0.7
Site 8	J6995	2100	12	9.06	48	9.3	-0.3
Site 9	J23710	6660	127	27.38	186	12.3	-0.3
Site 10	Pit-18621	2060	30	8.23	54	10.1	-0.5
Site 11	Pit-18250	4280	48	8.71	78	18.0	-0.3
Site 12	Pit-18251	3795	42	8.58	72	12.4	-0.5
Site 13	Pit-24527	2480	30	8.46	54	11.3	-0.7
Site 14	Pit-19692	2160	27	8.49	54	10.9	-0.7
Site 15	Pit-20089	5865	72	9.46	102	13.3	-0.8
Site 16	J6479	4360	72	12.89	126	11.4	-0.9
Site 17	Pit-16017	4395	50	8.8	78	11.6	-0.5
Site 18	Pit-23138	6800	73	9.25	102	19.4	-0.4

Appendix D – Wet Weather Flow Characteristics

As described in Section 5.1, wet weather flow characteristics were examined to determine the key assumptions for a conservative analysis of wet weather travel time in the remainder of Section 5. The raw results generated from the wet weather flow characteristics analysis is presented in Tables D1 to D6.

Table D 1 - Characteristics of runoff and flow transport simulating a 5 minute, 20 kL spill at top of the catchment (Junction pit-23138)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	36	23	37	8	21	8	30
Time to peak (mins)[2]	48	54	48	60	30	36	30	54
Maximum flow (m^3/s^3)[3]	3.1	-	3.2	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	6259	-	5179	-	1111	-	685.8
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	1226	-	1049	-	143.1	-	244.3
Routing Error (%)[6]	-0.2	10.2	-0.2	14.2	-0.5	5.5	-0.5	4.8

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	51	54	51	37	30	36	30	24
Time to peak (mins)[2]	84	72	84	78	66	54	66	48
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	14.2	-	14.2	-
Maximum concentration (mg/L)[3]	-	10280	-	13310	-	2963	-	4749
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	2278	-	3360	-	890.3	-	1322
Routing Error (%)[6]	0.0	9.0	0.0	16.5	-0.1	4.6	-0.1	13.3

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Table D 2 - Characteristics of runoff and flow transport simulating a 30 minute, 20 kL spill at top of the catchment (Junction pit-23138)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	36	23	42	8	24	8	30
Time to peak (mins)[2]	48	60	48	96	30	54	36	102
Maximum flow (m^3/s)[3]	3.1	-	3.1	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	4055	-	5303	-	441	-	700.9
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	919	-	298.9	-	246.5	-	339.9
Routing Error (%)[6]	-0.2	2.2	-0.2	4.8	-0.5	0.3	-0.5	4.1

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	51	60	51	44	30	42	30	30
Time to peak (mins)[2]	84	78	84	90	66	54	66	64
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	14.2	-	14.2	-
Maximum concentration (mg/L)[3]	-	8390	-	13060	-	2062	-	3551
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	1892	-	3357	-	671.1	-	1138
Routing Error (%)[6]	0.0	-3.8	0.0	1.4	-0.1	-9.4	0.4	-0.1

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Table D 3 - Characteristics of runoff and flow transport simulating a 2 hour, 20 kL spill at top of the catchment (Junction pit-23138)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	40	23	43	8	24	8	32
Time to peak (mins)[2]	48	258	48	282	30	222	30	2246
Maximum flow (m^3/s)[3]	3.1	-	3.1	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	20310	-	27840	-	5328	-	8257
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	6553	-	10680	-	1200	-	2087
Routing Error (%)[6]	-0.2	2.5	-0.2	3.1	-0.5	4.0	-0.5	5.1

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	51	60	51	52	30	42	30	35
Time to peak (mins)[2]	84	84	540	84	66	60	66	168
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	14.1	-	14.2	-
Maximum concentration (mg/L)[3]	-	2107	-	5212	-	525.3	-	1438
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	804.3	-	2441	-	117.6	-	739.6
Routing Error (%)[6]	0.0	-1.4	0.0	0.8	-0.1	-5.0	-0.1	0.8

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Table D 4 - Characteristics of runoff and flow transport simulating a 5 minute, 20 kL spill at middle of the catchment (Junction J102)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	24	23	30	8	12	8	18
Time to peak (mins)[2]	48	42	48	42	30	24	30	30
Maximum flow (m^3/s)[3]	3.1	-	3.1	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	22290	-	15340	-	5398	-	2353
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	5092	-	3486	-	1298	-	726.3
Routing Error (%)[6]	-0.2	-9.9	-0.2	-4.7	-0.5	-12.8	-0.5	3.2

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	48	42	51	6	30	24	30	18
Time to peak (mins)[2]	84	60	84	36	66	42	66	34
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	14.2	-	14.2	-
Maximum concentration (mg/L)[3]	-	66260	-	25230	-	22870	-	7507
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	18590	-	6136	-	5342	-	2113
Routing Error (%)[6]	0.0	-4.3	0.0	1.9	-0.1	-12.0	-0.1	0.7

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Table D 5 - Characteristics of runoff and flow transport simulating a 30 minute, 20 kL spill at middle of the catchment (Junction J102)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	36	23	30	8	18	8	18
Time to peak (mins)[2]	48	42	48	48	30	24	30	30
Maximum flow (m^3/s)[3]	3.1	-	3.1	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	13260	-	8270	-	1660		849.4
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	3099	-	2039	-	661.3	-	104.5
Routing Error (%)[6]	-0.2	-9.7	-0.2	-3.8	-0.5	-3.5	-0.5	0.9

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	50	48	51	10	30	30	30	5
Time to peak (mins)[2]	84	66	84	48	66	48	66	36
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	14.2	-	14.2	-
Maximum concentration (mg/L)[3]	-	34280	-	16060		9151	-	3794
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	9052	-	5357	-	2758	-	1745
Routing Error (%)[6]	0.0	-7.9	0.0	0.3	-0.1	-13.8	-0.1	0.3

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Table D 6 – Characteristics of runoff and flow transport simulating a 2 hour, 20 kL spill at middle of the catchment (Junction J102)

Event	3 month, 0.5 hour storm, spill at start of event		3 month, 0.5 hour storm, spill at middle of event		10 year, 0.5 hour storm, spill at start of event		10 year, 0.5 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	23	30	23	30	8	18	8	18
Time to peak (mins)[2]	48	150	48	168	30	138	30	150
Maximum flow (m^3/s)[3]	3.1	-	3.1	-	14.3	-	14.3	-
Maximum concentration (mg/L)[3]	-	7198	-	9505	-	1517	-	2556
Mean flow (m^3/s)[4]	0.3	-	0.3	-	1.5	-	1.5	-
Mean concentration (mg/L)[5]	-	2232	-	2631	-	429.3	-	625.2
Routing Error (%)[6]	-0.2	-2.0	-0.2	0.2	-0.5	-0.4	-0.5	1.6

Event	3 month, 12 hour storm, spill at start of event		3 month, 12 hour storm, spill at middle of event		10 year, 12 hour storm, spill at start of event		10 year, 12 hour storm, spill at middle of event	
	Flow	Quality	Flow	Quality	Flow	Quality	Flow	Quality
Time of travel (mins)[1]	51	48	51	12	30	31	30	6
Time to peak (mins)[2]	84	72	84	138	66	48	66	132
Maximum flow (m^3/s)[3]	3.2	-	3.2	-	1415.0	-	14.2	-
Maximum concentration (mg/L)[3]	-	8049	-	4367	-	2807	-	1234
Mean flow (m^3/s)[4]	0.6	-	0.6	-	3.3	-	3.3	-
Mean concentration (mg/L)[5]	-	1670	-	2763	-	381.8	-	726.2
Routing Error (%)[6]	0.0	-4.7	0.0	0.2	-0.1	-6.0	-0.1	0.2

[1] Time of travel – for 'Flow' refers to the time between the beginning of storm to the beginning of flow at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the pollutant at the catchment outlet

[2] Time to peak – for 'Flow' refers to the time between the beginning of storm to the observance of the peak flow rate at the end of catchment; for 'Quality' refers to the time between the occurrence of the spill and the appearance of the peak concentration of the pollutant at the catchment outlet

[3] Maximum value of 'Flow' and 'Quality' is the simulated peak flow and pollutant concentration, respectively

[4] Mean flow is the simulated mean flow at the point of harvest based on the period over which flow is observed above 0.001 m^3/s

[5] Mean concentration is the simulated mean concentration of pollutant at the point of harvest based on the period over which pollutant is observed at a concentration above 1 mg/L

[6] Routing error for 'Flow' and 'Quality' is based on the error in routing flow quantity and pollutant mass, respectively, through the catchment

Appendix E - Wet Weather Flow Spill Event Results

The statistics of wet weather spill simulations are presented in Tables C1 to C4. In each case, the tables indicate the site (based on the high risk locations identified in Section 4.1), the SWMM model pit ID, distance from catchment outlet, time of travel, duration of contaminant flow, time to peak flow and the corresponding quality and flow routing errors.

Table E 1– Results for the 3 month, 30 minute storm, 20 kL spill at the beginning of the storm

			Quality							Flow			
Site	Drain/Pit (Model ID)	Distance (m)	Time of travel (mins)	Time to peak (mins)	Max concentration (mg/L)[1]	Mean concentration (mg/L)[2]	Total time (hrs)[3]	Quality routing error (%)	Volume routing error (%)	Total time (hrs)[4]	Max flow (m^3/s)	Mean flow (m^3/s)	Total volume (m^3)
Site 1	Pit-25331	5100	36	54	6882	1446	1.4	1.9	1.4	9.6	3.1	0.27	9408
Site 2	Pit-25639	5180	36	54	6806	1439	1.5	2.1	1.5	9.6	3.1	0.27	9394
Site 3	Pit-20061	4090	36	48	10590	2323	0.9	-3.1	1.4	9.6	3.1	0.27	9411
Site 4	Pit-25356	3390	36	48	10480	2436	0.9	-0.7	1.4	9.6	3.1	0.27	9409
Site 5	J6219	3380	37	66	2741	720	1.7	-21.0	1.4	14.3	0.6	0.08	4286
Site 6	J16055	1630	18	36	59390	14510	1.5	5.4	1.4	14.3	0.6	0.08	4305
Site 7	J9321	2508	27	60	29370	6614	1.9	-10.1	1.4	14.3	0.6	0.08	4305
Site 8	J6995	2100	18	42	56110	13840	1.5	2.0	1.4	14.3	0.6	0.08	4305
Site 9	J23710	6660	62	150	14240	5457	28.4	-30.5	1.3	14.3	0.6	0.08	4324
Site 10	Pit-18621	2060	27	42	20150	4047	0.7	-5.8	1.4	9.6	3.0	0.27	9407
Site 11	Pit-18250	4280	31	48	10210	1409	1.4	-0.4	1.4	9.6	3.0	0.27	9412
Site 12	Pit-18251	3795	30	48	10990	2837	0.8	-3.7	1.4	9.6	3.0	0.27	9397
Site 13	Pit-24527	2480	24	42	17130	1382	2.6	0.1	1.4	9.6	3.0	0.27	9407
Site 14	Pit-19692	2160	24	42	17060	1412	2.6	0.4	1.4	9.6	3.0	0.27	9404
Site 15	Pit-20089	5865	36	54	6422	1438	1.5	1.5	1.4	9.6	3.1	0.27	9394
Site 16	J6479	4360	58	126	1841	307	9.9	-55.8	1.4	14.3	0.6	0.08	4287
Site 17	Pit-16017	4395	32	48	10870	2302	0.9	-2.4	1.4	9.6	3.0	0.27	9416
Site 18	Pit-23138	6800	42	66	582.6	162	1.9	5.7	1.4	9.6	3.0	0.27	9387

[1] Maximum concentration at point of harvest based on PCSWMM simulation
[2] Mean concentration at point of harvest based on PCSWMM simulation over total time
[3] Total time of pollutant is the total time over which the pollutant is observed at the outlet at a concentration above 1 mg/L
[4] Total time of flow is the total time over which flow is observed at the point of harvest above 0.001 m^3/s

Table E 2 - Results for the 10 year, 30 minute storm, 20 kL spill at the beginning of the storm

			Quality							Flow			
Site	**Drain/Pit (Model ID)**	**Distance (m)**	**Time of travel (mins)**	**Time to peak (mins)**	**Max concentration (mg/L)[1]**	**Mean concentration (mg/L)[2]**	**Total time (hrs)[3]**	**Quality routing error (%)**	**Flow routing error (%)**	**Total time (hrs)[4]**	**Max flow (m^3/s)**	**Mean flow (m^3/s)**	**Total volume (m^3/s)**
Site 1	Pit-25331	5100	24	42	297	111	0.7	-10.6	1.4	10.8	14.3	1.53	59690
Site 2	Pit-25639	5180	24	42	282	104	0.7	-10.6	1.4	10.8	14.3	1.53	59670
Site 3	Pit-20061	4090	18	36	375	144	0.6	-6.4	1.4	10.8	14.3	1.53	59680
Site 4	Pit-25356	3390	18	36	590	172	0.6	1.7	1.4	10.8	14.3	1.53	59680
Site 5	J6219	3380	21	36	342	106	0.6	-4.1	1.4	16.4	9.3	0.64	37410
Site 6	J16055	1630	12	24	9694	2637	0.6	-2.6	1.4	16.4	9.3	0.64	37420
Site 7	J9321	2508	18	30	4041	1148	0.8	-16.9	1.4	16.4	9.3	0.64	37420
Site 8	J6995	2100	12	24	6625	1681	0.7	-2.8	1.4	16.4	9.4	0.64	37430
Site 9	J23710	6660	36	78	1399	125	26.2	-36.1	1.4	16.4	9.3	0.64	37410
Site 10	Pit-18621	2060	18	24	1835	853	0.4	-1.0	1.4	10.8	14.3	1.53	59710
Site 11	Pit-18250	4280	18	36	998	343	0.7	2.1	1.4	10.8	14.3	1.53	59680
Site 12	Pit-18251	3795	18	30	2278	603	0.5	0.2	1.4	10.8	14.3	1.53	59660
Site 13	Pit-24527	2480	12	24	2533	489	0.9	5.8	1.4	10.8	14.3	1.53	59690
Site 14	Pit-19692	2160	12	24	2614	491	0.9	6.7	1.4	10.8	14.3	1.53	59710
Site 15	Pit-20089	5865	24	42	240	83	0.7	-10.8	1.4	10.8	14.3	1.533	59670
Site 16	J6479	4360	43	78	216	89	3.0	-55.4	1.4	16.4	9.348	0.6354	37410
Site 17	Pit-16017	4395	18	30	1669	498	0.6	2.5	1.4	10.8	14.3	1.533	59670
Site 18	Pit-23138	6800	31	54	72	29	2.0	2.4	1.4	10.8	14.3	1.534	59690

[1] Maximum concentration at point of harvest based on PCSWMM simulation
[2] Mean concentration at point of harvest based on PCSWMM simulation over total time
[3] Total time of pollutant is the total time over which the pollutant is observed at the outlet at a concentration above 1 mg/L
[4] Total time of flow is the total time over which flow is observed at the point of harvest above 0.001 m^3/s

Appendix F – Assessment of Model Capability

Table F-1 is adapted from CWMR and RCA (2010) with the addition of data pertaining to the PCSWMM model.

Table F 1 - Comparison of the capabilities of various modelling packages (adapted from CWMR and RCA, 2010)

Criteria	WaterCress	EPA SWMM	MUSIC	DRAINS	StormNET	MIKE Urban	PCSWMM
Rainfall and Surface Runoff							
Use sub-daily rainfall data	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Use long term rainfall data (e.g. 100 years)	Yes	Yes	Yes	No	Yes	Yes	Yes
Use different rainfall data for different location in a catchment	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Use input from multiple gauging stations, for using flow data as an input	Yes	Yes	Yes	No	Yes	Yes	Yes
Manage pervious and impervious catchment components	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Manage direct and indirect connections within a catchment	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Account for soil moisture content	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Account for varying soil moisture content, based on climate data input	Yes	Yes	Yes	No	Yes	Yes	Yes
Represent long term average flows	Yes	Yes	Yes	No	Yes	Yes	Yes
Represent short term average flows (e.g. sub annual)	Yes	Yes	Yes	No	Yes	Yes	Yes
Conduct flood analysis	Limited	Yes	limited	Yes	Yes	Yes	Yes
Produce runoff hydrographs	Limited[a]	Yes	limited	Yes	Yes	Yes	Yes
Predict flow peaks	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Predict flow duration	Limited[a]	Yes	Yes	Yes	Yes	Yes	Yes
Account for soil infiltration	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Account for biofiltration	No	No	Yes	No	No	No	No
Losses							
Account for evaporation losses	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Account for spatial variation in evaporation	Yes	Yes	No	No	Yes	Yes	Yes
Account for evaporation patterns over time	Yes	Yes	Yes	No	Yes	Yes	Yes

Criteria	WaterCress	EPA SWMM	MUSIC	DRAINS	StormNET	MIKE Urban	PCSWMM
Account for seepage losses in storages	Yes	Yes	No	Yes	Yes	Yes	Yes
Account for seepage losses in channels	Yes	No	No	Yes	No	No	No
Supply and Demand and Transfer of Flows							
Produce inflow and outflow hydrographs for storages	Limited[a]	Yes	Yes	Yes	Yes	Yes	Yes
Predict peak storage levels	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Predict times at which peaks will occur	Limited[a]	Yes	Limited	Yes	Yes	Yes	Yes
Accommodate multiple inputs and outputs to/from storages	Yes	Yes	Yes	No	Yes	Yes	Yes
Adjust supply to maintain storage levels	Yes	Yes	No	No	Yes	Yes	Yes
Adjust supply based on water quality	Yes	No	No	No	No	No	No
Model open channel hydraulics	No	Yes	No	Yes	Yes	Yes	Yes
Produce hydrographs for channels	No	Yes	Yes	Yes	Yes	Yes	Yes
Predict peak flows in channels	No	Yes	Yes	Yes	Yes	Yes	Yes
Account for water quality changes in channels	No	No	Yes	No	No	No	No
Allow for variation in off-take from wetlands/storage	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Hold water in wetlands based on water quality	Yes	No	No	No	No	No	No
Maintain wetland/storage water levels in a specified range	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Incorporate ASR systems into modelling	Yes	Yes	No	No	Yes	Yes	Yes
Vary ASR off-take and recovery based on water quality	Yes	No	No	No	No	No	No
Account for mixing and dispersion in groundwater	Yes	No	No	No	No	No	No
Input annual demand cycles	Yes	No	No	No	No	No	No
Input monthly demand cycles	Yes	limited	Yes	No	limited	limited	limited
Input and use daily demand cycles	Yes	No	No	No	No	No	No
Specify demand from various sources (e.g. agriculture, environmental flows)	Yes	Yes	No	No	Yes	Yes	Yes

Criteria	WaterCress	EPA SWMM	MUSIC	DRAINS	StormNET	MIKE Urban	PCSWMM
Identify failure to meet demand	Yes	No	No	No	No	No	No
Account for hydraulics of diversion structures (e.g. weirs)	Limited	Yes	Limited	no	Yes	Yes	Yes
Model long term overflows from storages (e.g. 50 years)	Yes	Yes	No	No	Yes	Yes	Yes
Water Quality							
Account for salinity in runoff and storages	Yes	Limited	Yes	No	Limited	Limited	Limited
Account for suspended solids	Limited	Limited	Yes	No	Limited	Limited	Limited
Account for colour	No	Limited	No	No	Limited	Limited	Limited
Account for nutrients (phosphorus, nitrogen)	No	Limited	Yes	No	Limited	Limited	Limited
Limit supply based on water quality	Yes	No	No	No	No	No	No
Account for water quality change in storages (mixing, vegetation uptake, etc.)	Limited	Yes	Yes	No	Yes	Yes	Yes
Use groundwater as storage (ASR)	Yes	Yes	No	No	Yes	Yes	Yes
Accommodate groundwater quality changes (e.g. mixing with natural groundwater)	Yes	No	No	No	No	No	No
Statistical Analysis							
Frequency analysis	Yes	No	Yes	No	No	No	Yes
Exceedance	Yes	No	Yes	No	No	No	Yes
Long term averages	Yes	No	Yes	No	No	No	Yes
Interaction and data formats							
Import from readily available sources (e.g. BOM)	No	No	Yes	No	No	No	No
Import recorded data for calibration	Yes	Yes	Yes	No	Yes	Yes	Yes
Display results as graphs	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Display results as tables	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Export results to other formats (e.g. spreadsheet)	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Costing							
Ability to estimate establishment costs	Limited	No	Yes	No	No	No	No
Ability to estimate operating costs	Limited	No	Yes	No	No	No	No
Other Considerations							

Criteria	WaterCress	EPA SWMM	MUSIC	DRAINS	StormNET	MIKE Urban	PCSWMM
Able to use multiple schemes in one catchment	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Model schemes in different configurations (e.g. series, parallel)	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Model over various scales (residential allotment or entire catchment)	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Provides comprehensive documentation (e.g. manual)	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Includes comprehensive help system	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Training available	Yes	No	Yes	Yes	No	Yes	No
Technical support available	Yes	Forum	Yes	Yes	Yes	Yes	Yes
Purchase cost	Free	Free	$330	$4000	$4995	≈$10,000	≈$10,000

Appendix G – Effects of Dilution

The following plots display the effect of dilution as a pollutant spill is routed through the Parafield catchments in a wet weather event. In each case, it is assumed that a spill occurs at the middle of the 10 year, 30 minute storm event i.e. the same event as that presented in Section 5.3. The findings form these charts are discussed in Section 5.4. Due to limited data on the nature of the stormwater distribution network in the Cobbler Creek catchment, the stormwater network was simplified in this region and the results are not reported for this catchment.

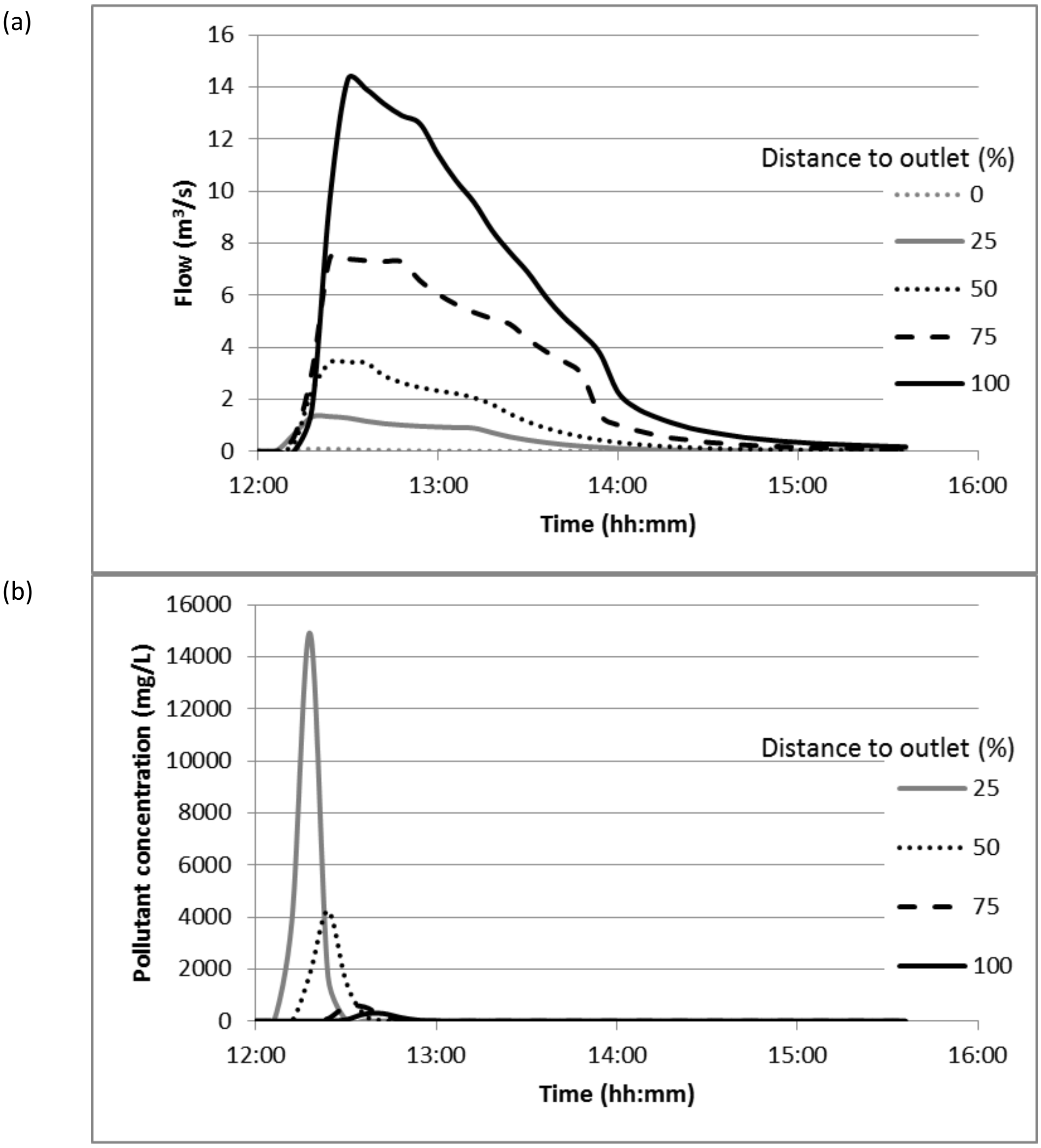
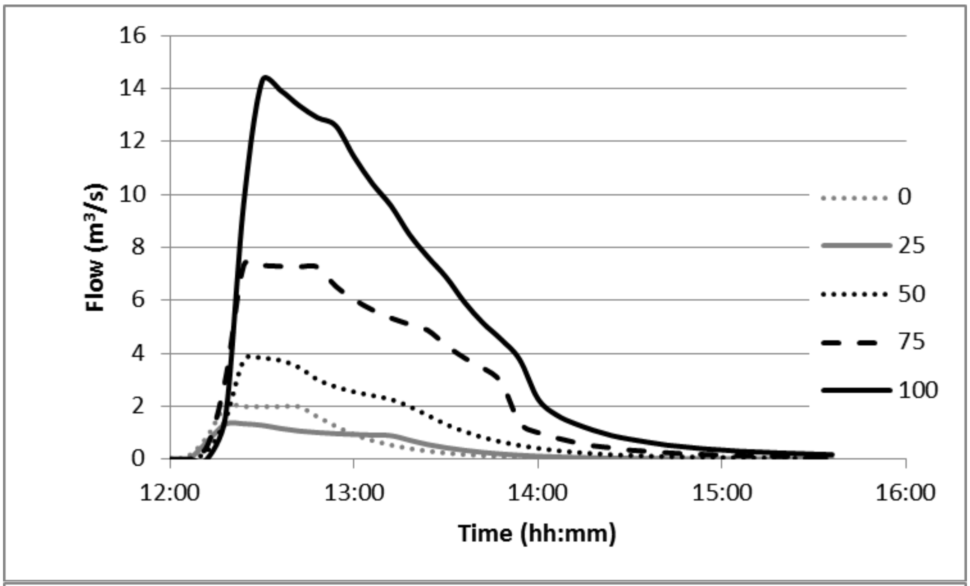


Figure G 1 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 1 (Pit-25331) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 5100 m.

(a)



(b)

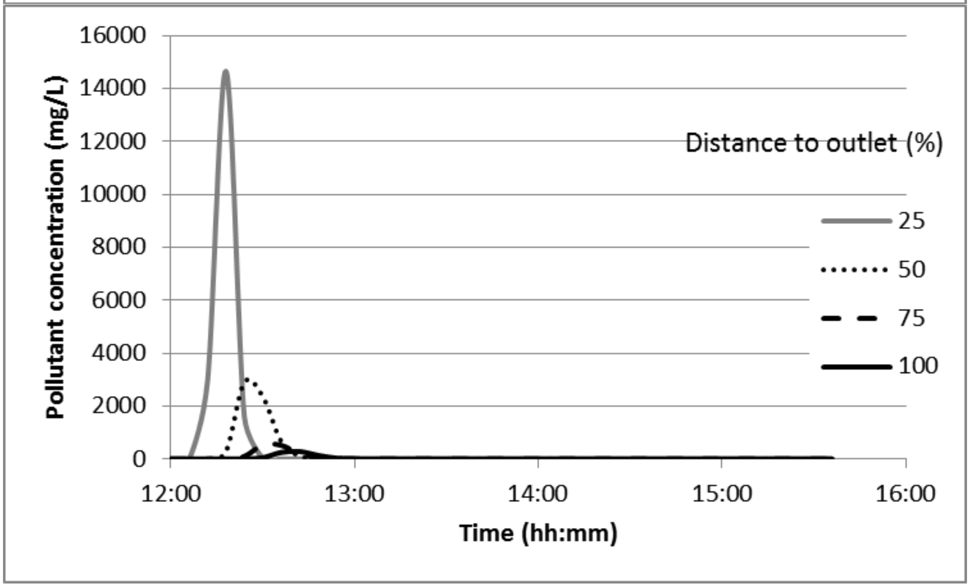
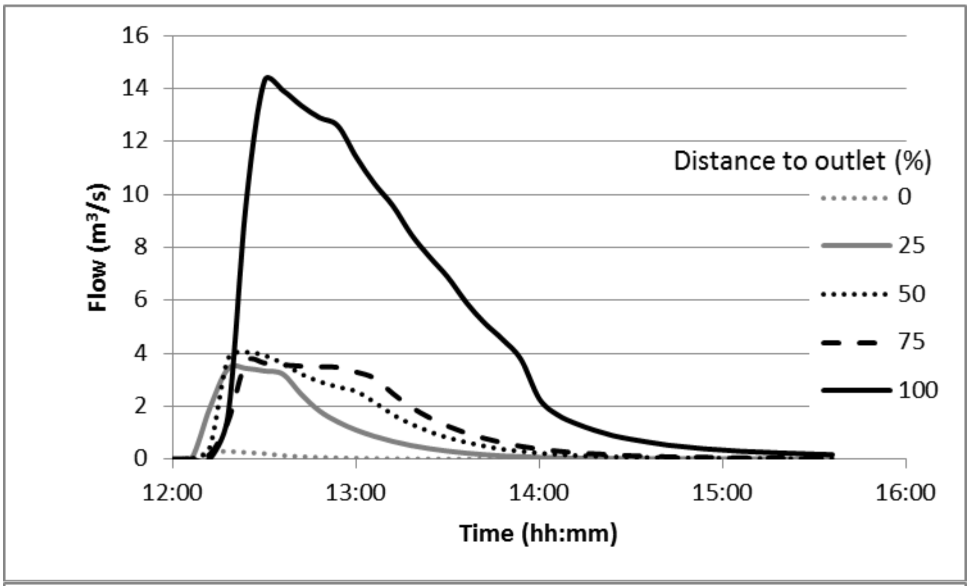


Figure G 2 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 2 (Pit-25639) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 5180 m.

(a)



(b)

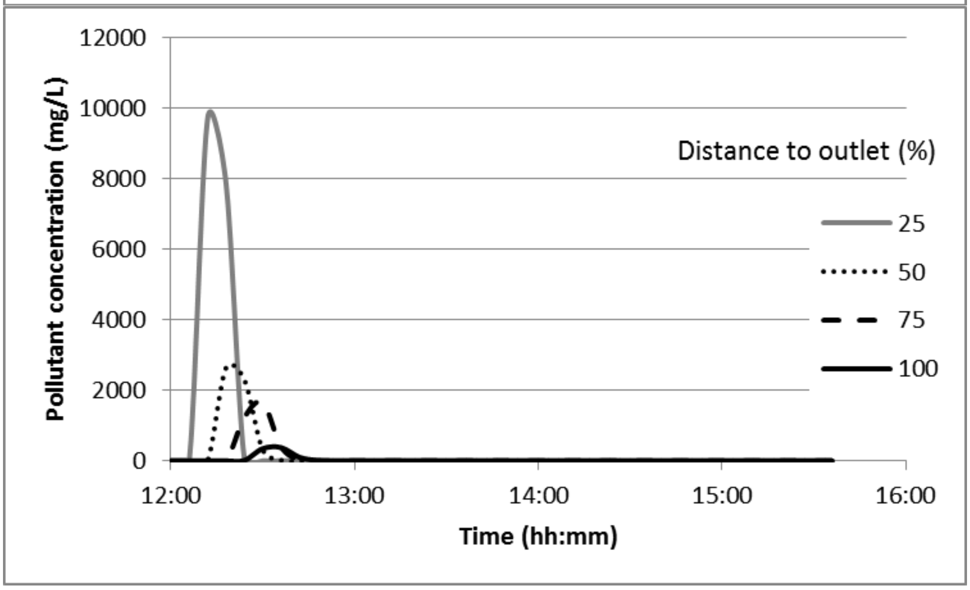


Figure G 3 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 3 (Pit-20061) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 4090 m.

(a)

(b)

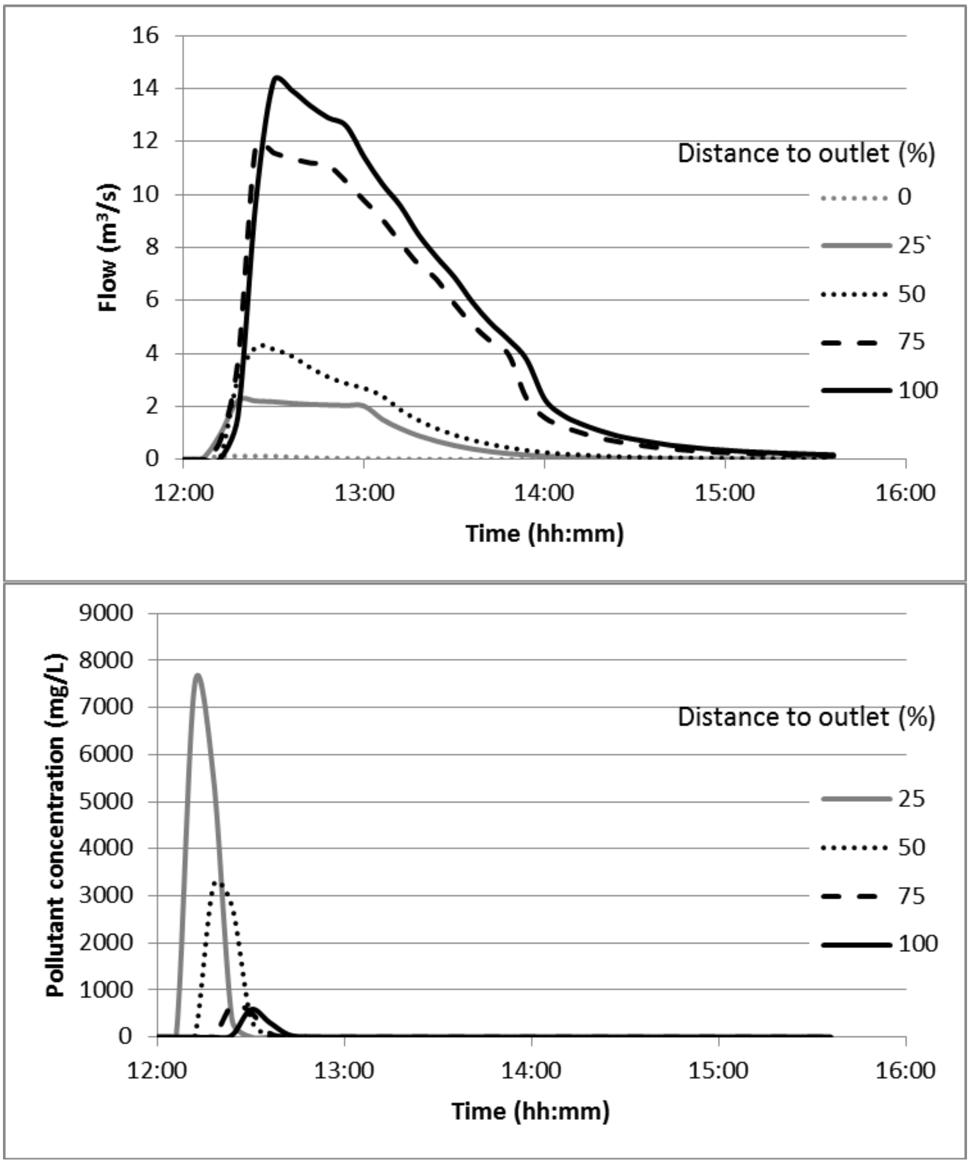


Figure G 4 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 4 (Pit-25356) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 3390 m.

(a)

(b)

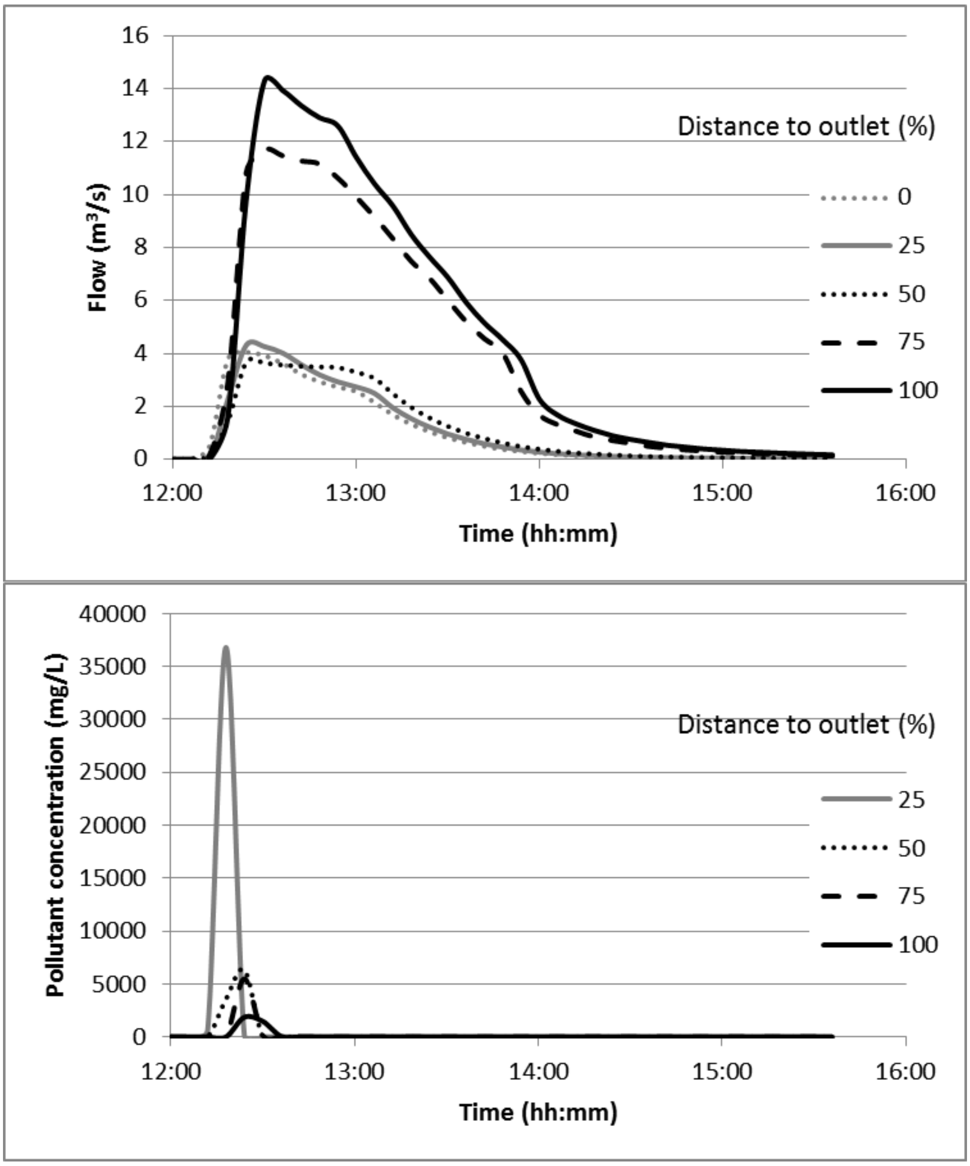


Figure G 5 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 10 (Pit-18621) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 2060 m.

(a)

(b)

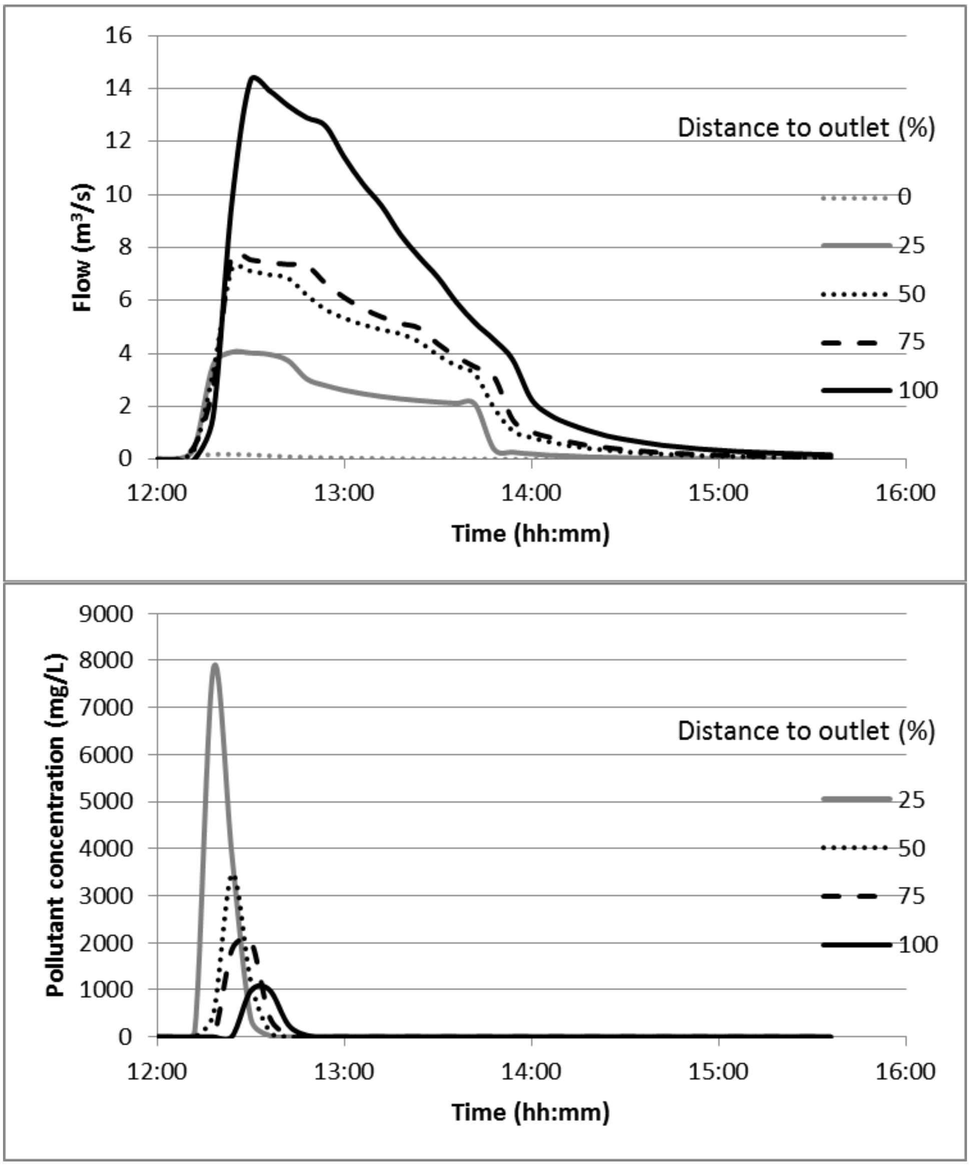


Figure G 6 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 11 (Pit-18250) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 4280 m.

(a)

(b)

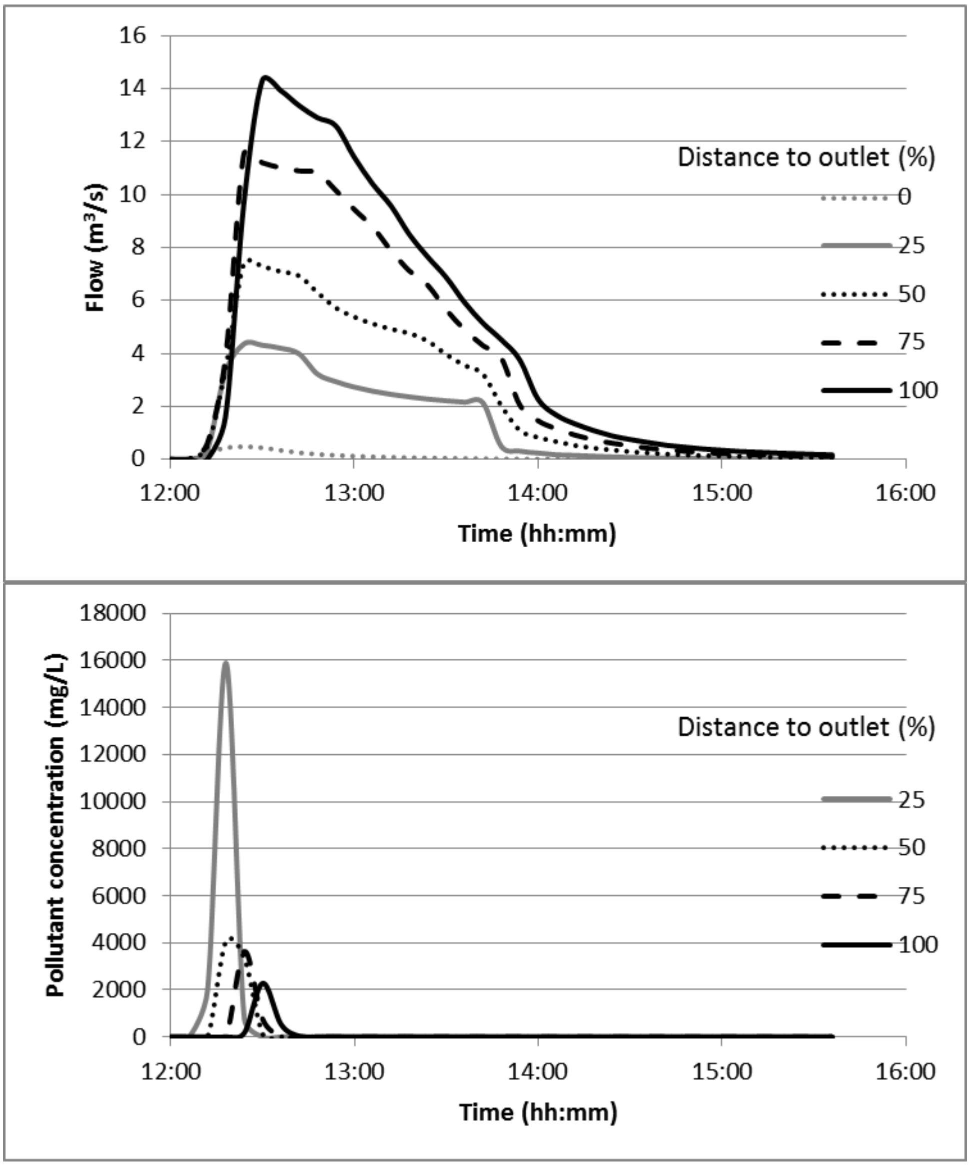
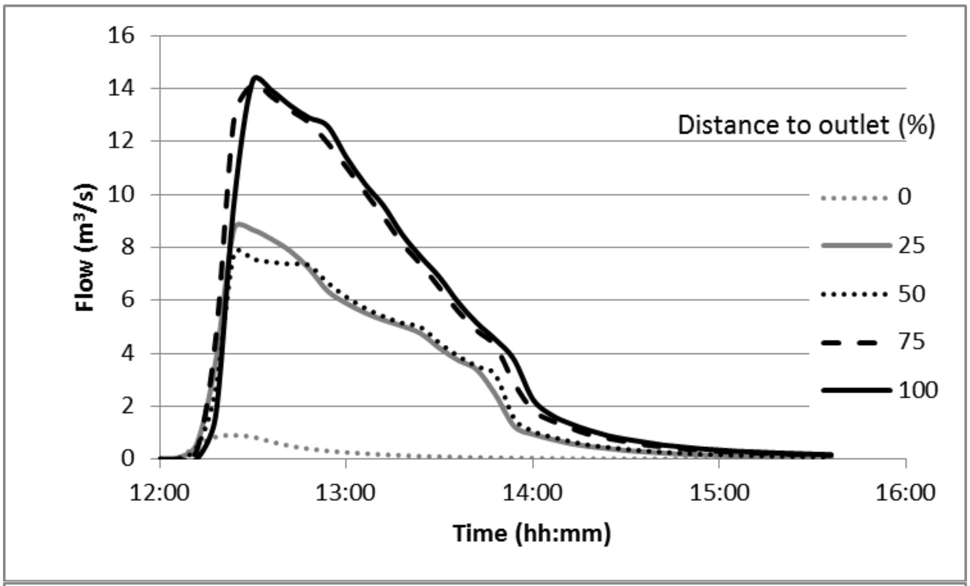


Figure G 7 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 12 (Pit-18251) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 3795 m.

(a)



(b)

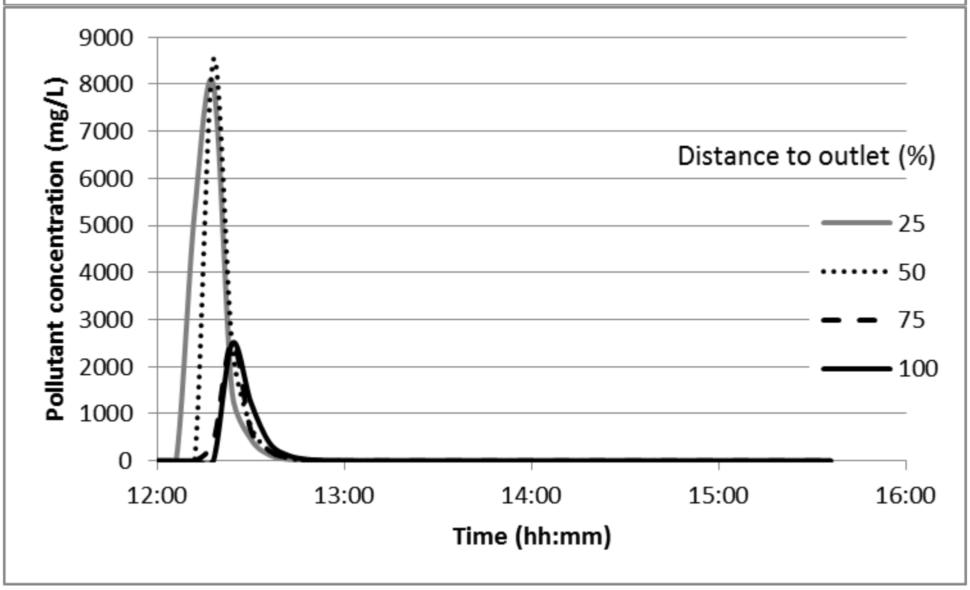
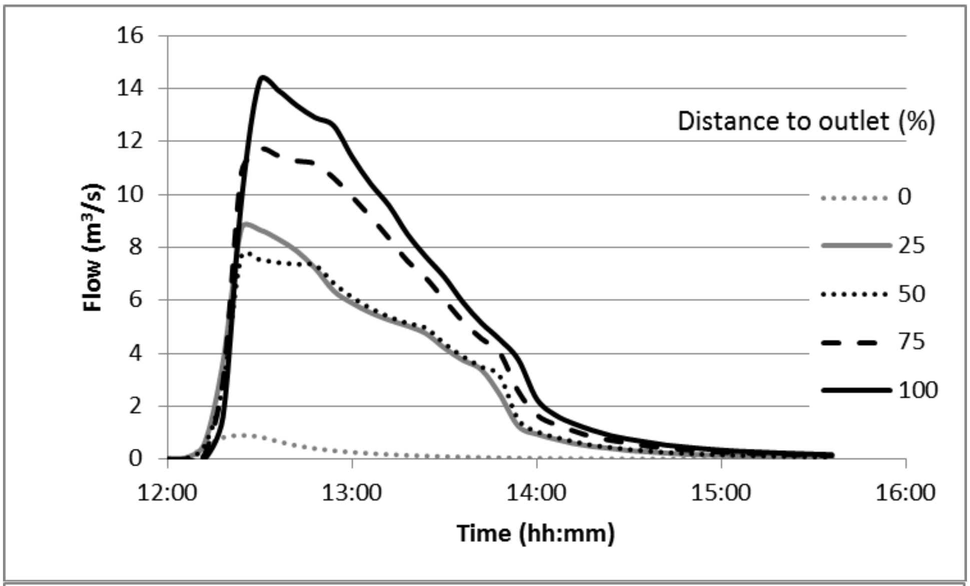


Figure G 8 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 13 (Pit-24527) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 2480 m.

(a)



(b)

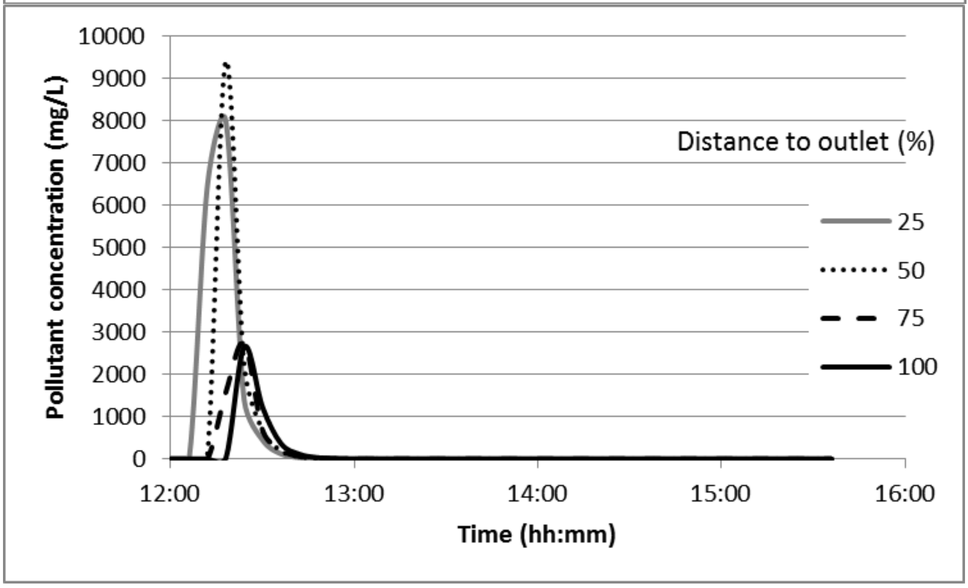


Figure G 9 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 14 (Pit-19692) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 2160 m.

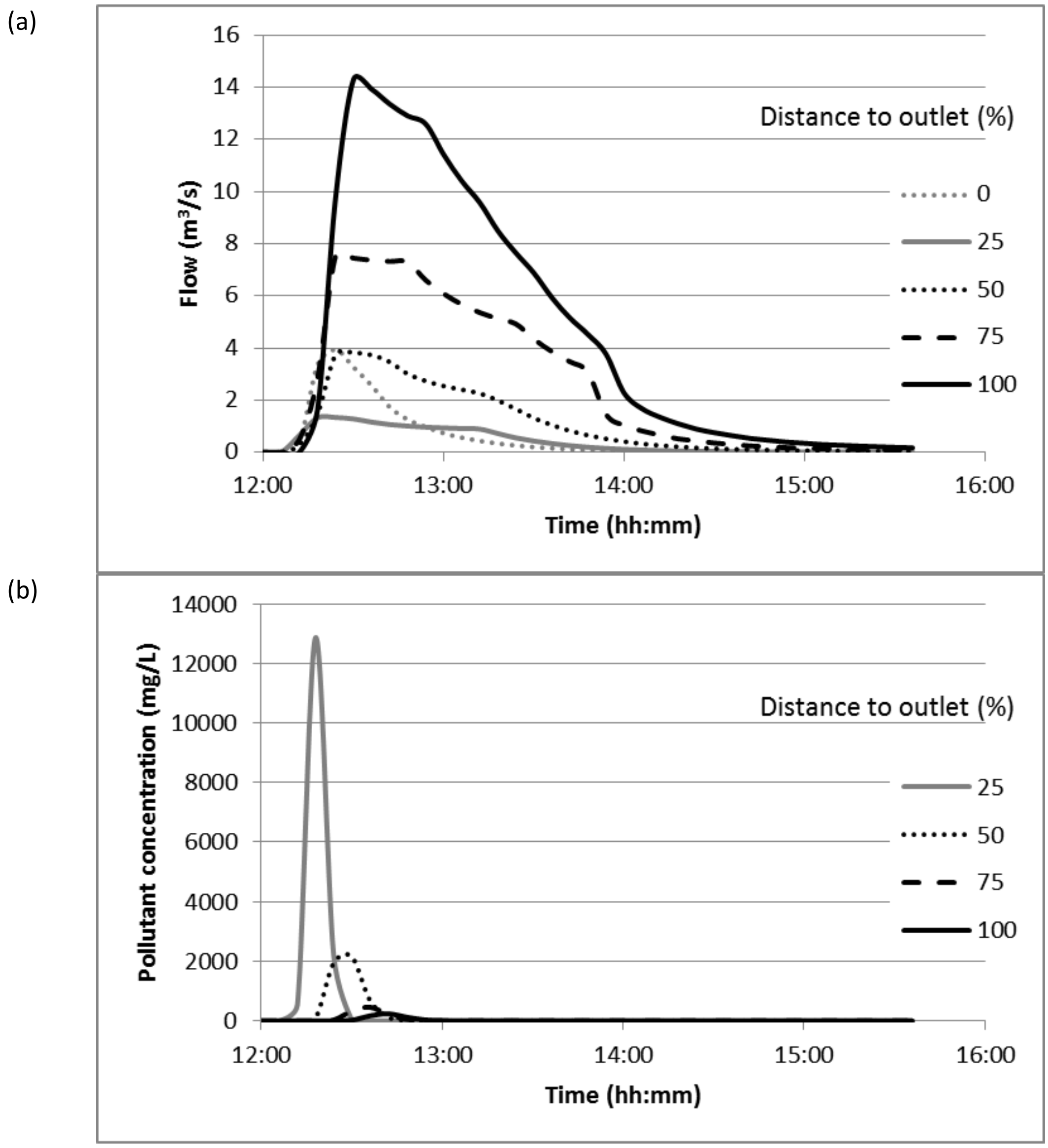


Figure G 10 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 15 (Pit-20089) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 5865 m.

(a)

(b)

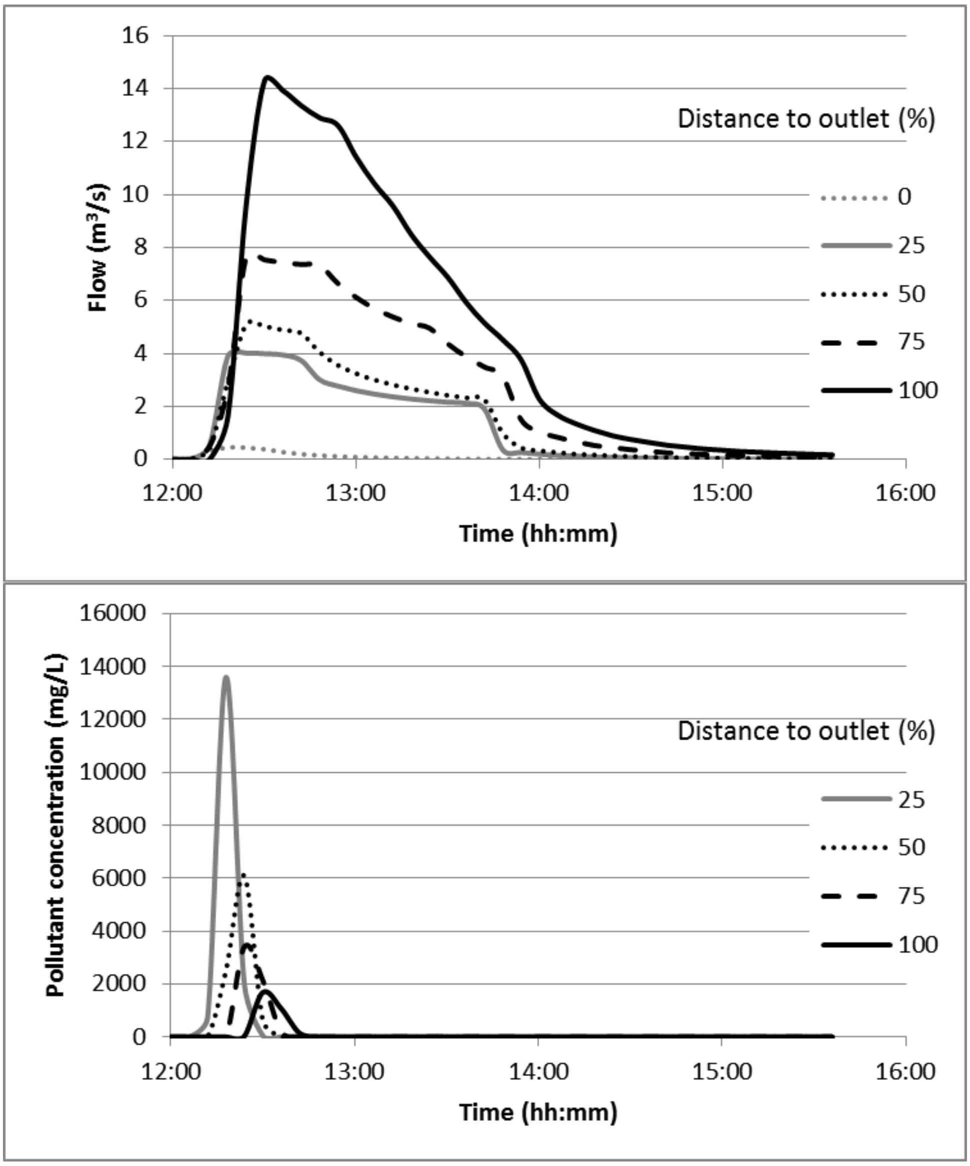


Figure G 11 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 17 (Pit-16017) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 4395 m.

(a)

(b)

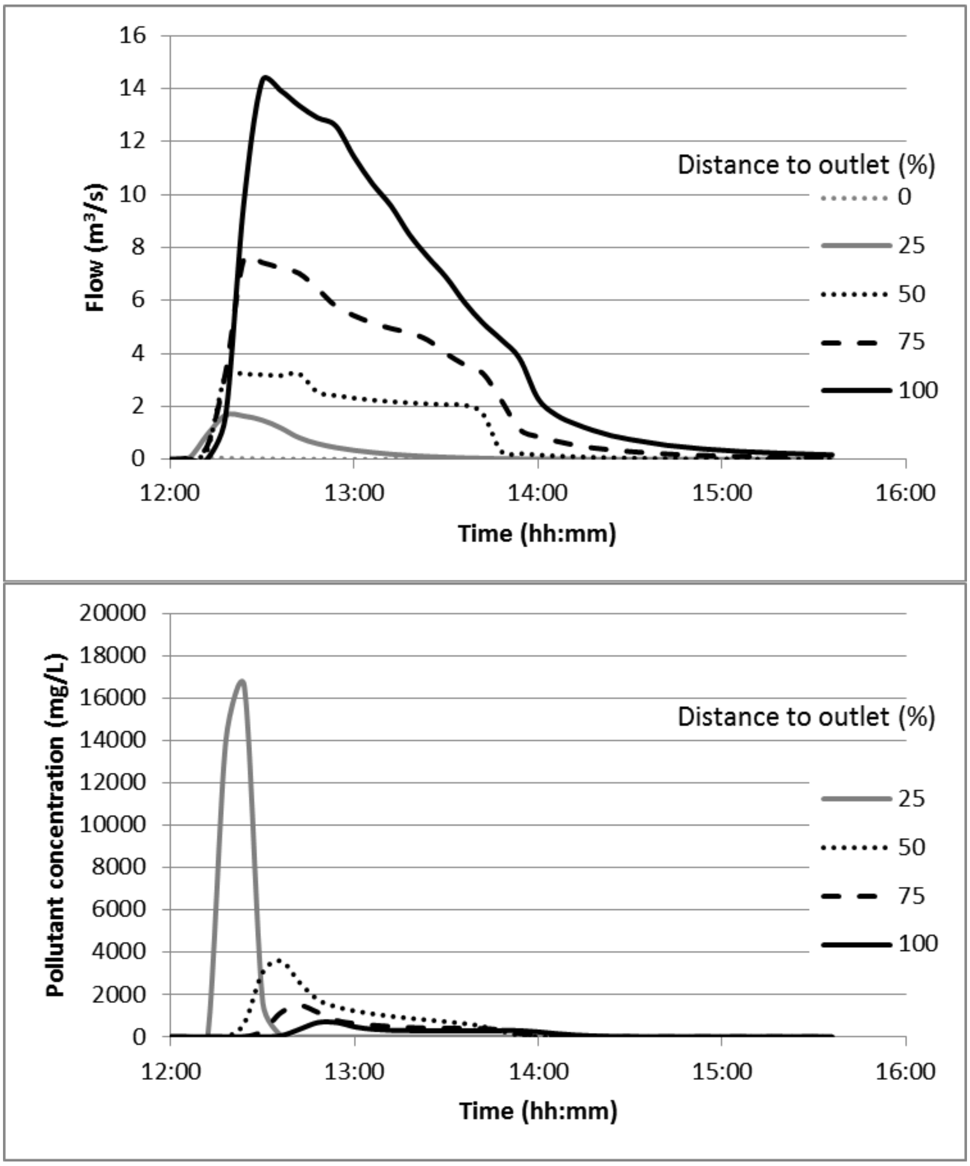


Figure G 12 – The (a) Hydrograph and (b) pollutograph at various points along the flow path from Site 18 (Pit-23138) to the Parafield catchment outlet (conduit C3). Distance is expressed as a percentage of the total 6800 m.

Appendix H – Change in Landuse

Aerial images of the Parafield and Cobbler Creek catchment area are shown in Figures H1 to H6. Figures H1 and H2 compare landuse in the western portion of the catchment from 2002 to 2010, respectively. Figures H3 and H4 compare landuse in the central portion of the catchment area from 2002 to 2010, respectively. Figures H5 and H6 compare landuse in the eastern portion of the catchment area from 2002 to 2010, respectively. In each figure individual subcatchment areas are represented by yellow lines.

The figures indicate that a majority of landuse change has occurred in the Eastern (Cobbler Creek) area of the Parafield and Cobbler Creek catchment areas. This was mainly attributable to continuing residential development proceeding in the vicinity of Golden Grove (compare the central regions Figures H5 and H6). It should be noted that this area only contributes flow to the Parafield drain via pumped flow to the Cobbler Creek dam at a fixed flow rate. There has been some development within the Parafield catchment area, most of which includes industrial development in the south west and residential development in the north east. For ease of comparison, these areas have been circled in red on Figures H1 and H2. It is also possible that there has been infill development of residential lots in the Parafield and Cobbler Creek catchment, which is difficult to identify using aerial imagery.

Figure H 1 - Image of the Western portion of the Parafield and Cobbler Creek catchment, 3 September 2002 (acquired using Google Earth, 2012)

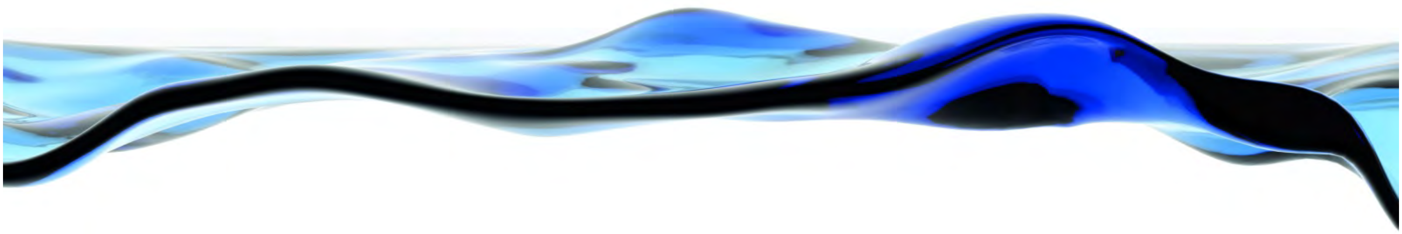
Figure H 2 - Image of the Western portion of the Parafield and Cobbler Creek catchment, 18 February, 2010 (acquired using Google Earth, 2012)

Figure H 3 - Image of the central portion of the Parafield and Cobbler Creek catchment, 3 September 2002 (acquired using Google Earth, 2012)

Figure H 4 - Image of the central portion of the Parafield and Cobbler Creek catchment, 18 February, 2010 (acquired using Google Earth, 2012)

Figure H 5 - Image of the eastern portion of the Parafield and Cobbler Creek catchment, 3 September 2002 (acquired using Google Earth, 2012)

Figure H 6 - Image of the central portion of the Parafield and Cobbler Creek catchment, 18 February, 2010 (acquired using Google Earth, 2012)



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