University of Southern Queensland

Faculty of Health, Engineering and Sciences

Inflow / Infiltration Strategic Management Project

A dissertation submitted by

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Towards the degree of

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Abstract

The impact of inflow and infiltration on hydraulic capacity of sewerage systems has long been known. Numerous attempts are made by sewer system operators to reduce the total flow contributed to the wastewater stream to be that of only domestic wastewater. This process of reduction can be a costly and non-beneficial exercise if not implemented correctly. The development and implementation of well-planned short and long term abatement programs will ensure an efficient and effective service for the community.

To develop a strategic management plan it is important to understand the historical design parameters that were used for the system development. In recent years sewer design codes have been developed to provide best practice methods that rely on the use of hydraulic models to simulate the actual system characteristics. These models attempt to replicate the actual system performance with local climatic characteristics.

The majority of sewer systems are designed to convey effluent via gravity flow. As a result of rainfall and groundwater, additional flows enter the system via pipe joints, cracks and illegal stormwater connections. This additional flow in known as Inflow / Infiltration (I/I) and during periods of heavy rainfall excessive I/I can occur. This results in failure of the sewerage network and effluent escaping to the surrounding environment. The design codes have traditionally incorporated defined values for I/I, these values are empirically included into the design to ensure that the system has adequate capacity to prevent overflows from occurring. The I/I values are not customised to the local climatic conditions and this may be the cause of high I/I during heavy rainfall causing failure of the sewer system.

Various case studies have been undertaken in recent years in the development of the models to customise the Inflow / Infiltration (I/I) values. These values are adopted for the design and operation to suit local climatic conditions. Case studies also provide knowledge of the lessons learnt from abatement strategies and the most effective means to identify and reduce high I/I in catchments.

The project uses a known problematic catchment within the Shoalhaven Water network and establishes baseline data of average flow during dry and wet periods.

This data is used with rainfall events to determine the peak weather flows associated with actual rain events.

A methodology is developed from best practice guidelines to undertake a field analysis of the problematic catchment. This enabled a trail investigation to be conducted during a wet weather event. The field results are analysed and the methodology is reviewed to determine the success of the detection of I/I flows as being a result of infiltration or inflow.

Rectification measures will be developed to provide the largest reduction of I/I that is cost effective and obtainable. This also includes improvements that can be made to the design guidelines, gathering/processing of data, field investigations and rectification measures.

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Limitations of Use

Candidates Certification

I certify that the ideas, designs and experimental work, results, analyses and conclusions set out in this dissertation are entirely my own effort, except where otherwise indicated and acknowledged.

I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

Mr. Ivan Peter Wady

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Signature

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Date

Acknowledgements

Whilst the journey towards obtaining my degree is not complete, the end is now visible. It has been a long time coming and is the first challenge I have ever committed myself to achieve. I never considered the world of opportunities it would open. I need to thank my wife for being patient and allowing me to spend countless nights tapping away at my computer and mumbling to myself. Her commitment and devotion to standing by me whilst I achieve what I considered unachievable is a commitment in its self. I would also like to thank my dog Boof, always by my side and knowing when to annoy me to take a break, have a beer and go for a walk to recollect my thoughts. In many ways this degree should have our three names on it; it is a commitment we have all undertaken. Often in the late of night I have vent my frustration when the realisation that the most difficult problems are simple and fundamental however they both have listened and stood by me.

I would like to acknowledge the assistance of the Southern Queensland University supervisor, Dr Vasantha Aravinthan for her guidance and pointers in the right direction. It is often difficult to express ideas via email however she has always found a way to realise where I am coming from and provided prompts to guide the direction I knew, but didn’t realise, I needed to follow. I would also like to thank Ms Carmel Krogh and Mr Andrew McVey for the opportunities they have provided me in my career, studies, employment and having faith in me. This faith has already allowed me to progress to my current position as No. 1 in No. 2’s!! It is my dream job that challenges me every day, makes me laugh and has only been made possible by all those mentioned above.

Lastly I want to thank my parents, my 2 brothers and sister who have always let me be me, and my friends and work colleagues who have stuck by me. They have all allowed my eccentric points of view to challenge the status quo of doing things different and let me have a crack at doing things my way. It’s nearly time for the drinks to be on me.

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Glossary of Terms

A	Area [ha]
AC	Asbestos Cement
ADF	Average Daily Flow [L/s]
ADWF	Average Dry Weather Flow [L/s]
AHD	Australian Height Datum [m]
ARI	Annual Recurrence Interval
BOM	Bureau of Meteorology
CCTV	Closed Circuit Television
DWF	Dry Weather Flow
EPA	Environmental Protection Agency
EP	Equivalent Population
ET	Equivalent Tenement
GIS	Graphic Information System
GWI	Ground Water Infiltration [L/s]

IFD	Intensity Frequency Duration
I/I	Inflow / Infiltration
IO	Inspection Opening
LGA	Local Government Area
MF	Minimum Flow [L/s]
MH	Manhole
ML	Mega litre
NSW	New South Wales
PDF	Peak Daily Flow [L/s]
PDWF	Peak Dry Weather Flow [L/s]
PVC	Polyvinyl Chloride
PWWF	Peak Wet Weather Flow [L/s]
RDII	Rainfall Dependant Inflow / Infiltration
SCADA	Supervisory Control and Data Acquisition
SCC	Shoalhaven City Council

SPS	Sewer Pump Station
SSOAP	Sanitary Sewer Overflow Analysis and Planning Toolbox
STP	Sewer Treatment Plant
SW	Shoalhaven Water
UPVC	Unplasticised Polyvinyl Chloride
US EPA	United States of America Environmental Protection Agency
VCP	Vitrified Clay Pipe
VSD	Variable Speed Drive
WSAA	Water Services Association of Australia
WW	Wastewater

Chapter 1: Introduction

Sewer system networks are designed on the basis of estimating the expected discharge from a catchment that has a variety of land uses. These land uses include residential, commercial and industrial which all have a variety of activities that are undertaken. Historical metered water records provide an indication into the types of water uses that may be occurring and from this an estimated discharge of wastewater can be approximated.

To enable a desktop analysis to be undertaken an assumption is made to the Equivalent Population (EP) of the area. The equivalent population is related to the discharge of a single person in a standard residential home, and the equivalent population in a home is known as an Equivalent Tenement (ET). Historical evidence has enabled a statistical relationship to be determined for the EP/ET of commercial, industrial and medium to high density residential living. This evidence needs to be correlated to localised conditions and occupancy rates.

The population estimates used to determine the catchment loading provides an indication of the expected Average Dry Weather Flow (ADWF). This flow is the average flow that is expected to occur on a normal dry day over a 24 hour period. Actual flows during this period however will vary and peaks of high discharge to wastewater will be evident, the peak flow is known as Peak Dry Weather Flow (PDWF). In a residential home these peaks are evident in the morning and afternoon as residents use the homes facilities to wash, shower etc. The diurnal curve for other land uses however is different from that of a home; the result is each catchment will develop its own unique characteristic diurnal curve.

At the design stage it is only possible to estimate, by empirical means, the ADWF of the catchment. Experience has shown though that as catchments grow, the PDWF is reduced. This reduction is factored into the empirical design of the catchment. To allow for inflow and infiltration into the sewer system either via groundwater or rainfall designers includes a “Storm Allowance”. This wet weather flow is known as the Peak Wet Weather Flow (PWWF) and this is the ultimate flow that the system is designed for. The PWWF or design flow is projected forward for a known horizon, usually 30 years, to allow for growth in the catchment without needing to augment the system.

Experience has also shown that it is not possible to develop a wastewater system that is not susceptible to inflow or infiltration from either rain water or groundwater. Pressure sewer systems in recent years are reducing the impact through the use of continuous pipe however illegal stormwater connections and leaking toilets and taps are still present.

It is only once a system has operated for a period of time that the true flow characteristics can be determined. These characteristics will also change with time as the catchment grows and land use/habits change. For this reason it is important for wastewater system operators to monitor flow trends and plan for system augmentation prior to the system reaching the ultimate design flow.

Rainfall Derived Inflow and Infiltration (RDII) can drastically reduce the hydraulic capacity of a system, whilst removal of a portion of RDII can extend the hydraulic life span of the system and thus delay expensive augmentation works. Once the hydraulic capacity of a system is exceeded overflows shall occur, these overflows can affect the health of the local environment. In recent years there has been a move by the industry to analyse the system based on the local climatic conditions and to ensure that system capacities are capable of dealing with the majority of rainfall events.

To enable the management of the system it is important for operators to monitor and manage the flows within catchments. When flows are exceeding expectations investigations need to be undertaken to determine the cause. As the largest flow contributor to the hydraulic capacity of the system is the PWWF, a reduction in this component of flow can represent the largest reduction in flows to maintain capacity. Thus the need for development of a strategic approach that enables both short and long term abatement programs to be implemented successfully.

In recent years there has been the development of software to assist with the detection of inflow and infiltration. Utilising flow monitoring and sewer pump station telemetry it is possible to determine the flow hydrographs during

ADWF/PWWF periods. These hydrographs enable the identification of excess wet weather flows as either inflow (immediate impact) or infiltration (delayed impact).

Good Practice guidelines and previous case studies that have been published outline operator’s attempts to identify and rectify I/I. These case studies outline the process undertaken and also the methods used to mitigate the I/I. This experience allows a more informed and cost effective management plan to be established from the lessons learnt by others.

As with all strategic plans it is important to also develop methods to measure the effectiveness of mitigation methods used to reduce the I/I.

1.1.Background

The Shoalhaven Region is located on the South Coast of New South Wales, approximately 160km south of Sydney. The Shoalhaven City Council (SCC) Local Government Area (LGA) is 4660km2 in size, approximately 120km long (North/South) and 80km wide (East/West). It encompasses 19 major waterways, including Jervis Bay, St Georges Basin, Crookhaven River and Shoalhaven River. Nearly 70% of the Shoalhaven is national park, state forest or vacant land. The region has 2 major centres being Nowra/Bomaderry in the north and Milton/Ulladulla in the south. A number of small townships and settlements make up the remainder of the urban areas (SCC, 2010).

It has a permanent population of approximately 85,000 people with a peak population in excess of 275,000 people during peak tourist periods. Shoalhaven Water (SW), a division of SCC, currently operates 13 wastewater schemes within the Shoalhaven Local Government Area.

The main employment sectors are summarised as follows

- Agriculture: Dairy and Oyster Industry
- Defence: HMAS Albatross is located at South Nowra with facilities as well on Commonwealth Land adjoining the southern side of Jervis Bay.
- Education: Including Wollongong University Southern Campus at West Nowra.

- Government agencies: Local and State government including the South Nowra Correctional Facility
- Health: Including a number of retirement villages, 1 large hospital and various smaller facilities.
- Manufacturing: Australian Paper Mill, Manildra ethanol processing and facilities servicing HMAS Albatross
- Tourism: A number of coastal areas have large tourist facilities, mainly caravan parks.

The 13 wastewater schemes are located at Berry, Bomaderry, Bendalong, Callala Bay, Culburra, Huskisson/Vincentia, Kangaroo Valley, Lake Conjola, Nowra, St Georges Basin, Shoalhaven Heads, Sussex Inlet and Ulladulla.

Figure 1.1 - Shoalhaven Local Government Area (LGA) is the extent of the Shoalhaven Region with the location of the various wastewater schemes.

Figure 1.1: Shoalhaven Local Government Area (LGA)

(Source Shoalhaven Water 2013)

Each sewerage system is designed to service the urban area of the various townships. The treated effluent from the treatment plants discharges to either the Ocean or Shoalhaven River. Reuse systems are in operation for 6 of the schemes, with the treated effluent reused on farmland. The Shoalhaven River and Crookhaven River in the north are connected with a large oyster industry located in the lower reaches of both rivers. Nowra and Bomaderry Sewer Treatment Plants (STPs) are licenced to discharge to the Shoalhaven River. Shoalhaven Heads and Culburra STPs are both in close proximity to the oyster leases and have a reuse scheme for discharge of their treated effluent. In the event of untreated effluent escaping to the local environment in these urban areas the impact can result in the closure of recreational and commercial activities due to potential impacts on health.

The 3 sewerage schemes servicing the townships of Lake Conjola, Bendalong and Kangaroo Valley have been commissioned in the past 7 years. These schemes were undertaken to improve the social amenity of the local area as the townships were serviced by either septic tank or onsite disposal systems.

The Nowra sewerage scheme was originally commissioned in 1937 and augmented as the township grew. A point has now been reached which requires major augmentation of both the Nowra and Bomaderry sewerage treatment plants. This will require a large capital investment in excess of $100 million dollars. The intent of the upgrade is to ensure that the communities of Nowra and Bomaderry are able to be serviced for the next 30 year horizon. Appendix B – Overview of Nowra Sewerage Scheme shows the general arrangement of the Sewer Pump Stations (SPSs) and STP in Nowra.

1.2. Problem Identification

All of the sewerage schemes within the Shoalhaven region are impacted upon by I/I to various extents. During wet weather events the hydraulic capacity of several SPS's and STP's is exceeded. At present no overall strategy exists to identify and rectify the issue, the impact of I/I includes

- Non- compliance with environmental licence conditions,
- Excessive cost for pumping and treatment,

- Impact on the local environment,
- Complaints from local residents,

To enable future planning and management of the wastewater system/s a strategy is needed to

- Identify resources required to identify problematic catchments,
- Review past practices and effectiveness of studies undertaken in the Shoalhaven region,
- Determine the impact and frequency of the events,
- Utilise best practice guidelines for the establishment of the strategy,

I have chosen this project as my current role of Wastewater Operations Manager requires me to operate and maintain the various sewer systems that Shoalhaven Water is responsible for. As part of this management I need to ensure that a strategic approach is developed to deal with operational issues. This strategic approach will enable a more efficient and effective use of resources, allow forward planning of system augmentation and ensure that compliance with environmental licence conditions is maintained.

For the development of the strategy a catchment has been identified that has substantial I/I during wet weather events. This catchment is part of the Nowra sewerage scheme and has had several overflow events occur during wet weather.

Chapter 2: Literature Review

The literature review has been undertaken in two (2) parts the first being the review of Standards used for the calculation of hydraulic capacity and the second being a review of case studies for the short and long term abatement of inflow and infiltration.

2.1. Review of Sewer Design Standards

An extensive review of the adopted design practices by Australian water utilities has been undertaken to determine the current practices used for the design and operation of sewerage systems. The Water Services Association of Australia (WSAA) currently has 2 codes that "*sets out to provide guidance by way of general principles, criteria and good practice*" (WSAA, 2004). These codes were originally released in 1999 with the current versions being

- Sewerage Code of Australia WSA 02-2002 Second Edition Version 2.3,
- Sewage Pumping Station Code of Australia WSA 04-2005 Second Edition Version 2.1

The introduction and release of these codes enabled a common approach for Australian water utilities to plan, design, construct and operate sewerage systems. The past practices of water utilities have been to utilise a number of methods and criteria to determine hydraulic capacity. In 1989 at the 13th Federal Convention of Australia Water and Wastewater Association, the manager for planning at the Water Board Sydney noted that

> "*A survey of national design criteria has shown that there is not only a range of methods in use for determining sewer hydraulic capacity but a wide scatter of design allowance. This variation appears to be greater than the variation in prevailing climatic, geographic and geological conditions*" (Browne, 1989).

In Australia there are 65 sewer utilities providing sewerage services to a population of 10,000 or more people. Each of these utilities sets its own design standards for their respective systems.

Table 2.1 - Water Utilities serving 10,000 or more customers is a summary of the number of utilities and the population groups that they service. Australian Capital Territory, Northern Territory, South Australia, Tasmania and Western Australia have only one sewer service provider whilst New South Wales has 25. Each service provide however may operate a number of regions which then control a number of wastewater schemes.

Table 2.1: No. of Water Utilities Serving 10,000 + customers

State	Region Size (based on customers)				No. of Utility Providers
	100,000 +	50,000 to 100,000	20,000 to 50,000	10,000 to 20,000	
Australian Capital Territory	1				1
New South Wales	3		9	13	25
Northern Territory		1		1	1
Queensland	4	3	3		10
South Australia	1			2	1
Tasmania		1			1
Victoria	4	5	5	2	16
Western Australia	1			6	1
Total	14	10	17	24	56

(Source National Water Commission 2014)

Note: This summary excludes Melbourne Water which is a bulk water utility.

Appendix C - Water Utility and Design Standard is a summary of the various standards used by water utilities. The main design standards used was those published by the Water Services Association of Australia (WSAA) however variations of these standards have been developed for Victorian water utilities, Sydney and Hunter Water.

Other standards are also still being used. A summary of the standards is as follows;

- WSA 02-2002 Sewerage Code of Australia Version 2.3.
- WSA 02-2002 Sewerage Code of Australia Sydney Water Edition Version 3.
- WSA 02-2002 Sewerage Code of Melbourne Retail Water Agencies Edition (MRWA) Version 1.0.
- WSA 02-2002 Sewerage Code of Australia Hunter Water Edition.
- NSW Public Works Manual of Practice Sewer Design (1984).
- Aus –Spec Development Design Specification D12 Sewerage System.
- Metropolitan Water and Sewerage Drainage Board - Sydney Manual Design. of Separate Sewer Systems (1979).
- Queensland Planning guidelines for water supply and sewerage 2010.

The majority of the utilities that adopted the WSSA sewerage code also had supplements to the design code for local variations. In NSW some utilities used more than one code, whilst in Queensland the traditional method as outlined in the planning guidelines is also used by smaller utilities i.e. Calliope Shire Council. In Victoria there was a consistent approach across all utilities to utilise the same standard whilst in the Northern Territory the standard specified is the 1979 edition of the now Sydney Water Board. It is also important to note in NSW that the three (3) Water Authorities being Sydney Water, Hunter Water and the recently formed Central Coast Water Corporation have individual legislation for their operation, whilst the remaining water utilities are under the management of the local Councils and the NSW Office of Water.

A review of the WSAA sewerage code recommended that for existing catchments the actual system performance should be used for analysis of flows however an empirical method can be used where flow monitoring is not available. This same methodology was echoed in the Queensland planning guidelines. The NSW Public Works however only provided an empirical methodology which is also repeated in the Hunter Water version of the WSAA sewerage code.

2.2. Hydraulic Capacity

The basis of all design criteria is to determine a design flow to ensure hydraulic capacity of the system is adequate; the following definitions are used in the determination of the design flow.

Average Dry Weather Flow (ADWF): The combined average sanitary flow into a sewer from domestic, commercial and industrial sources.

Design Flow: The estimated maximum flow into a sewer comprising the sum of peak dry weather flow (PDWF), ground water infiltration (GWI) and stormwater inflow and infiltration (IIF) (WSAA, 2002).

Equivalent Population (EP): The equivalent hypothetical residential population that would produce the same peak dry weather flow as that contributed by the area under consideration i.e. all zonings including residential, commercial and industrial. For a single residential dwelling the occupancy rate adopted is 3.5 (WSAA, 2002). Examples of EP for different zonings have been provided in Appendix D – Sewer Design Code Equivalent Populations for Synchronous discharges.

Equivalent Tenement (ET): This value is the equivalent residential houses that would produce the same ADWF as that contributed by the area under consideration i.e. all zonings including residential, commercial and industrial. A local occupancy rate can be used to determine the EP. Appendix E – Water Directorate NSW Standard ET

Groundwater Infiltration (GWI): is caused where the long-term non-rainfall dependent groundwater table or seawater level exceeds pipe invert and enters the sewer network (WSAA, 2002).

Inflow/Infiltration (I/I): This is the peak (rainfall dependant) inflow and infiltration that may enter the sewer network as inflow via illegal stormwater connections or localised flooding of yard gullies and as rainfall infiltration through pipe and maintenance structure defects (WSAA, 2002)

Peak Dry Weather Flow (PDWF): The most likely peak sanitary flow in the sewer during a normal day.

Based on the above definitions the design flow, also known as the Peak Wet Weather Flow (PWWF) is able to be calculated as follows:

$$\textbf{Design Flow (PWWF)} = \textbf{PDWF} + \textbf{GWI} + \textbf{IIF} \qquad \textbf{(2.1)}$$

2.3. Average Dry Weather Flow

The estimation of ADWF can either be done by flow monitoring or empirical estimation. In existing systems, where practical, flow monitoring is undertaken to establish the flow characteristics of the catchment. Where it is not possible to physically gauge the flow in a system the empirical method can be used. For new growth areas (land subdivision) the empirical method is adopted to determine the estimated flow.

The empirical method requires the establishment of an estimated flow per person, this value is then multiplied by the EP of the catchment to determine the ADWF of the catchment. "*Based on empirical evidence, ADWF is deemed to be180L/EP/d or 0.021 L/s/EP*"(WSAA, 2002).

In Queensland the "*Planning Guidelines for Water Supply and Sewerage 2010*" notes that "*generally ADWF will range from 150-275 L/EP/d*" (Queensland Water Supply Regulator, Water Supply and Sewerage Services, Department of Energy and Water Supply).

In NSW the Water Directorate notes that "*Average dry weather sewage rates generally lie between 0.004 L/s/ET and 0.011 L/s/ET. It is generally accepted that a sewer ET represents an average loading of around 0.008 L/s at both state and local level with the accepted design value being 0.011 L/s/ET*" (Water Directorate, 2005).

The NSW Public Works recommended at design value of 0.011 L/s/ET (NSW Public Works, 1984). In 1994 the NSW Public works department recommended a value of 240L/EP/d (NSW Office of Water, 2012).

Based on the occupancy rate of 3.5 as defined by WSAA, the NSW ADWF as determined by the Water Directorate would be equal to 271L/EP/d with an average of 197L/s/d.

In non-metropolitan NSW the NSW Office of Water recommends an ADWF value of 200 L/EP/d be adopted. This value represents a 75% sewer discharge factor for the 250kL/annum medium residential water supplied per connected property for inland utilities. This value represents a typical occupancy rate of 2.6 persons per house (NSW Office of Water 2012). Sydney Water has adopted a value in line with WSAA of 180L/EP/d, the average water consumption in Sydney is 623L/ET/d.(Sydney Water, 2014).

When flow monitoring of the catchment is undertaken to determine ADWF the flow will consist of domestic wastewater and GWI, the GWI can be determined as the flow that occurs during the early morning hours i.e. 12am to 4am (USQ, 2011).

In summary the ADWF as adopted by WSAA of 180L/EP/d is within the ranges recommended both in Queensland and NSW. Local assessment of the ADWF, where possible, should be undertaken to ensure consistence with internal water usage. This can be done by either calibration of metered water usage with measured sewer treatment plant (STP) flows or flow monitoring. Table 2.2 – ADWF Adopted Value Comparison summarises the variations noted above.

Table 2.2: ADWF Adopted Value Comparison

Authority	ADWF (L/EP/d)
Water Service Association of Australia	180
Water Directorate (NSW)	99 - 271
Department of Energy and Water (QLD)	150 - 275
Public Works (NSW)	240
Office of Water (NSW)	200
Sydney Water	180

2.4. Peak Dry Weather Flow

The PDWF can be related to the ADWF by a peaking factor (d).

$$\text{PDWF} = d \times \text{ADWF} \tag{2.2}$$

The WSAA sewer code relates the peaking factor to the gross development area.

The value of d can be calculated using the following formulae;

$$d = 0.01(\log A)^4 - 0.19(\log A)^3 + 1.4(\log A)^2 - 4.66(\log A) + 7.57 \tag{2.3}$$

'A' is the gross plan area of the development catchment, in hectares. This relationship may be used for catchments up to 100,000 hectares.

The NSW Public works adopt a different method for the calculation of the peaking factor; this factor is denoted 'r' in the 1984 sewer design standard and uses the no. of tenements (T) for the calculation.

$$r = \sqrt{1.74 + \frac{56}{T^{0.4}}} \; for \; T > 30 \tag{2.4}$$

The historical Queensland approach adopted a different method for the calculation of peaking factor, this factor is denoted 'C_2' and uses the EP for the calculation

$$C_2 = 4.7 \times (EP)^{-0.105} \tag{2.5}$$

(Queensland Water Supply Regulator, Water Supply and Sewerage Services, Department of Energy and Water, 2010)

Figure 2.1: Comparison of Peaking Factors for PDWF is a comparison of the three (3) peaking factor methods. The Queensland EP has been converted to ET using an occupancy rate of 3.1 (USQ, 2011). As it can be seen, the reduction of the peaking factor occurs as the population increases although once greater than 200 ET's there is minimal reduction. It appears, based on the graph, the WSAA peaking factor allows for a higher density of population for an area which is consistent with a move towards high population densities in urban areas.

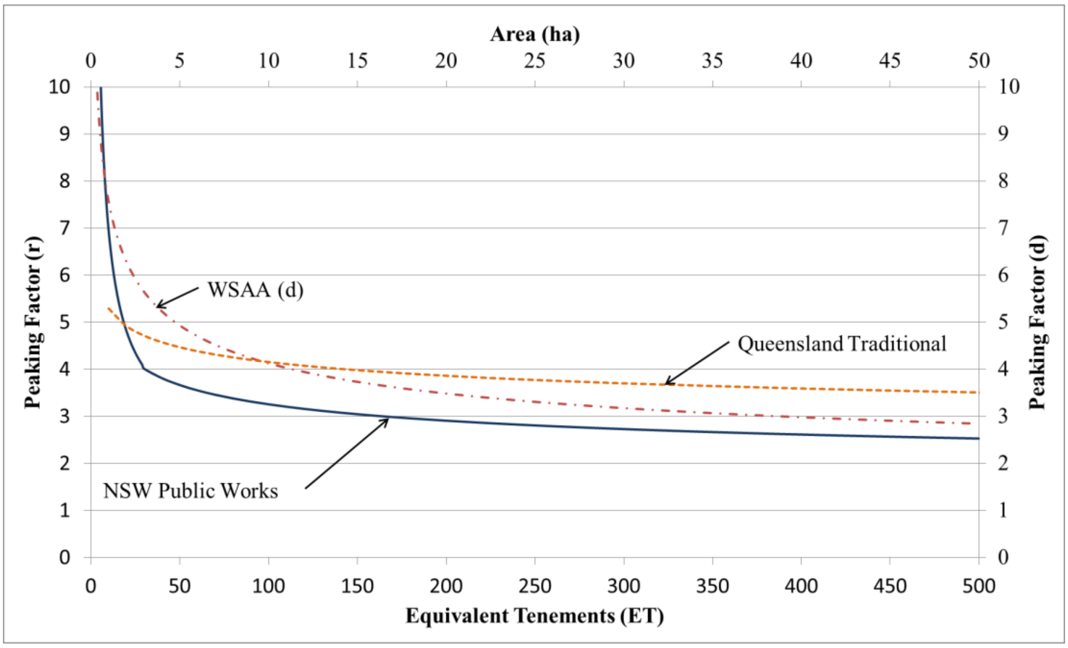

Figure 2.1: Comparison of PDWF Peaking Factors

Note: NSW Public Works is based on ET's, Queensland traditional method is based on EP, and an occupancy rate of 3.1 has been applied to determine ET. The WSAA method uses area.

A discrepancy between peaking factors is still evident at 500 ETs, this difference results in a variation in the calculated PDWF. The Queensland method uses the EP of a residential home whilst the Public Works method uses equivalent tenements to calculate the peaking factor. In NSW an ET is based on 2.6 EP which is lower than the Queensland adopted value of 3.1EP thus if 2.6 EP was used the Queensland peaking factor would be closer to that of the other two peaking factors shown.

Dry weather residential flows over a period of 24 hours will vary according to internal water usage; this pattern is known as the diurnal curve and is different for weekdays/weekends. Figure 2.2 – Residential Diurnal Curve shows this variation in flow. It can be seen that there are 2 peak flows that occur, the first peak in the morning and the second in the afternoon. The curves are similar for the weekdays whereas Saturday and Sunday have a distinctly different flow pattern with higher wastewater discharge on the Sunday. This is indicative of habitual patterns with residents being home on the weekends and using more water for washing etc.

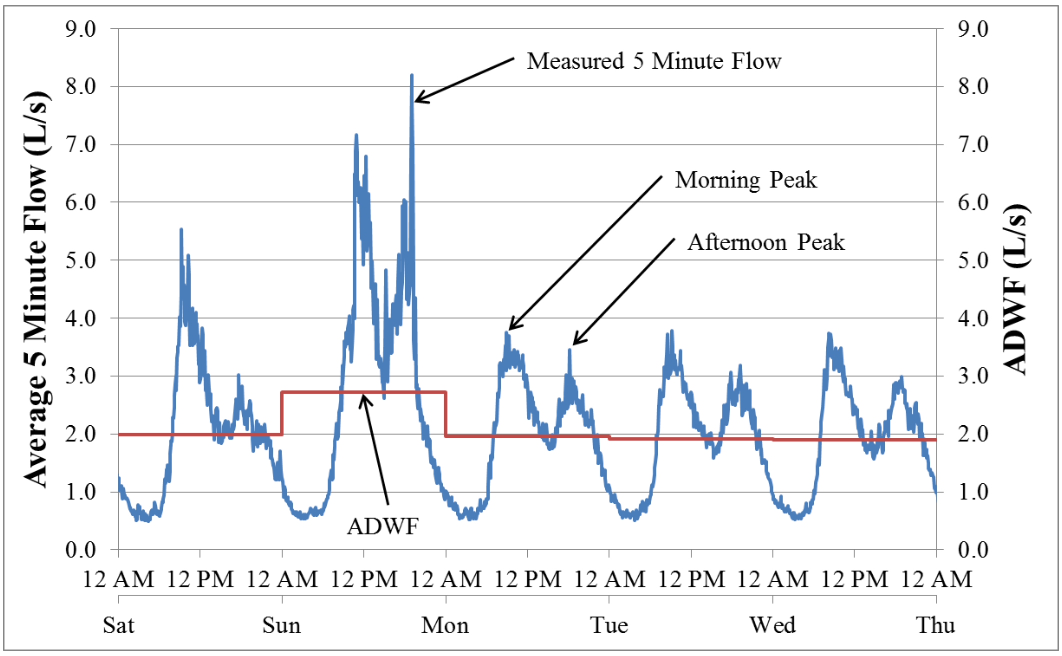

Figure 2.2: **Residential Diurnal Curve**

(Source Shoalhaven Water Flow Monitoring Records)

Figure 2.3 – Industrial Estate Daily Flow Variation shows the diurnal pattern for wastewater discharge in an industrial area. This curve indicates one peak in the 24 hour period showing a consistent wastewater discharge during the hours of operation.

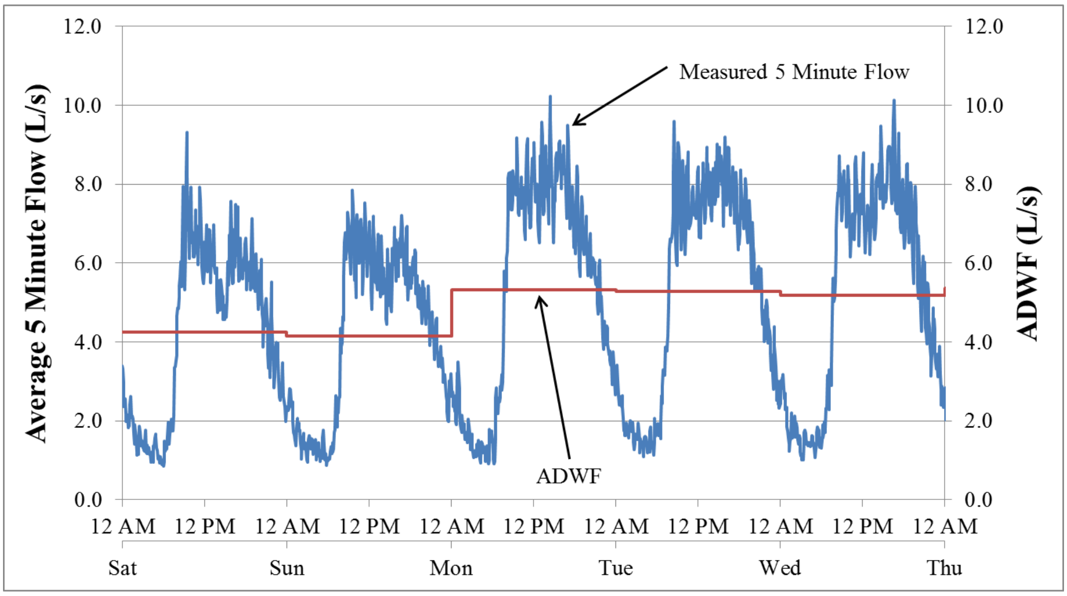

Figure 2.3: Industrial Estate Daily Flow Variation

(Source Shoalhaven Water Flow Monitoring Records)

2.5. Peak Wet Weather Flow

Three methods for calculating PWWF are shown below

1. NSW Public Works Method

$$PWWF = PDWF + SA \quad (2.6)$$

Where SA = Storm Allowance [0.058 L/s/ET]

2. Queensland Traditional Method

$$PWWF = 5 \times ADWF \text{ or} \quad (2.7)$$

$$PWWF = C_1 \times ADWF \quad (2.8)$$

Where $C_1 = 15 \times (EP)^{-0.1587}$ (Note C_1 Minimum = 3.5)

3. WSAA Method

$$PWWF = PDWF + GWI + I/I \quad (2.9)$$

Where GWI = Groundwater infiltration [L/s]
I/I = Inflow/infiltration

The calculation of ground water infiltration, using WSAA, is done using the following formulae

$$GWI = 0.025 \times A \times Portion_{Wet} \quad (2.10)$$

Where A is the gross plan area of the developments catchment, in hectares.
$Portion_{Wet}$ is the portion of the planned pipe network estimated to have groundwater tables in excess of pipe inverts. For example if 70% of the sewer system is below groundwater table levels, then $Portion_{Wet} = 0.7$.

If flow monitoring data is available an estimate of GWI can be made "*by analysing the minimum night time flow (12am to 4am). For primary residential areas up to 80% of the minimum flows can be due to GWI with the remaining night time flows attributed to domestic use, in particular leaking cisterns. In commercial/industrial* *areas, the potential for 24 hour industries and automatic urinal flushing needs to be taken into account*" (WSAA, 2011).

The WSAA method for calculating inflow/infiltration is as follows

$$IIF = 0.025 \times A_{Eff} \times C \times I \tag{2.11}$$

A_{Eff} is the effective area capable of contributing rainfall dependant infiltration. For residential developments A_{Eff} is a function of the development density

$$A_{Eff} = A \times (Density/150)^{0.5} \text{ for Density} < 150 \text{ EP/ha} \tag{2.12}$$

$$A_{Eff} = A \text{ for Density} > 150 \text{ EP/ha} \tag{2.13}$$

Where A is the gross plan area of the developments catchment, in hectares. Density is the developments EP density per gross hectare.

For commercial and industrial developments A_{Eff} is a function of the expected portion of the catchment to be covered with impervious structures, i.e. roofs, sealed roads, car parks.

$$A_{Eff} = A \times (1 - 0.75 \text{ Portion}_{Impervious}) \tag{2.14}$$

Where A equals the gross plan area of the developments catchment

$Portion_{Impervious}$ equals the portion of the gross plan area likely to be covered by structures that drain directly to the stormwater system i.e. 20% = 0.2.

C equals the leakage severity co-efficient and it defines the contribution of rainfall run-off to sewer flows. It is the sum of contributions from soil movement and network defects.

Table 2.3 – Leakage Severity provides guidance for the range of values that can be adopted for leakage. This value is a combination of soil and network conditions. Sand is classified as a low impact soil whilst aging pipework is often classed as a high impact network.

Table 2.3: Leakage Severity

Leakage Severity Co-efficient (C)		
Influencing aspect	**Low Impact**	**High Impact**
Soil aspect, S_{aspect}	0.2	0.8
Network aspect, N_{aspect}	0.2	0.8
$C = S_{aspect} + N_{aspect}$	Min = 0.4	Max = 0.8

(Source WSAA, 2002)

I is a function of rainfall intensity at the developments geographic location, catchment area and required sewer system containment standard.

$$I \quad = \quad I_{1,2} \text{ x } Factor_{Size} \text{ x } Factor_{Containment} \qquad (2.15)$$

$I_{1,2}$ is the 1 hour duration rainfall intensity at the location, for an average recurrence interval of 2 years.

$Factor_{Size}$ accounts for the faster flow concentration times in smaller catchments.

$Factor_{Containment}$ reflects local environmental aspects and regulations on wet weather sewerage containment (overflow frequency).

The design should incorporate the Average Reoccurrence Interval (ARI) of sewage overflow, which is adopted by the water agency. Given a specified ARI, $Factor_{Containment}$ may be either taken from Table 2.4 or calculated as follows:

$$\boldsymbol{Factor_{Containment} \quad = \quad 0.77 \; x \; \frac{10^{0.43X}}{10^{0.14X^2}}} \qquad (2.16)$$

Where X = Log_{10} (ARI) and ARI is the specified containment in years.

Table 2.4: ARI Containment Factor

ARI	1 month	3 month	6 month	1 year	2 years	5 years	10 years
Factor ($_{Containment}$)	0.2	0.4	0.6	0.8	1.0	1.3	1.5

(Source WSAA 2002)

After a review of the NSW Public Works sewer design manual and discussion with other engineers in the workplace, it was uncertain how the value of 0.058 L/s/ET has been derived. Advice from Manly Hydraulics Staff (a division of NSW Public Works) is that the value was adopted as the maximum metered water usage of a standard residential household in the late 1970's (Dakin, SK, 2014 pers. comm 2nd August). Whilst the storm allowance does allow for ease of calculation in essence this value has no relationship to actual rainfall derived I/I nor does it take into account climate variation across different regions of NSW.

There is evidence of Councils in NSW modifying the storm allowance. Shoalhaven City Council (SCC) is currently using two storm allowances with SA = 0.058 L/s/ET for old areas and 0.030 L/s/ET being adopted for new works (SCC, 2013). Wagga Wagga City Council (WWCC) adopted a storm allowance of 0.029 L/s/ET. WWCC noted that this was 50% of the NSW Public Works value (WWCC, 2013) and justified the reduction with a comparison of rainfall data between Wagga Wagga and Sydney.

There was evidence of a simple empirical method of calculating PWWF by multiplying ADWF by a factor. This method appears to have been used historically as it was difficult to quantify Rainfall Derived Inflow and Infiltration (RDII). The Sydney Metropolitan Drainage Board used 6 x ADWF for pump station and 8 x ADWF for sewers (Dunning, 1958). This method was justified in 1958 for the design of the Wellington (New Zealand) Pumping Station which was based on a maximum flow of 230 gal/head/day, being 6 times the ADWF (Dunning, 1958). This method is still used by utilities today with Byron Bay Council adopting PWWF = 7 x ADWF as a standard to be met for its level of service (Byron Bay Council, 2013).

2.6. Review of Inflow / Infiltration Case Studies

The problems with I/I in sewer systems have long been known. In 1956 in New Zealand an Engineer noted that the illegal connection of stormwater downpipes was resulting in increased flows and the city had employed 2 inspectors to review household compliance (Mawson, 1956).

In the 1960's flexible jointed sewers were introduced and the flexible joint was extended to property service connections in Melbourne in 1973 (Barnes et al, 1975). This was an attempt to reduce the impact of infiltration by the use of improved construction materials.

In 1975 it was noted that the Melbourne sewerage system was impacted upon by I/I. This impact was a result of

- Groundwater infiltration via fractured pipes and joints
- Wastewater from leaking fittings
- Stormwater inflow from illegal connections
- Flooding at manhole covers.

It was noted that allowances had been made at the design stage with the existing system commissioned 77 years early and that increase in dry weather flows had taken up capacity and thus these allowances for I/I had been reduced. An infiltration steering committee was assigned the responsibility of reviewing current practices and modifying design and construction methods to minimise infiltration (Barnes Et al, 1975).

The Sydney Water Board detailed in 1992 its sewer gauging strategy that had been developed on 10 years of historical gauging data. The intent of the strategy was to significantly reduce the main sources of I/I by:

- Identifying areas which contribute to most of the problem
- Use accurate computer modelling of the system
- Plan and measure the effectiveness of remedial work
- Enable management of the entire sewer network

It was intended that this would be undertaken using both short and long term gauging strategies. It was also noted that overseas experience showed that 4 to 10 significant storms are needed to identify high I/I areas using gauging methods (Beardsley, 1992).

A recent guideline, "*Management of Wastewater System Infiltration and Inflow Good Practice Guideline*" released by WSAA (2013) concluded that unless at least 40% of the total piped system within a catchment is rehabilitated there is no guarantee of reducing RDII. Table 2.5 – Reduction of RDII for Public Sewer Rehabilitation summaries WSAA findings.

Table 2.5: Reduction of RDII for Public Sewer Rehabilitation

% of Total Public System Rehabilitation	Reduction in RDII (%)	Reduction in GWI (%)
100	60	+/- 80
80	40	+/- 70
60	20	+/- 50
40	0	+/- 30

(Source WSAA, 2011)

North Shore City Council (New Zealand) rehabilitated 32 catchments over a 10 year period. They discovered that two (2) variables could be used to provide a prediction of the reduction of RDII% and PWWF. The two variables were the percentage of total network (private and public) rehabilitate and the initial leakiness of the system (RDII%). This lead to the development of the following 2 equations (WSAA, 2013)

$$\text{RDII\%} = \text{Initial RDII factor x Percentage complete} \tag{2.17}$$

$$= (0.257 \text{ Ln } (-0.0445x + 0.0445 + \text{RDII}_{pre}) + 0.988) * X^{1.055}$$

$$\text{Peak Flow \% reduction} = \text{Initial RDII factor x percentage complete} \tag{2.18}$$

$$= (0.303\text{Ln } (-0.0445x + 0.0445 + \text{RDII}_{pre}) + 1.163) * X^{0.761}$$

WSAA noted that the above has not been calibrated in Australia to date due to insufficient I/I analysis.

Goulburn Valley Water, Victoria has undertaken a I/I study since 2008. It was determined that if the flow and rainfall data collection methodology is well developed and executed this will result in more successful calibration of I/I estimations (WSAA, 2011).

From seven case studies reviewed by WSAA (WSAA, 2011) it was determined that the following steps should be followed in the establishment of an I/I abatement strategy:

- A survey to quantify major inflow will need 3 to 4 storms of significant wet weather events,
- There is a near proportional relationship between rainfall depth and inflow,
- Effective management of data i.e. SCADA to enable ready interface with analysis platforms,
- Calibrate the hydraulic model for dry weather flow,
- Calibrate for wet weather flow,
- Verify the model,
- Undertake an Options Assessment,
- Develop a remediation plan,
- Implement remediation plan,
- Benefits realisation review,

The remediation plan can consider a number of options that includes, maintenance hole inspection, smoke testing, dye testing, CCTV and data management of inspections.

All of these abovementioned remediation works are currently undertaken, to varying extent, by Shoalhaven Water. The relining of the pipe network that has been completed appears to have minimal impact on I/I.

The U.S Environmental Protection Agency (US EPA) in 2009 released its first version of the Sanitary Sewer Overflow Analysis and Planning Toolbox (SSOAP). This toolbox was developed by the EPA and CDM Inc. and has been effective in sewer condition assessment and rehabilitation to support wastewater system improvements (US EPA, 2012).

The SSOAP toolbox requires flow monitoring data, rainfall data and sewer system data to generate RDII hydrographs and determine dry and wet weather flow. The software has been made available in the public domain free of charge with the intended user to be wastewater system operators. The intent of this is in the interest of the community and to remove the cost prohibition that commercial software can be for smaller organisations to undertake an analysis.

The software is available at http://www.epa.gov/nrmrl/wswrd/wq/models/ssoap/.

WSAA has also recently released an excel toolkit for analysing SCADA pump run time data, this toolkit also produces hydrographs. A paper on the WSAA Good Practice Guideline for the management of Wastewater system infiltration and inflow was discussed at the Ozwater conference in April 2014 (paper 26).

A copy of this software does not appear on the WSAA website however WSAA have been contacted to obtain a copy of the software.

2.7.Conclusion

The current sewer design practice as set out by WSAA in its sewer design code of Australia is the most suitable method for estimating flow parameters. This method utilises actual system performance via flow monitoring and determines the flow components in relation to the local climatic conditions for a known ARI. This method follows Best Practice Guidelines and provides a degree of certainty to the actual performance of the system during a wet weather event.

Utilising the SSOAP toolbox or the WSAA toolkit will enable an I/I analysis to be undertaken using existing SPS SCADA records and Bureau of Meteorology (BOM) rainfall data. The hydrographs will provide an indication as to whether the majority of the problem is a result of Inflow or Infiltration.

A comparison of the PWWF value that has been adopted by Shoalhaven Water, using the traditional method, will also be compared to the PWWF determined by the WSAA method. This will determine if the existing system is exceeding design capacity as a result of local climatic rainfall conditions.

A field analysis of the system will be undertaken; the methodology for this analysis will be built upon using Best Practice Guidelines established by WSAA. The field analysis will be undertaken during wet weather events and is reliant on suitable rainfall occurring during the investigation period.

The strategy for the management of I/I will be developed taking into account the ease of desk top analysis and the success / failure of the field analysis. Rectification measure to mitigate I/I will be developed and suggestions for improvements to the design guidelines will be made.

Whilst it is not intended to solve the complex issue of I/I into sewer networks the strategy will provide guidance to the work that needs to done and the processes that need to be improved to permit better management of the effects of I/I.

Chapter 3: Overall Aim and Objectives

The Nowra sewerage scheme has a permanent population of 29,400 people with minimal tourist population. The scheme consists of a number of catchments that transfer effluent via gravity to sewer pumping stations (SPSs) or direct to Nowra Sewer Treatment Plant (STP). The Nowra Scheme consists of 29 SPSs and 210km of gravity sewer mains. One catchment within this scheme shall be used as a project site for analysis.

3.1.Project Site

The Project site is one catchment from the Nowra scheme; this catchment is the St Ann's St Catchment. This catchment transfers effluent via a SPS, known as SPS 3. It consists of multiply land uses including residential, commercial and industrial. In addition the catchment also has 5 SPSs discharging to the gravity network.

The 5 SPS's and their land uses are summarised as follows

- SPS 15: Residential and receives pump flows from 2 upstream SPSs including HMAS Albatross industrial services,
- SPS 21: Residential with a large integral energy complex,
- SPS 23: South Nowra Industrial precinct,
- SPS 26: University of Wollongong southern campus,
- SPS 29: South Nowra Correctional Facility.

Figure 3.1 – St Ann's St Catchment overview shows the location of SPS 3, the main trunk gravity mains (shown blue) and the 5 SPS's (15, 21, 23, 26 and 29) that discharge into the trunk gravity system. The rising mains for the SPS's are shown in red.

Figure 3.2 - St Ann's St Gravity Catchment (page 30) shows an overview of the gravity portion of the catchment. The Shoalhaven Water Graphic Information

System (GIS) shall be utilised for the location and size of the sewer infrastructure as part of the field analysis.

During wet weather events, the St Ann's Street catchment is heavily impacted by I/I, SPS 3 and manholes within the gravity system draining to the SPS overflow during these events.

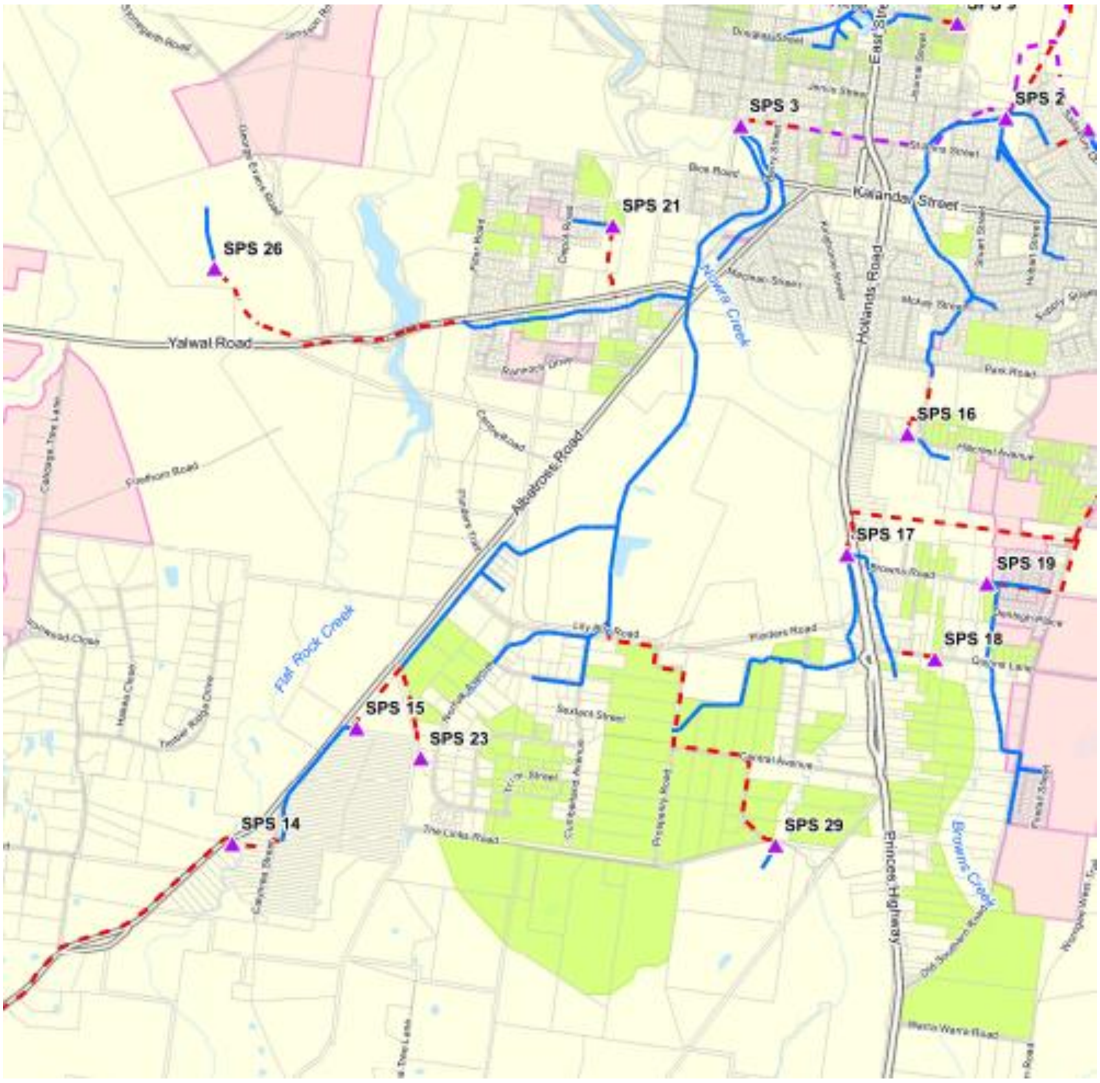

Figure 3.1: St Ann's St Catchment Overview
(Source: Shoalhaven Water, 2013)

3.1.1. Catchment Properties

The ET's for each catchment and catchment size is based on Shoalhaven Waters' records for 2011. Shoalhaven Water has adopted a figure of 2.37 EP / ET. It has been adopted that due to minimal development occurring in each of the sewer catchments that the ET's shall be fixed for the adopted analysis period of 3 years. Table 3.1 – Catchment details is a summary of the catchments population and area.

Table 3.1: Catchment Details

SPS Catchment	Equivalent Tenements (ET)	Equivalent Population (EP)	Catchment Area (ha)
3	1263	2993	199.7
15	154	365	68.6
21	234	555	29.6
23	37	88	22.1
26	9	21	2.9
29	253	600	3.0
3 (All)	1950	4622	325.9

3.1.2. Sewer Network

The sewer network consists of extensive gravity mains with upstream catchments connecting via rising mains from each of the respective SPS's. Within catchment 3 there is 27.4km of gravity mains ranging in size from 150mm up to 450mm with a short section of 600mm main connecting the network to SPS 3. Table 3.2 – Catchment 3 Gravity Pipeline Summary provides details of the type of pipe, size, age and lengths installed.

Approximately 17.4km of this network consists of either Asbestos Cement (AC) or Vitrified clay pipe (VCP). 90% of the gravity pipes have been operating for 25 or more years.

Table 3.2: Catchment 3 Gravity Pipeline Summary

Catchment 3	Pipe Size (mm)	Pipe Length (m) / Age (Yr's)										Total
		5	10	15	20	25	30	35	40	45	55	Length (m)
AC/C	150							65	1393	2307		3764
	225							406	528			934
	300							732	1064			1796
	450							498	2979			3478
PVCP	150			327	131	2340	2345	1472				6615
	225					32			404			436
UPVC	150	360	1552	1013					52			2977
VCP	150										6500	6500
	225										877	877
	600										15	15
DICL	150									50		50
Total (m)		360	1552	1340	131	2372	2345	3174	6419	2356	7392	27441

Details of the gravity pipelines for the upstream catchments are provided in Appendix F – SPS Gravity Pipeline Summary.

3.1.3. Upstream SPS Flow Transfer Times

The upstream sewer catchments transfer pumped flows to catchment 3 via rising mains. The velocity of flow in each rising main has been adopted from SW's records to enable an estimated time to be calculated for the flow to reach the gravity catchment. The length and grade of the gravity mains that transfer each of the pump flows to SPS 3 has been adopted from SW's records. A velocity of 1m/s has been adopted for the pumped flows within the gravity section. The basis for this is no hydraulic modelling of the network flows has been undertaken, however field measurements at various points in the network, during dry weather, indicate that this value is reasonable. Table 3.3 – SPS Flow Delay Times summaries the total delay for each SPS that has been adopted. It can be seen that the flows from SPS 21 arrive at SPS 3 after 30 minutes whilst the flows from the jail take approximately 115 minutes.

Table 3.3: **SPS Flow Delay Times**

SPS	15	21	23	26	29
M.H Discharge I.L	54.34	28.95	54.34	50.033	35.7
SPS Inlet I.L	13.425	13.425	13.425	13.425	13.425
Length Of Main	4005	1560	4005	2356	3202
Gravity Grade	1.02%	1.46%	1.02%	0.97%	0.83%
Velocity	1	1	1	1	1
Travel Time (Gravity)	4005	1560	4005	2356	3202
Rising Main Length	402.5	443	474	1484	1869
Velocity	0.6	1.7	1.7	0.5	0.5
Travel Time (Rising Main)	671	261	279	2968	3738
Total Time (s)	4676	1821	4284	5324	6940

These delay times are taken into consideration for the calculation of flows within catchment 3. Each delay time is rounded to the nearest 5 minute period for the purpose of developing the diurnal curve for catchment 3's gravity section.

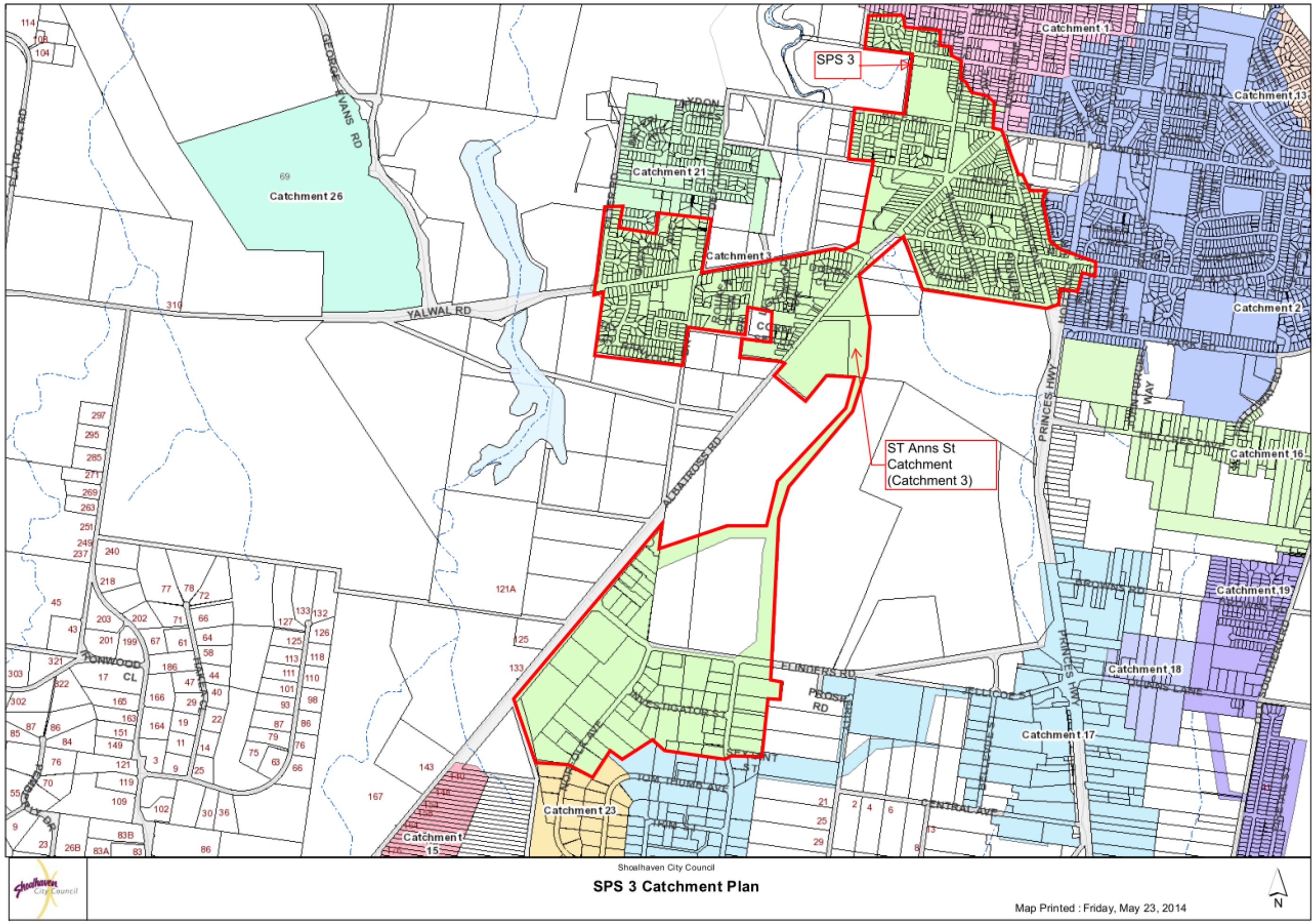

Figure 3.2: St Ann's St Gravity Catchment (Source: Shoalhaven Water GIS, 2014)

3.2. Project Overview

The intent of the project is to develop a strategy to be used throughout the Shoalhaven region to identify and rectify I/I issues. At present no overall strategy exists and different approaches have been utilised to at least address the impacts.

Whilst the project is not intended to solve the issue of inflow and infiltration it will provide the framework to develop a systematic approach to managing and maintaining the wastewater system. It will enable Shoalhaven Water to prioritise future works, including maintenance and capital programs, taking into account the severity and risk associated with the impacts of inflow and infiltration.

The project will use the following steps to develop the strategy

1. Utilise the literature review on the design guidelines from various states and use 3 common design methods to estimate PWWF.

2. Use the knowledge learnt from other studies to customize the I/I values that suit the local conditions.

3. A problematic catchment within Shoalhaven wastewater network (SPS 3 – Nowra) shall be investigated using baseline data of ADWF during dry days, rainfall events and corresponding PWWF for the catchment.

4. A methodology will be developed for field analysis of the problematic catchment.

5. A Trial investigation during a wet weather event (subject to wet weather) using the developed methodology in 4 shall be carried out.

6. The field results will be analysed to determine if the field methodology was successful in identifying the primary source of I/I causing pump station / manhole overflows during wet weather.

7. Rectification measures to mitigate inflow / infiltration will be undertaken if possible during the project timeframe.

8. Suggested improvements to the design guidelines based on the gather data and field investigation shall be made.

3.3. Conclusion

The intent of using a project site is to enable a review of the processes that need to be undertaken to identify I/I within a catchment. This will allow process improvements to be readily identified, as well as permitting a review of the different design guidelines to be undertaken. This review will allow recommendations to be made for improvements to Shoalhaven Water current sewer design code and help with establishing process improvements that need to be made to deal with the issue of excessive I/I into the wastewater system.

Chapter 4: Project Methodology

The project has been based on a 3 year period, from April 2011 to April 2014. This period has been adopted as a new pump station, SPS 29, was commissioned in July 2010 to service a correctional facility. Based on operational knowledge of SPS 29 the correctional facility was fully operational by January/February 2011.

4.1. Existing Flow

The existing daily flow profile from SPS 3 and the 5 contributing upstream catchments is calculated using the Shoalhaven Water historical records. These records are based on the SPS "Supervisory Control and Data Acquisition" (SCADA) system. These records contain the pump run times for each SPS (with time steps) and this data has been extracted from the system and exported into Excel. The average flow between pump runs is calculated using the pump flow rates from Shoalhaven Waters draw down test records. The drawdown tests were completed in 2012 by Shoalhaven Water staff.

For SPS 3 there are two pumps which operate as duty and standby, these pumps also operate at either low speed or high speed depending on the rate of incoming flow. The five upstream pump stations have single speed pumps that operate as either duty/standby or combined. The pumps are regularly rotated from duty to standby.

The details for each SPS pump performance and associated identifier is provided in Appendix G – SPS Pump Performance and Identifier.

4.2. Excel Processing

The SCADA data is imported to Excel to enable a flow file to be created. The average flow in a time period is calculated on the basis that inflow occurs from the time the pump stops through to the next time the pump stops after a pump run. The outflow is the volume of effluent pumped whilst the pump is running

$$\text{Average Flow} = \frac{\text{Pump Flow Rate x Time of Pumping}}{\text{Total Inflow Time}} \text{ (L/s)} \qquad (4.1)$$

Figure 4.1: Average Flow per Period of Time illustrates equation 4.1, the period of time that the pump is off / on constantly varies. When high inflow periods are experienced for SPSs with dual speeds the pumped flow increases accordingly as shown in inflow period 2 in Figure 4.1.

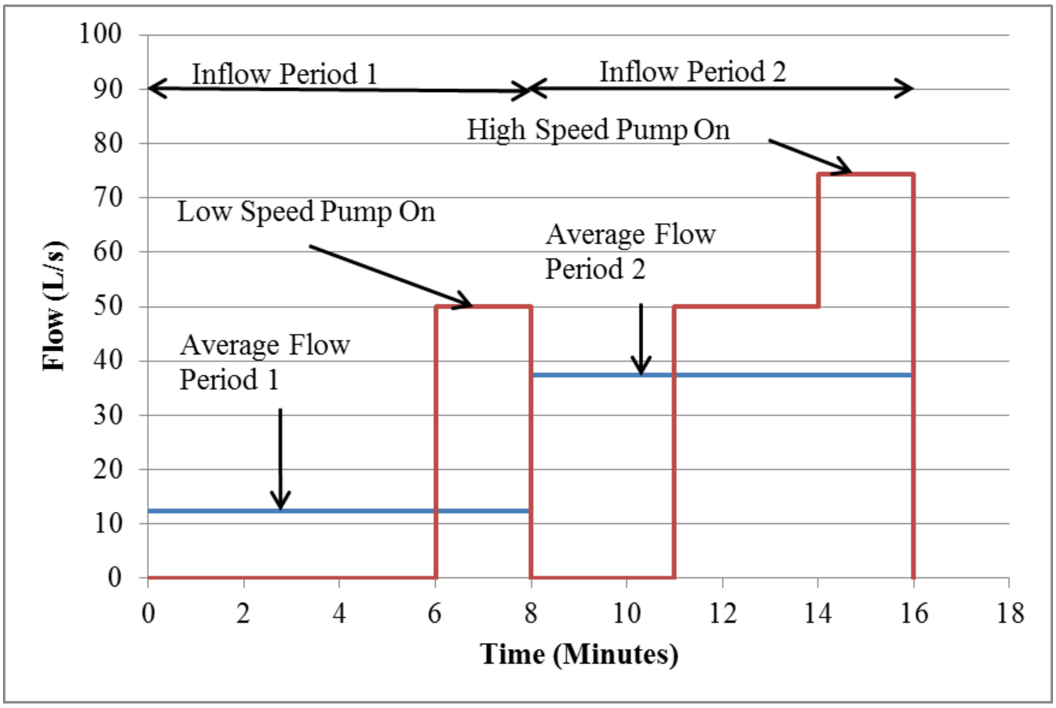

Figure 4.1: Average Flow per Period of Time

This calculation is repeated for the analysis period of 3 years for SPS 3 and the five upstream SPSs. The following values are then calculated for each day.

- Average Day Flow (ADF),
- Peak Day Flow (PDF),
- Minimum Flow (MF).

Using the calculated values for all SPSs the gravity catchment flows for SPS 3 are as follows;

ADF Catchment 3 = ADF (SPS 3) – ADF (SPS 15) – ADF (SPS 21) – ADF (SPS 23) – ADF (SPS 26) – ADF (SPS 29) (4.2)

PDF Catchment 3 = PDF (SPS 3) – PDF (SPS 15) – PDF (SPS 21) – PDF (SPS 23) – PDF (SPS 26) – PDF (SPS 29) (4.3)

A check of the time period when each peak day flow occurs at each SPS will need to be taken to ensure that the peaks occur during a similar time period. This will also need to take into account an approximate travel time for the flows to reach SPS 3.

MF Catchment 3 = MF (SPS 3) – MF (SPS 15) – MF (SPS 21) – MF (SPS 23) – MF (SPS 26) – MF (SPS 29) (4.4)

The minimum flow should occur during the period of 12am to 4am; this is checked for each upstream SPS as they have various industries discharging into them. The minimum flow indicates the level of groundwater infiltration into the gravity system.

For each SPS an Excel file is created as shown in Table 4.1: Excel SCADA Data Format. The data is imported to Excel from the SCADA historical records. The raw SCADA data has 3 values being date/time stamp, Pump State (1 = On, 0 = Off) and pump identifier i.e. PS29P1 relates to SPS 29 Pump 1.

Table 4.1: Excel SCADA Data Format

Date Time Stamp	State	Pump	Check 1	Check 2	Time (s)	Total Inflow Time (s)	Pump Outflow Time (s)	Average Flow Time Start / Finish	Flow (L/s)
8/03/2011 21:36	0	PS29P2	Pump OK	Ok				8/03/2011 21:36:12	0.9
8/03/2011 22:32	1	PS29P1	Pump OK	Ok	3369	3793	424	8/03/2011 21:36:12	0.7
8/03/2011 22:39	0	PS29P1	Pump OK	Ok	424			8/03/2011 22:39:25	0.7
8/03/2011 23:58	1	PS29P2	Pump OK	Ok	4768	5157	389	8/03/2011 22:39:25	0.5
9/03/2011 0:05	0	PS29P2	Pump OK	Ok	389			9/03/2011 0:05:22	0.5
9/03/2011 3:09	1	PS29P1	Pump OK	Ok	11053	11443	390	9/03/2011 0:05:22	0.2
9/03/2011 3:16	0	PS29P1	Pump OK	Ok	390			9/03/2011 3:16:05	0.2
9/03/2011 8:23	1	PS29P2	Pump OK	Ok	18456	18879	423	9/03/2011 3:16:05	0.2
9/03/2011 8:30	0	PS29P2	Pump OK	Ok	423			9/03/2011 8:30:44	0.2

The spread sheet is then programmed to check the data; check 1 ensures that the pump state changes from on to off to on. Check 2 ensures that if pump 1 is on the next state change is pump 1 off.

The time between state changes is calculated along with the total inflow time and the time that the pump is running. This enables the average flow per period of time to be calculated in accordance with equation 4.1. For example in Table 4.1: Excel SCADA Data Format the flow changes from 0.9 L/s to 0.7 L/s at 21:36:12 and changes flow again at 22:39:25 from 0.7 L/s to 0.5 L/s. Figure 4.2: SPS SCADA Diurnal Curve is the graphical representation of the processed SCADA data for SPS 3. The flows shown are the average per inflow period.

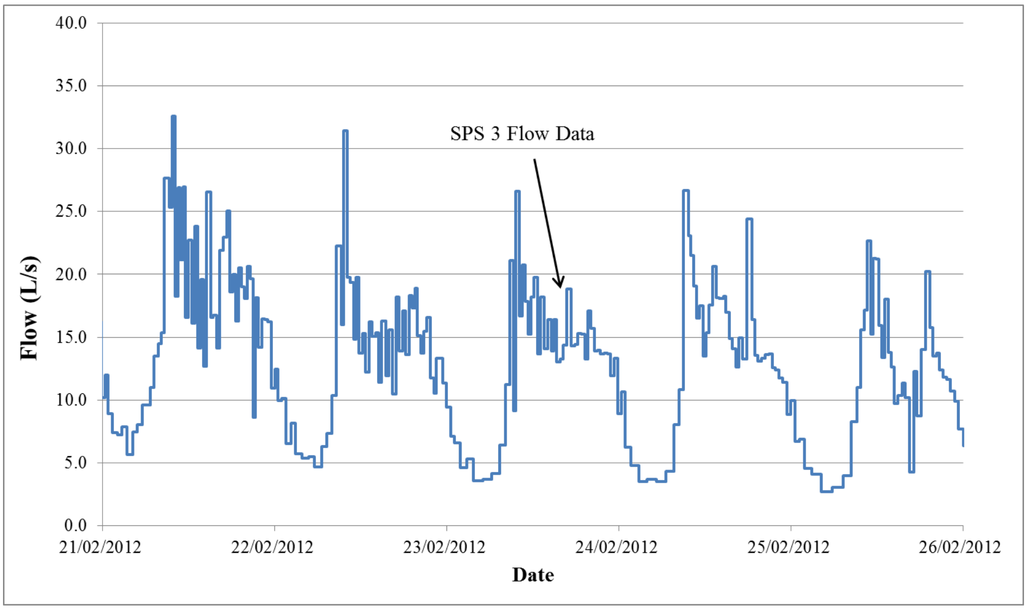

Figure 4.2: **SPS SCADA Diurnal Curve**

A visual check of the data is undertaken by graphing the flow for the entire period for each of the SPSs. The visual check enables periods of suspect data to be identified and more closely examined. Figure 4.3: SPS 3 SCADA Flow Data shows a period of suspect data, 2 smaller periods to the right of this suspect period were related to wet weather events. The suspect period is then investigated further to determine if the data was corrupted.

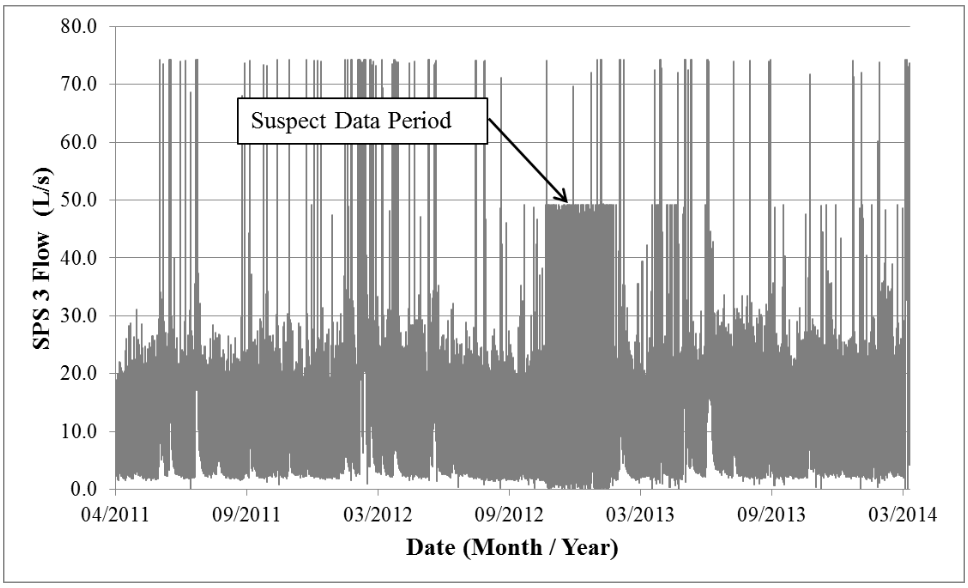

Figure 4.3: **SPS 3 SCADA Flow Data**

Figure 4.4: SPS 3 SCADA Data Exclusion shows that the flow during this period regularly reached the maximum flow of the pump at low speed and did not follow a consistent pattern as evident on either side of the suspect data. The days during this period are excluded from the data set as they would bias the peak daily flow and subsequent peaking factor.

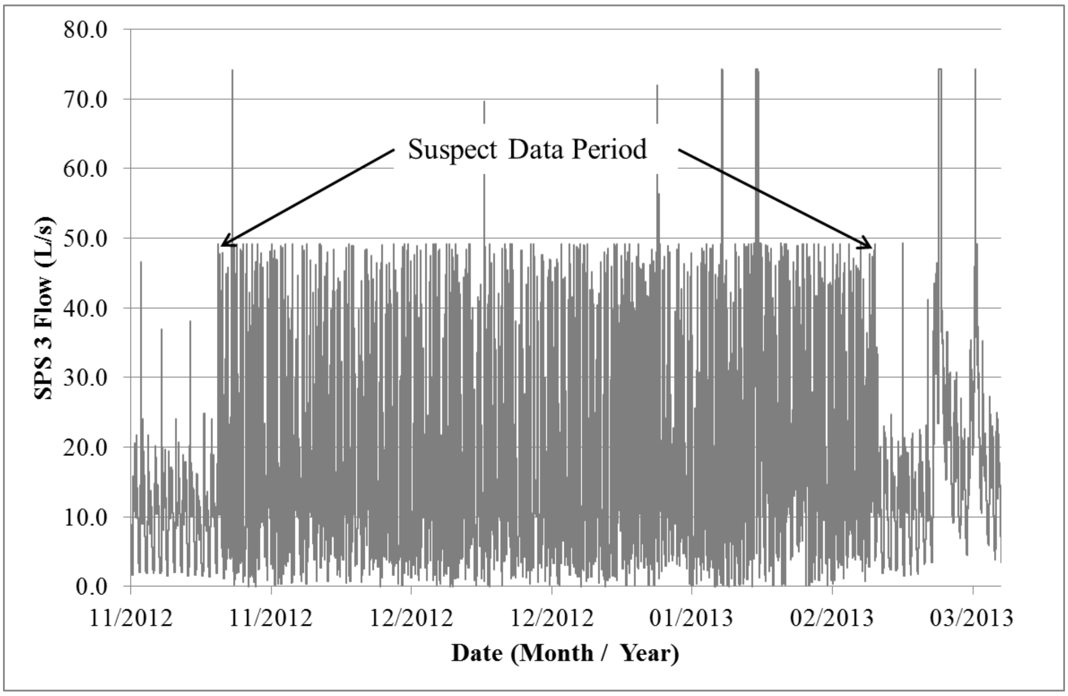

Figure 4.4: **SPS 3 SCADA Data Exclusion**

Appendix H: SPS SCADA Graphs has the results for SPS 15, 21, 23, 26 and 29.

4.3. Dry Weather Criteria

In order to calculate the ADWF for each SPS, rainfall parameters were adopted to classify each day as either dry or wet. It was adopted that rainfall up to 5mm in total for the 7 day period prior to a dry day would have minimal impact. Based on the dry day calculation it was established that there were 151 dry weekdays and 69 dry weekend days in the analysis period. This equated to approximately 25% of the days during the 3 year period were classified as dry days. Appendix I: Rainfall Data is the tabulation of the daily rainfall data used for the classification of each day as dry / wet.

4.4. Maximum Inflow Time

The inflow time, the time period between pump runs, for each SPS was checked for gross errors. It was adopted that the maximum inflow period should occur on a typical dry day when flow is a minimum.

From these dry days a random number generator was used in Excel and 16 dry weekdays and 16 dry weekend days were selected. The maximum inflow time for each random day is used to calculate the average and standard deviation of maximum inflow times for each SPS.

Days which had inflow periods greater than 3 standard deviations from the average were deemed not suitable. For inflow data that exceeded 24hrs from the previous day were also removed from the data set.

For SPS 3 days which were rejected for upstream SPSs are also rejected from SPS 3 data set. The basis for this is that peak flows may have been biased as a result of flows being retained for an extended period of time and thus not reflecting the catchments typical diurnal flow.

4.5. Dry Day Flow

The data set is used to determine the ADWF, PDWF and minimum flow on days classified as dry days. The data set is refined by removing the days of suspect data and those days that have exceeded the maximum allowable inflow time periods. The values are calculated for weekdays/ weekends for each month, annually and for the entire analysis period. This process is used to identify changes/trends in the wastewater discharge from the catchments over an extended period of time and to also determine if a peak month/season exists.

For the development of diurnal curves a method recently published by WSAA (WSAA, 2013) was utilised. This method uses a visual analysis of the 5 minute flow for each day. Days which do not follow the usual trend are removed from the data set. The ADWF, PDWF and minimal flows are calculated using the WSAA method to provide a reality check on the statistical method being used. Figure 4.5: WSAA Flow Analysis is an example of the graphical representation used to determine the

ADWF, PDWF and minimum flow. It also enables a flow hydrograph of a wet day to be easily produced.

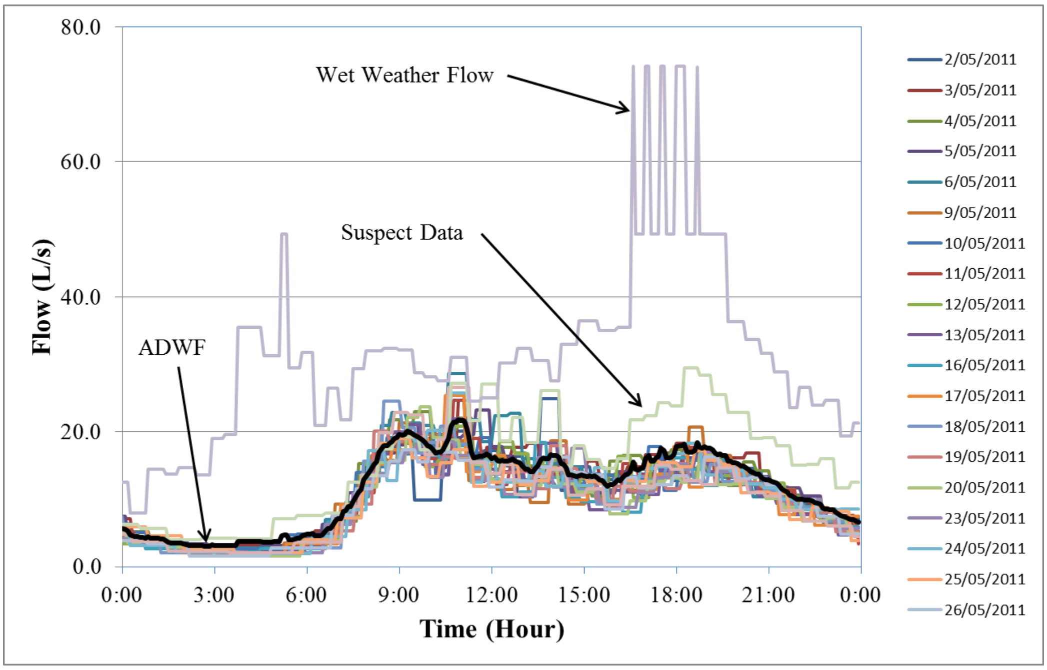

Figure 4.5: **WSAA Flow Analysis**

An average annual diurnal curve for both weekday and weekend is calculated using the 5 minute flow data for all SPS's.

4.6. Peak Wet Weather Flow

Rainfall data (5 minute increment) from Bureau of Meteorology (BOM) Nowra RAN rain gauge is used for the analysis period to rank the storm events and determine the ARI of each storm event using the Intensity Frequency Duration (IFD) charts. The recently revised 2013 IFD charts are adopted as suitable as the BOM site advises that the South Coast of NSW has extensive historical rainfall records.

Using the WSAA method the PWWF is calculated for the project site. This is then compared to the Shoalhaven Water and Queensland's Traditional PWWF empirical methods. This also provides a reality check to determine if the system is operating within acceptable design limits i.e. are the overflow events a result of system capacity already being exceeded.

4.7. Field Analysis Methodology

The development of the field analysis methodology is completed at the early stages of the project and then refined as the method is trial. It involves the following work

- Preparing a plan of the catchment and dividing the catchment into even portions,
- Selecting suitable manholes to measure the flow during rain events,
- Calculating the expected ADWF/PWWF based on upstream EP's,
- Undertaking a dry day run to ensure that manholes are accessible and also to determine the time needed to complete a system run,
- Measuring the depth and velocity, of flows, in manholes whilst rainfall is occurring,
- Record the results as the field work progresses.

The flows in the manhole shall be measured using a "Flow Probe". This device enables the depth to be estimated and the velocity of the flow to be measured. As the size of the pipe is known it is possible to determine the flow (L/s) at the time of measurement.

4.8. Field Analysis

The field analysis is highly dependent on wet weather events. As wet weather can result in unsafe workplace conditions, this work will only be undertaken during daylight hours. The other reason for this is access to private properties; home owners will not want people turning up at all hours of the night. The intent of the field work is to detect sections of the network which are heavily impacted upon by I/I, this should reduce the catchment portions down to small areas i.e. between manholes.

The extent of the storm event will also need to considered, the intent is to undertake field analysis for a minimum of 4 storm events whilst rainfall is occurring. This part of the project is highly reliant on the nature of each storm event i.e. intensity and duration of each storm. It will help to determine the minimum storm size required for field work, i.e. analysis during small storms may not yield results.

As noted in the field analysis methodology it is intended that once the field analysis has been undertaken for the first wet weather event that the scope of work can be refined to suspect parts of the catchment only i.e. between manholes.

The results of the field analysis will be reviewed to determine the success of the method used and determine what types of further detection methods need to be used i.e. CCTV of network system and private laterals, smoke testing and/or dye testing of internal household connections.

4.9. Rectification methods to mitigate I/I

The WSAA Good Practice Guideline provides a number of mitigation methods currently used and the success of each type of method. The various methods to be considered will be reviewed along with the additional work/resources required to complete mitigation measures.

These measures will be part of the strategy, whilst one measure may be suitable in this instance for I/I mitigation a different measure may be more suitable in another catchment i.e. the methods are each reliant on the type /extent of the problem.

4.10. Improvements to the Design guidelines based on data / field investigations

Upon completion of the desk top and field analysis, recommendations will be made to improvements that can be made to the process of detection and rectification of I/I. These recommendations will suggest improvements that can be made to the Shoalhaven Water design guidelines by using local climatic conditions and customisation of I/I values for the region. It is also anticipated that due to the size of the region that the development of customised I/I values will need to be undertaken for each sewerage system.

4.11. Conclusion

The intent of the project methodology is to go through the process needed to determine the PWWF for catchment 3 and to also compare the results from using different design methods. Field work is being undertaken to develop a connection between the results and actual visual interpretation. It will allow an appreciation for the scale of the issue of I/I into the sewerage system.

The overall aim of the project is to develop a strategy which provides an overview of the work required to enable the successful identification of rectification of I/I into the sewerage system.

Chapter 5: Results

5.1.Review of SCADA

The historical records that are kept in the SCADA database provide valuable information for reviewing the performance of the sewer network. The current format of the data does not permit analysis of the data to be undertaken in an efficient and effective manner. Whilst the system performance of individual or multiple SPSs should be able to be measured and monitored with minimal effort the SW system is not currently structured for engineering or management needs.

SCADA Inefficiencies

Extensive periods of time are required to extract data and undertake quality checks for each individual SPS. Based on the 3 year analysis period for the 6 SPSs the data extraction and conversion to a daily flow profile required up to 20 hours per SPS to achieve. This restriction does not permit the system performance to be regularly monitored for changes or inconsistencies.

In addition to the above the system functionality is very poor. Individual variables require processing to produce usable information. For instance the extraction of data for SPS 3 required the pump on and off times for each pump and for high and low speeds to be collated within excel. Extensive data manipulation was then required to produce a usable flow file.

The SCADA historical records do have a reporting function; this function is meant to provide summaries of daily flow. The current historical reports are only available in PDF format and were highly inaccurate. The reports rely on the daily pump run times however they do not take into account the high or low pump speeds. For SPS 3 on a typical dry day the pumps will operate from 50 to 90% of the time at low speed, the remaining time is at high speed.

With respect to SPS 3 the daily reports indicate that 1.25ML of flow was transferred on the 16th August 2014 however only 1.04ML was actually pumped. This was a direct result of the reporting function using the high speed flow rate in its calculation.

SPS 3 also operates with a variable speed control (VSD) the VSD controls the speed at which the pump operates. When the pump switches on the pump takes 5 to 10 seconds to reach low speed status i.e. 49L/s, the same occurs after the pump has switched off. The difference in actual flow pumped as a result of the VSD operating was not taken into account for the purpose of this analysis. The basis for this is I was not aware that the SCADA system did not accommodate VSD operation until the later stages of the report. The effect of not taking into account the operation of a VSD is the calculated ADWF are slightly higher, approximately 3%, than actual.

Further investigations into the cause of the above noted inaccuracies were undertaken. The cause mainly related to incomplete programming of the SCADA system, this results in engineering and management requirements not being satisfactorily provided.

SCADA Visual Checks

The visual check of the SCADA data resulted in an extended period of time from the 15th November 2012 to the 14th February 2013 not being of suitable quality for SPS 3, 15, 21, 26 and 29. A gap in the data for SPS 23 was also evident for the period from the 30th June 2013 to 7th July 2013. This was a result of the transmission aerial being stolen. The graphs for the visual checking of the SCADA data is provided in Appendix H: SPS SCADA Graphs.

SCADA Data Quality Checking

The current SCADA system does not record periods of prolonged down time which may be a result of pump failure, maintenance etc. As part of the quality checking of the data the maximum inflow time was calculated for the 16 random dry days used in the dry flow analysis. It was adopted that a dry day would have the longest period between pump runs. The 3 times standard deviation was used to detect suspect days, any day that had an inflow time greater than the calculated value was rejected.

Based on the maximum allowable inflow time for each SPS it was determined that each SPS had a different maximum allowable inflow time. SPS 3 and 21 both had relative short maximum inflow times whilst SPS 23 and 26 had long maximum inflow times especially for the weekends. This reflects the catchments land usage with large residential components in SPS 3 and 21 catchments whilst SPS 23 is light industry and SPS 26 a relatively small university campus. There was little variation between weekday and weekend maximum inflow time for SPS 3, 21 and 29.

Table 5.1: Maximum Inflow Time summaries the results of the analysis. For SPS 3 the maximum inflow time was 3.1 hours for weekdays and 2.9 hours for weekends.

Table 5.1: Maximum Inflow Times

SPS No.	Maximum Inflow Time						Rejected Days		
	Weekday		Weekend		Adopted		No. of Days		
	Seconds	Hours	Seconds	Hours	Seconds	Hours	Weekday	Weekend	Total
3	11216	3.1	10306	2.9	10306	2.9	10	7	17
15	21551	6.0	23176	6.4	21551	6.0	6	2	8
21	7412	2.1	7029	2.0	7029	2.0	10	7	17
23	71643	19.9	107983	30.0	71643	19.9	23	14	37
26	60793	16.9	114159	31.7	60793	16.9	4	11	15
29	17751	4.9	19465	5.4	17751	4.9	4	2	6

A long period of inflow with no pumping can be a result of the SPS operation being inhibited as a result of maintenance, emergency repair work or a wet weather event. The impact of overflow at SPS 3 is reduced by inhibiting the operation of upstream SPSs. For SPS 3 the rejected days removed from the data set also included those of the upstream SPSs. These days were removed as they may impact on the PDWF or minimum flow as a result of the built up flow being released in a short period of time.

There were also a number of days which were common to several SPSs. The tabulation of the analysis for maximum inflow time and the list of days for each SPS that has been rejected from the data sets are provided in Appendix J: Maximum Inflow Time.

5.2. Dry Weather Flow

SPS 3 had 112 weekdays and 50 weekend days over the entire analysis period that were deemed suitable based on the dry day criteria. The results indicate that there was minimal change between period 1 and 2 and an increase in ADWF and PDWF in period 3. The weekdays displayed higher values for ADWF and PDWF whilst the weekends had a higher peaking factor and minimum flow.

Table 5.2: SPS 3 Dry Day Flow summaries the results, the results for the upstream SPSs are provided in Appendix K – Dry Day Flow.

Table 5.2: SPS 3 Dry Day Flow

SPS 3								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	10.8	9.73	10.9	9.99	12.4	10.7	11.4	10.2
Std Dev (L/s)	0.86	0.72	1.29	1.02	2.01	0.96	1.56	1.00
3 x Std Dev (L/s)	13.4	11.9	14.8	13.1	18.5	13.5	16.1	13.2
PDWF (L/s)	23.6	22.9	24.1	24.2	26.9	25.6	25.0	24.4
Std Dev (L/s)	2.89	2.48	3.83	3.04	3.95	3.20	3.81	3.14
3 x Std Dev (L/s)	32.2	30.4	35.6	33.4	38.8	35.2	36.4	33.8
Peaking Factor	2.19	2.36	2.20	2.43	2.17	2.40	2.18	2.40
Minimum Flow (L/s)	2.27	2.43	2.15	2.53	2.34	2.71	2.26	2.57
Std Dev (L/s)	0.53	0.51	0.69	0.46	0.57	0.77	0.61	0.62
3 x Std Dev (L/s)	3.87	3.97	4.23	3.92	4.05	5.03	4.09	4.44
No. of Dry Days	33	15	37	16	42	19	112	50
No. of Wet Days	211	81	137	49	191	69	539	199

The limited number of dry days did not provide a suitable range of data to determine the peak month of each period. It was adopted that the calculated 3 times standard deviation values for ADWF, PDWF and minimum flow for would be used as the criteria to re-evaluate the entire data set.

The new criteria was applied to the data set and resulted in a total of 463 weekdays and 188 weekend days being deemed suitable for analysis of the design parameters. The results for SPS 3 are summarised in Table 5.3: SPS 3 Dry Day Flow Expanded Data Set.

The ADWF, PDWF were similar to the previous data set with the majority of the values having a lower standard deviation. There was a slight increase in the minimum flow values with an increase of 0.3L/s for weekdays and 0.08L/s for weekends over the entire analysis period. This data was adopted as being suitable to determine the peak month of each period.

Table 5.3: SPS 3 Dry Day Flow Expanded Data Set

SPS 3								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	10.8	9.55	11.1	9.99	12.0	10.8	11.3	10.1
Std Dev (L/s)	0.92	0.67	1.11	0.97	1.44	1.02	1.21	1.00
PDWF (L/s)	23.5	22.6	23.6	23.7	25.8	26.0	24.3	24.0
Std Dev (L/s)	3.12	2.56	3.48	3.23	3.66	3.42	3.51	3.31
Peaking Factor	2.17	2.36	2.13	2.37	2.15	2.41	2.14	2.38
Minimum Flow (L/s)	2.58	2.65	2.40	2.53	2.56	2.76	2.54	2.65
Std Dev (L/s)	0.56	0.56	0.65	0.51	0.61	0.68	0.64	0.58
No. of Dry Days	160	68	129	53	150	68	463	188
No. of Wet Days	84	28	45	12	83	20	188	61

Similar trends were also present for the analysis of the upstream SPSs with approximately 75% of days classified as "Dry Days". In addition the expanded data sets showed minimal variation when compared against the original data sets. The result of the upstream SPSs flow analysis based on the adopted revised criteria is provided in Appendix K: Dry Day Flow.

5.3.Peak Month Average Dry Weather Flow

Within the SPS 3 catchment no consistent peak month was detected for the 3 periods. No month was the peak month for all three periods. February and July were the peak weekday months and June was the peak weekend month based on the entire analysis period. The ADWF for September was consistently below the average for both weekday and weekends in all periods. Based on season the winter period has the highest ADWF in 2 of the annual periods, although August in period 2 has an ADWF below the annual average for that period.

Table 5.4: SPS 3 Monthly ADWF, summaries the calculated ADWF for each period and the entire data set for both weekdays and weekends.

Table 5.4: SPS 3 Monthly ADWF

SPS 3 (L/s)								
Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	10.5	11.7	10.7	10.8	8.9	10.3	9.7	9.6
May	10.4	12.3	11.7	11.6	9.5	10.8	9.7	10.1
June	10.9	12.0	11.7	11.6	10.1	10.4	10.8	10.7
July	11.5	11.6	12.9	12.0	9.8	10.8	11.7	10.6
August	10.9	10.6	12.5	11.3	9.8	9.5	11.9	10.4
September	10.5	10.0	11.8	10.8	9.1	9.2	10.9	9.5
October	11.5	10.1	11.2	11.0	9.9	9.9	10.1	10.1
November	11.0	9.9	12.4	11.4	9.8	8.8	11.4	10.2
December	10.8		12.5	11.4	9.6		10.7	10.1
January	10.2		11.2	10.7	9.2		10.1	9.6
February	11.5	10.6	13.1	12.0	9.6	10.3	10.6	9.9
March	12.8	11.3	12.9	12.0	10.3	9.0	10.4	10.0
Annual	10.8	11.1	12.0	11.3	9.6	10.0	10.8	10.1

A visual representation of the weekday results is provided in Figure 5.1: SPS 3 Weekday Monthly ADWF. It appears that the ADWF follows a general trend with a dramatic change evident for November and December between period 1 and 3. Unfortunately the data for these months in period 2 was not suitable for analysis.

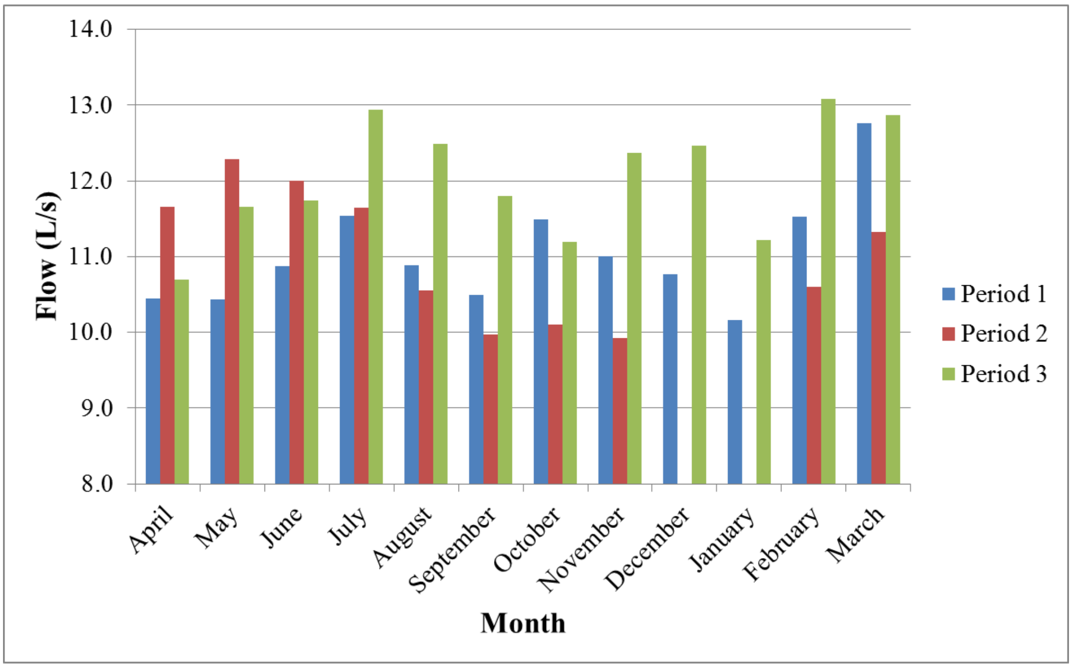

Figure 5.1: **SPS 3 Monthly Comparison of Weekday ADWF**

It can be seen in Figure 5.1 that the ADWF varies considerable throughout the year. In period 1 the ADWF varied from 10.2 L/s to 12.8 L/s in the space of 2 months.

Similar variations were also detected in period 2 and 3 with difference between the maximum and minimum months being 2.4 L/s for both periods.

5.4. Comparison of Statistical Dry Day Method and WSAA Method

A comparison of the ADWF using the statistical dry day method and the method recommended by WSAA (WSAA, 2013) resulted in the ADWF for the majority of periods being similar between the 2 methods. Table 5.5: Comparison of ADWF methods provides a summary of monthly ADWF calculated for each period using both methods. Data for December and January (period 2) was deemed not suitable using both methods.

Table 5.5: Comparison of ADWF methods

SPS 3	ADWF (L/s) Period 1		ADWF (L/s) Period 2		ADWF (L/s) Period 3	
	Weekday		Weekday		Weekday	
Month	Statistical	WSAA	Statistical	WSAA	Statistical	WSAA
April	10.5	10.5	11.7	11.6	10.7	10.7
May	10.4	10.4	12.3	12.3	11.7	11.5
June	10.9	10.9	12.0	12.0	11.7	11.5
July	11.5	11.5	11.6	11.6	12.9	12.9
August	10.9	10.9	10.6	11.2	12.5	12.5
September	10.5	10.5	10.0	9.9	11.8	11.8
October	11.5	11.5	10.1	10.1	11.2	11.3
November	11.0	11.0	9.9	10.0	12.4	12.4
December	10.8	10.8			12.5	12.1
January	10.2	10.2			11.2	11.3
February	11.5	11.5	10.6	10.6	13.1	12.2
March	12.8	12.8	11.3	11.3	12.9	11.9
Annual	10.8	10.8	11.1	11.2	12.0	11.8
Total Days	160	163	129	137	166	159

On an annual basis there was a variation of 0.1 L/s for period 2 and 0.2 L/s for period 3. The largest variation was for February in period 3 with the WSAA ADWF 0.9L/s higher than calculated using the statistical method.

This WSAA method enabled more days to be included in the WSAA data set for February (period 3) but an overall reduction of 7 days for the entire period.

The WSAA method had advantages to the traditional approach these advantages include:

- Visual representation of the daily, monthly and annual diurnal curve,
- Ease of identification of periods of extensive inflow and suspect data,
- Identification of period of high flow.

Both methods resulted in sets of data followed similar trend patterns with no consistent peak month evident. The WSAA method also provides the ability to compare inline gravity flow monitoring results with the derived 5 minute flows based on the SCADA records.

5.5. Peak Day Average Dry Weather Flow

The Peak Day ADWF was calculated by adding 3 standard deviations to the average monthly ADWF. A comparison of both the statistical method and WSAA method was completed for SPS 3 period 3. The results are tabulated in Table 5.6: Peak Day ADWF SPS 3.

Table 5.6: Peak Day ADWF SPS 3

Period 3	Statistical (L/s)			WSAA (L/s)		
Month	ADWF	Std Dev (1)	Peak ADWF	ADWF	Std Dev	Peak ADWF
April	10.7	0.46	12.1	10.7	0.46	12.1
May	11.7	1.29	15.5	11.5	1.28	15.4
June	11.7	0.59	13.5	11.5	0.57	13.3
July	12.9	0.67	14.9	12.9	0.66	14.9
August	12.5	0.73	14.7	12.5	0.73	14.7
September	11.8	0.76	14.1	11.8	0.76	14.1
October	11.2	0.68	13.2	11.3	0.78	13.6
November	12.4	0.54	14.0	12.4	0.56	14.1
December	12.5	2.11	18.8	12.1	1.64	17.0
January	11.2	0.78	13.5	11.3	0.62	13.2
February	13.1	2.52	20.6	12.2	1.44	16.6
March	12.9	2.28	19.7	11.9	0.49	13.3
Annual	12.0	1.45	16.4	11.8	1.04	15.0

The statistical method resulted in a peak day ADWF of 20.6 L/s (February) whilst the WSAA methods resulted in a peak day ADWF of 17.0 L/s (December). The annual results are 16.4 L/s (statistical) and 15.0 L/s (WSAA). The minimum peak day was April for both methods with an ADWF of 12.1L/s which is similar to the annual ADWF of 12.0 L/s calculated using the statistical method. The summer months of December and February both had high peak values whilst the month of January was one of the lowest peak values.

5.5.1. Calculation of L/EP/Day

The different land uses within each catchment resulted in a variation to the actual discharge per person per day. Similar trends were detected within each catchment for the 3 periods in the analysis; these trends are shown in Table 5.7: ADWF Weekday / Weekend Summary.

Table 5.7: ADWF Weekday / Weekend Summary

Weekday									
Catchment	ET's	EP's	Type	Period 1		Period 2		Period 3	
				ADWF	L/EP/day	ADWF	L/EP/day	ADWF	L/EP/day
3 (All)	1950	4622	Weekday	10.8	202	11.1	208	12.0	224
			Weekend	9.6	179	10	187	10.8	202
15	154	365	Weekday	0.48	114	0.5	118	0.41	97
			Weekend	0.51	121	0.54	128	0.42	99
21	234	555	Weekday	1.13	176	1.29	201	1.28	199
			Weekend	1.2	187	1.35	210	1.38	215
23	37	88	Weekday	0.65	640	0.45	443	0.38	374
			Weekend	0.43	424	0.14	138	0.16	158
26	9	21	Weekday	0.07	284	0.2	810	0.15	608
			Weekend	0.06	243	0.14	567	0.2	810
29	253	600	Weekday	1.39	200	2.14	308	2.62	378
			Weekend	1.24	179	2.1	303	2.55	367
3 (Gravity)	1263	2993	Weekday	7.08	204	6.52	188	6.96	201
			Weekend	6.16	178	5.73	165	6.09	176

The results indicate that the ADWF in the gravity portion of the catchment has had minor variation over the 3 year period. In comparison to the SW adopted value of 180L/EP/day (residential), the gravity portion of the catchment 3 had similar values for weekends whilst the weekday flows are approximately 10% higher. The catchment does have commercial / industry discharges which contribute to the average L/EP/day.

The peak ADWF value of 20.6L/s in Table 5.6 equates to a wastewater discharge of 385L/EP/day. This demonstrates the degree of variation to wastewater discharge that can occur throughout the year.

The largest increase over the 3 year period was for SPS 29 which services the South Coast Correctional Facility. The facility has similar wastewater discharge for both weekdays (378L/EP/day) and weekends (367 L/EP/day), this is to be expected as the residents are full time occupants. The increase in the wastewater discharge from SPS 29 appears to be the major contributor to the increase in the total flows to SPS 3, with the flows increasing from 1.39L/s in period 1 to 2.62 L/s in period 3. The correctional facility also undertakes commercial / light industry work which may contribute to the higher waste discharge per person.

Whilst the university campus does have high water usage per person this may be a result of the EP's not being correct. It is anticipated that more than 21 students / teachers attend this campus and the adopted figure is based on an annual average.

SPS 15 and 21 both have industry land uses within the catchment; the low figure for SPS 15 may be a result of over estimation of the EP's for the industry component whilst for SPS 21 the figures are similar to that of SPS 3. The one difference between SPS 3 gravity catchment and SPS 21 is a reversal of the higher discharge between weekday / weekend. For SPS 21 the weekend has the higher discharge compared to SPS 3 gravity whereby the weekdays are higher than the weekend.

5.5.2. Industrial Wastewater Discharges

South Nowra has two industrial premises that contribute significant volumes of wastewater to the system. One of these facilities is for cheese manufacture and it discharges directly to the gravity portion of SPS 3. The second facility is a liquid treatment facility; its discharge point is located within the SPS 23 catchment. Shoalhaven Water has limited records of the volume, frequency and rate of discharge of these facilities. Based on the records it is known that the cheese manufacturing facility is licenced to discharge up to 2L/s however the frequency and duration are not known. Field observations during working hours indicated a constant flow however the facilities total daily operational discharge hours are unknown.

The liquid treatment facility uses rainwater in the treatment process thus metered water usage does not provide an indication of actual discharge. Analysis of the SPS data indicated that up to 20kL of wastewater is discharged in a short period of time (approximately 20 minutes). This peak discharge occurrence was evident on several visits to SPS 23. The regularity of the discharge is not consistent, a the review of the SPS SCADA historical results indicated that this volume of discharge occurs several times per week and at times several times per day. Data from a recently installed flow weir was used to determine the quantity of daily discharge from the facility

Figure 5.2: SPS 23 versus Liquid Treatment Daily Discharge illustrates the quantity of wastewater that the facility contributes towards the total catchments contribution.

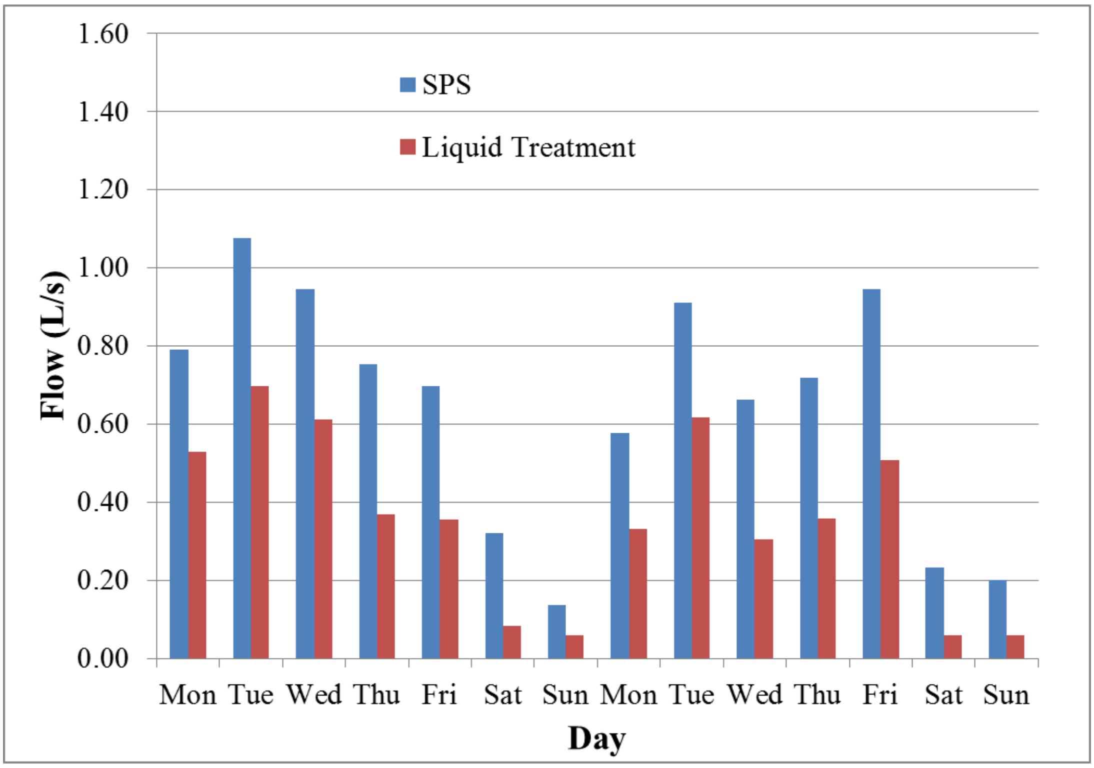

Figure 5.2: **SPS 23 versus Liquid Treatment Daily Discharge**

The treatment facility contributes at least 50% to the total daily flow at SPS 23. The impact of this facility is that the total discharge per EP in the light industrial estate is inflated. The results indicate that during weekdays, in period 3, 374 L/EP was the average discharge for the industrial estate. This highlights the importance of assessing commercial / industrial premises using customised EP values as provided in Appendix D: Sewer Design Code Equivalent Populations for Synchronous Discharges (WSA, 2002).

5.6. Peak Dry Weather Flow

The PDWF for SPS 3 catchment has been determine using the SCADA data and the results are provided Table 5.3: SPS 3 Dry Day Flow Expanded Data Set. Based on the results from the statistical method the average PDWF was calculated as being 25.8L/s (weekday) and 26 L/s (weekend) for period 3. The standard deviation of the peaking factor was calculated as 3.66 L/s (weekdays) and 3.42 L/s (weekends). This indicates that the peak dry weather flow could be as high as 36.8L/s (weekdays) and 36.3 L/s (weekends) for 3 standard deviations. The results of the calculated peaking factors for the upstream SPSs are provided in Appendix K –Dry Day Flow.

The peaking factor was calculated for each of the periods by dividing the average peak dry weather flow by the ADWF. The maximum peaking factor was calculated by dividing the maximum statistical peak dry weather flow (3 standard deviations) by the ADWF. Table 5.8: Peaking Factors is a summary of calculated peaking factors for period 3 for all of the SPSs and provides a comparison to 3 design methods.

Table 5.8: Peaking Factors

Period 3 - Weekday	SPS 3	SPS 15	SPS 21	SPS 23	SPS 26	SPS 29
ADWF (L/s)	12	0.41	1.28	0.49	0.15	2.62
PDWF (L/s)	25.8	1.2	3.23	3.18	0.34	5.5
Std Dev (PDWF)	3.66	0.66	0.61	3.96	0.31	0.84
Maximum PDWF (L/s)	36.8	3.18	5.06	15.1	1.27	8.02
Average Peaking Factor	2.15	2.93	2.52	6.49	2.27	2.10
Maximum Peaking Factor	3.07	7.76	3.95	30.7	8.47	3.06
WSAA (d)	2.08	2.67	3.19	3.41	5.70	5.65
Public Works (r)	2.11	3.03	2.84	3.87	5.00	2.80
Qld (C)	1.94	2.53	2.42	2.94	3.41	2.40

For SPS 3 and 15 the average peaking factor was similar to the factor calculated using the public works method. In comparison the peaking factor for SPS 21 was close to the value calculated using the Queensland method. It should also be noted that instantaneous peak discharge from premises may be higher; the SPS storage acts as a buffer and thus reducing the actual peak discharge. The maximum flow rate of the pumps also reduces this impact however may result in longer pump times whilst the storage is emptied.

The most significant peaking factor was for SPS 23, this site has been identified as having a high quantity of discharge in a short period of time. Figure 5.3: Liquid Treatment Facility Peak Discharge shows the occurrence of the peak discharge on 2 consecutive days. On the first day shown there is are two peak discharges of 12.5L/s and 18 L/s. On the second day there is 1 large peak discharge 12.0 L/s and 2 smaller discharges.

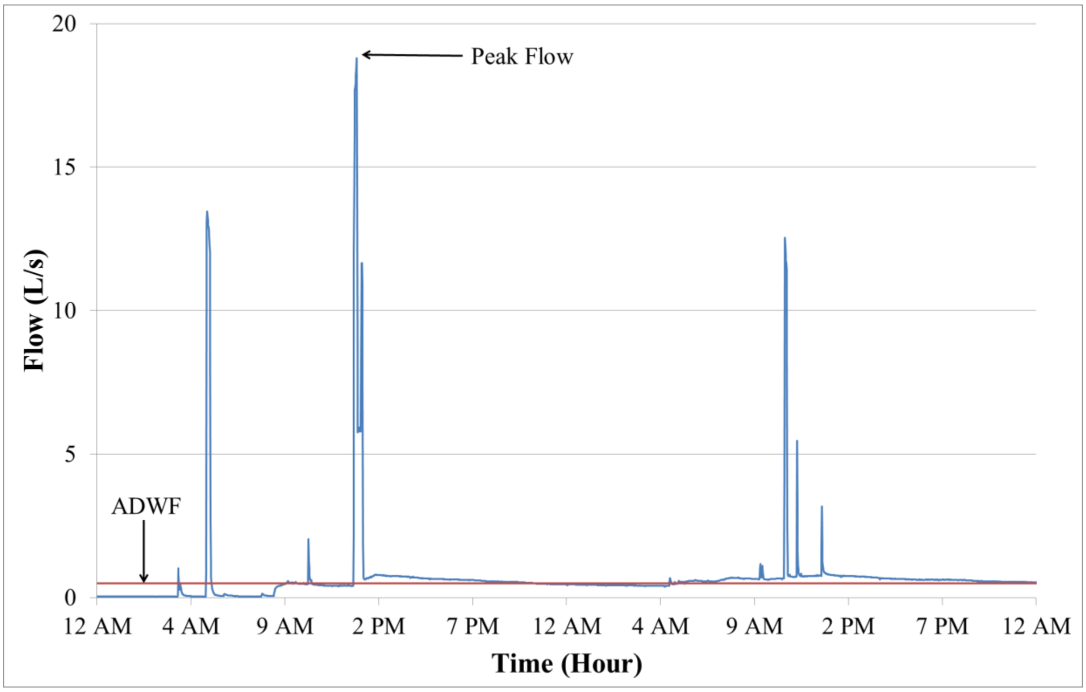

Figure 5.3: **Liquid Treatment Facility Peak Discharge**

Whilst the three methods do provide similar results for peaking factors, the values are an average and higher irregular peaks are vastly reduced as a result of averaging and also due to the SPS well buffering the flows.

The WSAA method for calculating the average monthly daily flow profile is based on averaging the same 5 minute time step for each day of the month. This resulted in a lower PDWF than calculated using the statistical method which averages the peak flow that occurred on each day. The development of a diurnal curve using the WSAA method does result in a longer period of time for the peak flow occurrence.

For weekdays the average peaking factor using the WSAA results was 1.87 whilst for weekends it was 2.24 based on the entire analysis period. The weekday value was similar to the Queensland calculated design value whilst the weekend value was higher than both the WSAA and Public Works design values.

For catchments that have a significant contributor towards to total daily flow the impact of a high instantaneous peak is an important factor to be taken into consideration when investigating the impact of overflows occurring as a result of wet weather events. If the discharge occurs during a wet weather event it may appear that excessive flows are occurring at the SPS as a result of I/I.

The results of the ADWF, PDWF, Minimum Flow and Peaking Factor for SPS 3 using the WSAA method are provided in Table 5.9: SPS 3 WSAA Dry Day Flow. The standard deviation for the entire analysis period for the peaking factor was 0.1 for weekdays which shows that there is a strong relationship between ADWF and PDWF. The results for the WSAA method for upstream SPSs are provided in Appendix K – Dry Day Flow.

Table 5.9: SPS 3 WSAA Dry Day Flow

SPS 3 Flow Analysis April 2011 to March 2014								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	9.82	8.71	18.78	18.65	1.91	2.14	2.27	2.69
May, 2011	10.06	9.18	20.83	22.21	2.07	2.42	2.00	2.03
Jun, 2011	11.00	10.14	21.34	21.59	1.94	2.13	2.81	2.85
Jul, 2011	11.32	9.47	23.80	21.35	2.10	2.25	2.43	2.30
Aug, 2011	10.76	9.67	20.03	21.57	1.86	2.23	2.67	2.90
Sep, 2011	10.31	8.80	19.96	22.18	1.94	2.52	2.17	2.04
Oct, 2011	11.21	10.01	19.87	22.59	1.77	2.26	2.61	2.26
Nov, 2011	10.78	9.84	18.95	21.26	1.76	2.16	2.54	2.72
Dec, 2011	10.81	9.40	18.73	19.18	1.73	2.04	2.66	2.50
Jan, 2012	9.85	8.58	19.09	18.14	1.94	2.11	2.11	2.05
Feb, 2012	11.54	9.75	19.81	20.42	1.72	2.09	3.52	2.78
Mar, 2012	12.51	10.54	21.91	24.10	1.75	2.29	3.94	3.16
Apr, 2012	11.55	10.34	22.26	21.18	1.93	2.05	3.06	2.87
May, 2012	11.58	10.47	22.68	23.99	1.96	2.29	2.37	2.48
Jun, 2012	12.20	10.41	21.19	23.35	1.74	2.24	3.09	2.71
Jul, 2012	11.55	10.46	20.32	22.65	1.76	2.17	2.68	2.67
Aug, 2012	10.36	9.66	19.70	22.49	1.90	2.33	1.86	2.06
Sep, 2012	9.86	9.18	18.80	21.14	1.91	2.30	1.92	1.95
Oct, 2012	9.90	9.89	17.45	19.84	1.76	2.01	1.87	2.30
Nov, 2012	9.92	8.77	18.85	20.59	1.90	2.35	1.85	1.80
Dec, 2012								
Jan, 2013								
Feb, 2013	10.60	10.35	19.45	23.50	1.84	2.27	2.00	2.35
Mar, 2013	10.26	9.48	19.96	20.64	1.94	2.18	2.05	2.24
Apr, 2013	10.58	9.69	19.32	23.25	1.83	2.40	2.33	2.42
May, 2013	10.27	9.40	20.08	20.42	1.95	2.17	2.11	1.88
Jun, 2013	11.57	10.75	21.66	25.36	1.87	2.36	2.83	3.07
Jul, 2013	12.82	11.06	24.20	23.71	1.89	2.14	3.11	2.90
Aug, 2013	12.04	11.18	23.67	30.24	1.97	2.71	2.41	2.47
Sep, 2013	11.58	10.22	22.75	25.08	1.97	2.45	2.19	2.04
Oct, 2013	10.93	10.00	20.32	22.80	1.86	2.28	2.30	2.24
Nov, 2013	12.42	11.19	24.66	24.56	1.99	2.19	3.19	2.28
Dec, 2013	11.44	10.57	20.66	22.17	1.81	2.10	2.75	2.75
Jan, 2014	11.11	10.09	21.04	22.63	1.89	2.24	2.38	2.49
Feb, 2014	11.14	9.79	19.38	21.39	1.74	2.18	2.13	2.17
Mar, 2014	11.98	9.64	20.53	20.57	1.71	2.13	2.54	2.32
Average	11.05	9.90	20.65	22.20	1.87	2.24	2.49	2.43
Std Dev	0.82	0.68	1.72	2.21	0.10	0.14	0.49	0.35

5.7.Minimum Flow

The minimum flow that occurs in a sewerage system is generally between 12am and 4am. During this period it is adopted that up to 80% of flows may be a result of ground water infiltration into the sewerage system (WSAA 2013).

The minimum flow calculated for SPS 3 using the two methods is provided in Table 5.3: SPS 3 Dry Day Flow Expanded Data Set for the statistical method and Table 5.8: SPS 3 WSAA Dry Day Flow. The results indicate that the average minimum flows during the weekday are approximately 2.52 L/s whilst on the weekend is 2.54L/s (average of traditional and WSAA values). The estimated minimum flow in the gravity portion of Catchment 3 is 1.3 L/s. Table 5.10: Minimum Flow summaries the results for each SPS catchment and also for the gravity potion of SPS 3. It also includes a comparison of the two methods used to determine the minimum flow based on the entire analysis.

Table 5.10: Minimum Flow

SPS / Catchment	Period 1		Period 2		Period 3		All Periods			
							Weekday		Weekend	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Statistical	WSAA	Statistical	WSAA
3 (L/s)	2.58	2.65	2.4	2.53	2.56	2.76	2.54	2.49	2.65	2.43
15 (L/s)	0.14	0.16	0.13	0.14	0.13	0.12	0.13	0.13	0.14	0.13
21 (L/s)	0.33	0.36	0.29	0.3	0.31	0.35	0.31	0.32	0.34	0.33
23 (L/s)	0.2	0.26	0.14	0.1	0.08	0.12	0.18	0.15	0.18	0.11
26 (L/s)	0.04	0.04	0.09	0.07	0.03	0.15	0.1	0.04	0.09	0.04
29 (L/s)	0.39	0.33	0.55	0.54	0.73	0.75	0.6	0.52	0.59	0.51
3 (Gravity)	1.48	1.5	1.2	1.38	1.28	1.27	1.22	1.33	1.31	1.31

Based on the results in Table 5.9 there has been a reduction of minimum flow of 0.2L/s in the gravity portion of catchment 3. If 80% of the flow is attributed to ground water infiltration this equals 1.0 L/s for both weekdays and weekends for the entire analysis period.

SPS 29, correctional facility, is a new development and it is expected that the minimum flows indicated are a result of actual wastewater discharge and not from groundwater infiltration. The basis for this is the internal pipe network is shallow, less than 3 years of age and the residents do typically leave the facility.

The WSAA sewer design manual provides a method for estimating groundwater infiltration, equation (2.9). Based on a catchment area of 199.7 ha and 1.0 L/s this equates to 20% of the pipework being wet.

This value does not appear to be consistent with the location of the 29km of pipework servicing the gravity portion of SPS 3. Excavation by maintenance crews on sections of the main generally indicates dry ground conditions. It is more probable that a higher majority of night time flows are a result of leaking taps/cisterns and actual wastewater discharge.

5.8. SPS 3 Diurnal Curve

The WSAA method enabled the development of an average diurnal curve for each of the catchments. The diurnal curve provides a visual representation of the daily flow variation, peak flow and minimum flow. It also allows for a visual comparison between weekday and weekend wastewater discharge. For each catchment diurnal curves have been developed to assist visualising the actual flow variation that occurs throughout the day. The diurnal curves for each of the upstream SPSs are provided in Appendix K – Dry Day Flow. Figure 5.4: SPS 3 Diurnal Curve represents the flow profile for SPS 3 based on period 3.

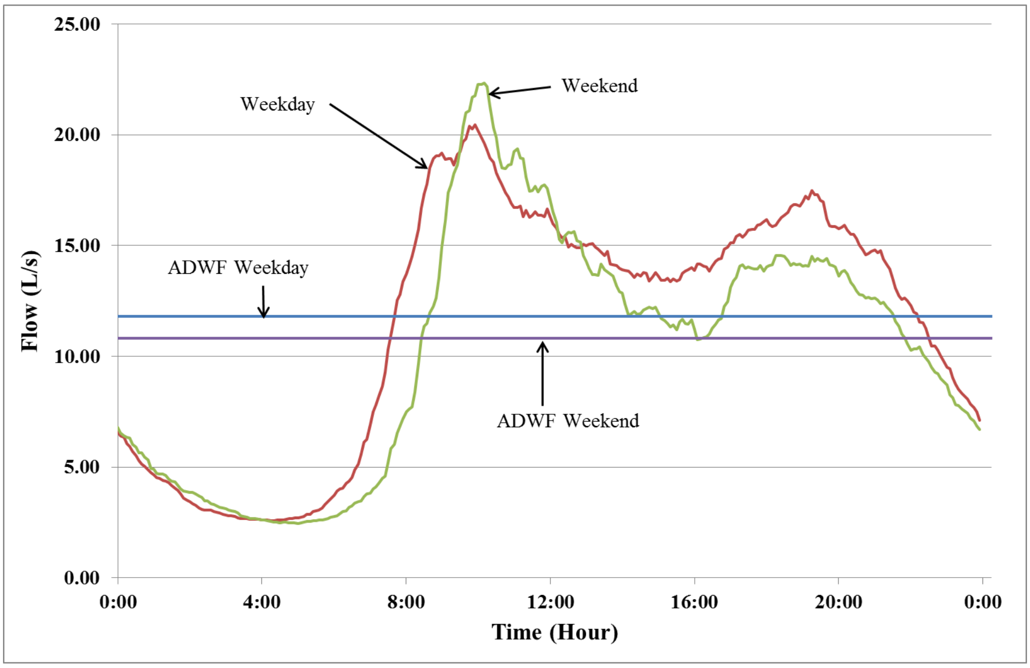

Figure 5.4: **SPS 3 Diurnal Curve**

Figure 5.4 indicates that the peak morning period for the weekend occurs slightly later in the morning than that of a weekday whilst the peak afternoon period is higher for a weekday compared to the weekend.

The minimal flow for both periods is similar whilst the rise from minimum night time flow to peak morning period occurs earlier for the weekday compared to the weekend.

To determine the diurnal curve for the gravity portion of catchment 3 i.e. with no flows from upstream SPSs the flows from upstream catchments were delayed for the times calculated in Table 3.3: SPS Flow Delay Times. Figure 5.5: Weekday Diurnal Curve Catchment 3 illustrates the results and provides a comparison to the diurnal curve for SPS 3. The catchment displayed similar characteristics to SPS 3 flow profile.

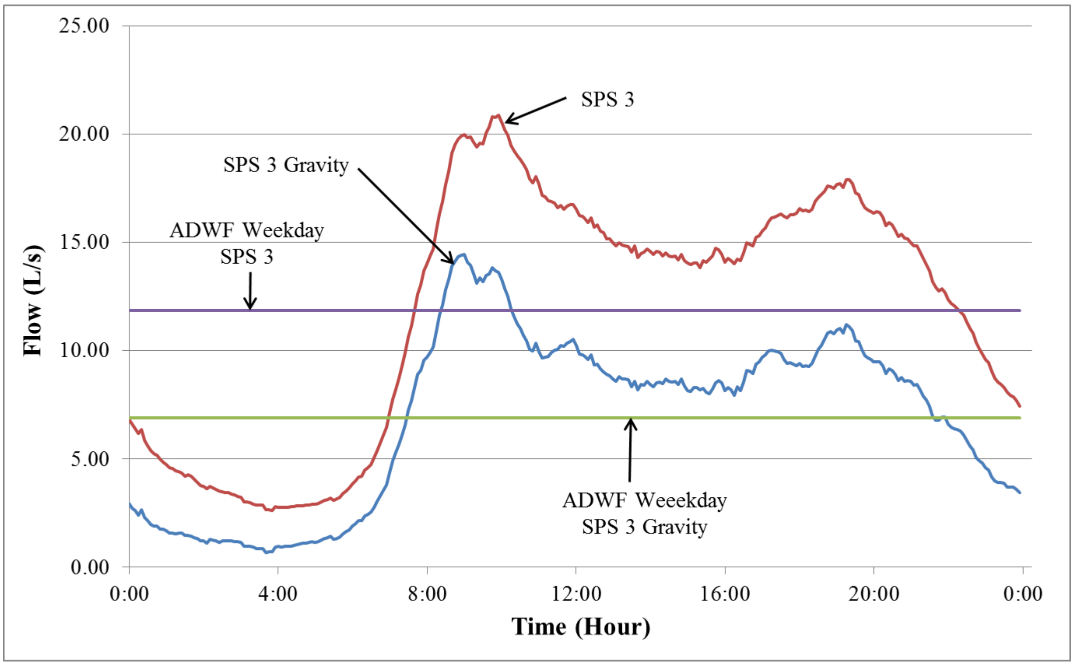

Figure 5.5: **Weekday Diurnal Curve Catchment 3**

As a result of using the average for each time step over an extended period of time the impact of an irregular peak discharge appears minimal. It also does not highlight the variation of flow that can occur throughout the daily flow profile. The diurnal curves for each of the upstream SPSs is provided in Appendix K – Dry Day Flow

The standard deviation for each time step used to develop the diurnal curve was calculated along with the minimum flow that was recorded for each time step.

Figure 5.6: SPS 3 Variability of Diurnal Flow highlights the potential variation in flow that can occur on a given dry day. The maximum flow is the calculated using 3 standard deviations from the mean for all time steps. It can also be seen that there is potential for the maximum ADWF to be equal to or greater than the average PDWF.

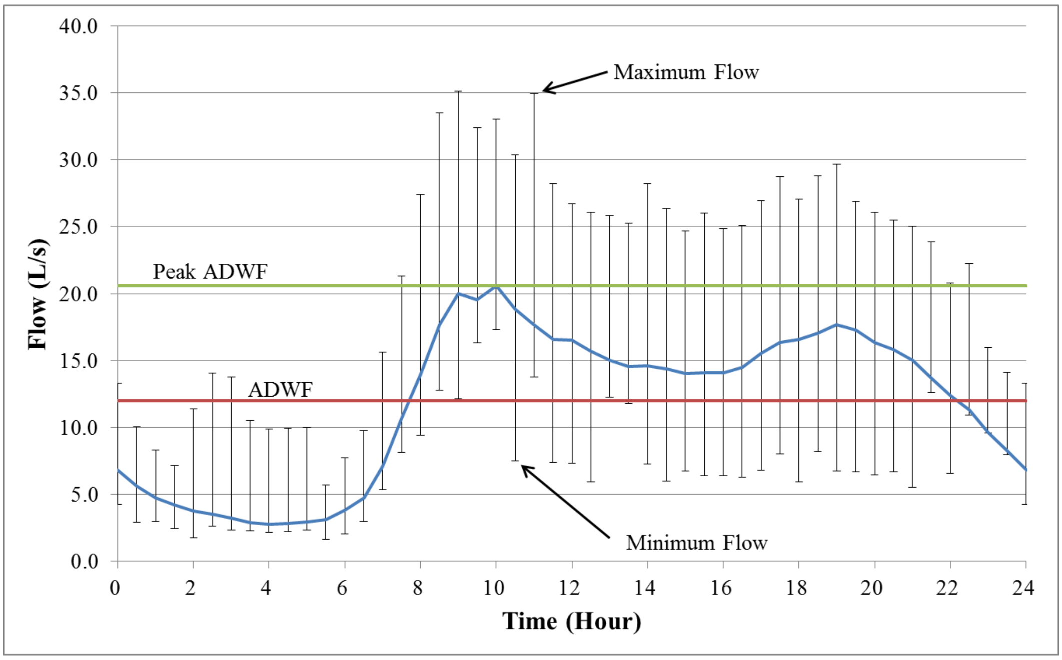

Figure 5.6: **SPS 3 Variability of Diurnal Flow**

The significance of the variability of dry day flow can be seen in Figure 5.7: SPS 23 Variability of Weekday Diurnal Flow. It is know that SPS 23 has a large discharger and whilst the diurnal curve shows a PDWF of approximately 1.2L/s the potential for higher flows is highlighted by the 3 standard deviations. For this SPS it can be seen that maximum flows exceeds 10L/s and for approximately 6 hours the flow can be in excess of 4L/s which is approximately 25% of the calculated ADWF of SPS 3.

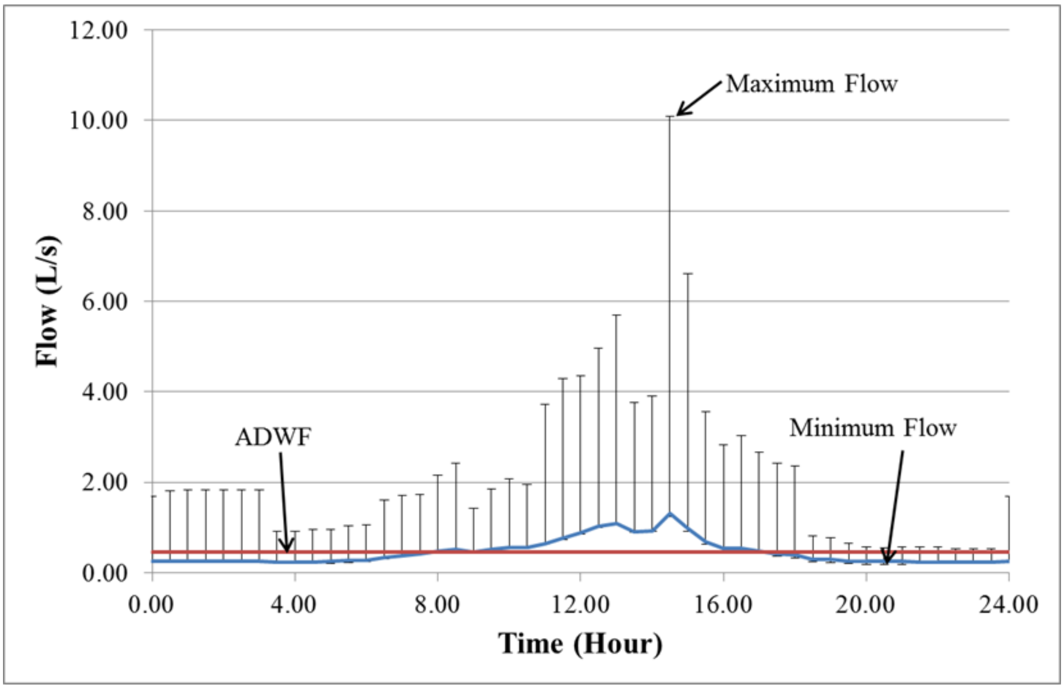

Figure 5.7: **SPS 23 Variability of Diurnal Weekday Flow**

5.9. Peak Wet Weather Flow

The calculation of the Peak Wet Weather Flow (PWWF) has been undertaken for SPS 3 using three design methods:

- Shoalhaven Water method (Equation 2.6),
- Queensland Method (Equation 2.7),
- WSAA method (Equation 2.8).

For the purpose of the calculation it has been adopted that the ADWF is equal to 12.0 L/s. and peak day ADWF equals 20.6 L/s (Table 5.3).

<u>Shoalhaven Water PWWF Method</u>

Table 5.11: Shoalhaven Water PWWF method calculated the design PWWF using both the ADWF and peak ADWF. The peaking factor was calculated using the Public Works method Equation 2.4. The results indicate the largest contributor to the PWWF is the storm allowance of 0.058 L/s/ET.

The calculated PWWF was 138.4 L/s based on an annual ADWF of 12.0 L/s and 156.6 L/s based on a peak ADWF of 20.6 L/s.

Table 5.11: Shoalhaven Water PWWF Method

Shoalhaven Water PWWF Method		
ET's	1950	1950
ADWF (L/s)	12.0	20.6
Peaking Factor (d)	2.11	2.11
PDWF (L/s)	25.3	43.5
Storm Allowance	113.1	113.1
PWWF (L/s)	138.4	156.6

Queensland PWWF Method

Table 5.12: Queensland Traditional PWWF method provides a summary of the calculations using Equation 2.7 and Equation 2.8. The peaking factor is calculated using Equation 2.5. The calculated PWWF flows were significantly lower than the values calculated using the Shoalhaven Water method. Using Equation 2.7 the PWWF equates to 103 L/s for a peak ADWF of 20.6 L/s. Using equation 2.8 resulted in a reduction of the PWWF from 103 L/s to 81 L/s. Based on the annual ADWF the calculated PWWF was 60.0 L/s and 47.2 L/s using Equations 2.7 and 2.8 respectively.

Table 5.12: Queensland Traditional PWWF Method

Queensland PWWF Method		
EP's	4622	4622
ADWF (L/s)	12	20.6
Peaking Factor (C_2)	1.94	1.94
PDWF (L/s)	23.3	39.9
PWWF (L/s) 5 x ADWF	60.0	103.0
PWWF (L/s) C x ADWF	47.2	81.0

WSAA PWWF Method

The WSAA PWWF method requires the use of the 2013 IFD charts for Nowra as part of the calculation. The design standard recommends using the design rainfall depth value for a 2 year 1 hour storm event. These values are used by WSAA to allow customisation of the PWWF to local climatic conditions. There are 4 factors that influence the design PWWF, a sensitivity of each factor is undertaken to determine the influence each factor has on the calculated PWWF. It is adopted that for each sensitivity analysis the other influencing factors shall be kept consistent. These 4 factors and adopted values are as follows;

- Leakage Severity = 1.0
- Containment Standard = 5 years
- Storm Duration = 1 hour
- Rainfall Event Occurrence = 2 years.

The diurnal curve for period 3 (Figure 5.6) with an ADWF of 11.8 L/s has been used for the sensitivity calculations. The following calculations have been used to determine the relevant peaking factor for each 5 minute flow.

5 minute flow = 6.0 L/s

1 Standard Deviation = 2.0 L/s

Peak 5 minute flow = 6.0 L/s + 2.0 L/s = 8.0 L/s

Peaking Factor = 8.0 L/s / 6.0 L/s = 1.25

The diurnal flow values for SPS 3 weekday period 3, standard deviation, Peak flow and peaking factor are provided in Appendix L: WSAA Sensitivity Calculations. The rainfall event and duration are as per the WSAA recommendation.

ARI IFD Charts

The 2013 Nowra IFD charts were obtained from the Bureau of Meteorology Web Site for the Nowra Area. The sites latitude is 34.95 and longitude 150.54 and the 1 hour storm event with a 2 year re-occurrence rainfall depth is 26.8mm. A copy of the IFD Charts for Nowra is provided in Appendix M: Nowra IFD Charts.

Leakage Severity

The leakage severity is rated from 0.4 (low) to 1.6 (high) and is a combination of the soil aspect and network defects. Figure 5.8: PWWF Leakage Severity Co-efficient Sensitivity illustrates the effect that the leakage coefficient has on the design PWWF.

The average PWWF ranges from 45.8 L/s to 138 L/s, the peak PWWF i.e. highest flow based on the diurnal curve, ranged from 55.9 L/s to 148.1 L/s. In comparison using the calculated peaking factor of 2.08 (Table 5.8) the PWWF ranged from 55.3 L/s to 147.5 L/s. This demonstrates that a PWWF diurnal curve can be developed using the 5 minute flow values with a peaking factor equal to 1 standard deviation. Appendix L – WSAA sensitivity calculations has the tabulated results based on an ADWF of 11.8 L/s and a peaking factor of 2.08.

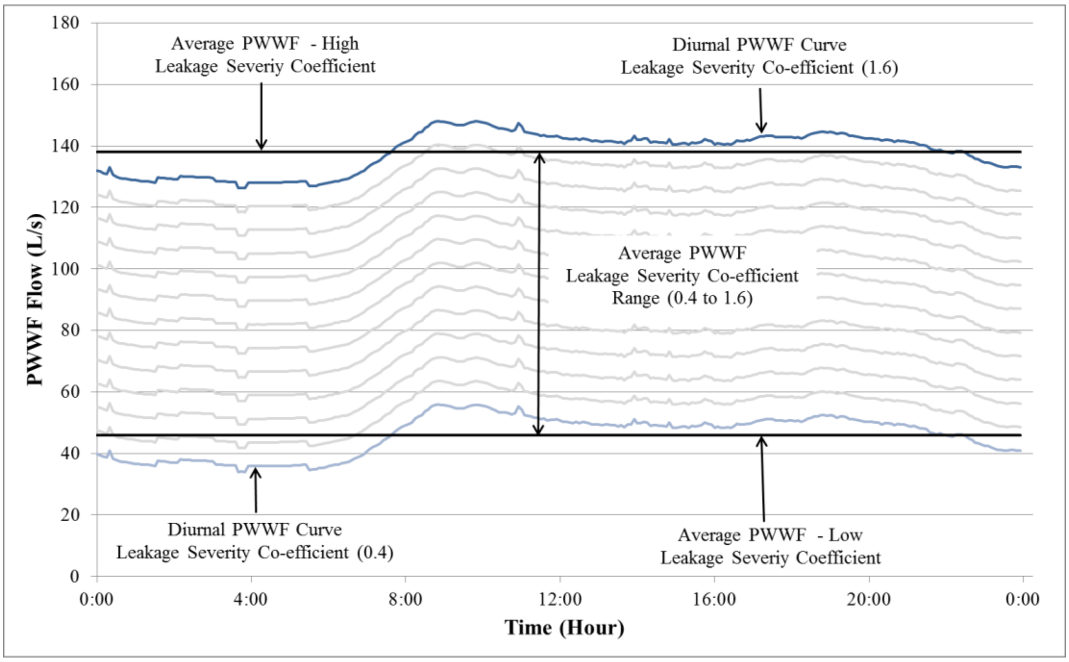

Figure 5.8: PWWF Leakage Severity Coefficient Sensitivity

The Queensland PWWF value of 47.2 L/s (Table 5.12) is similar to the WSAA diurnal average PWWF of 45.8 L/s with a leakage severity coefficient of 0.4 i.e. low impact from both soil and network defects. The Shoalhaven Water PWWF of 138.4 L/s (Table 5.11) is similar to the WSAA diurnal average PWWF of 138 L/s with a leakage severity coefficient of 1.6 i.e. high impact from both soil and network defects. In summary the WSAA leakage severity coefficient provides a range of values to determine the quality of the network and the impact that the soil has on PWWF.

Containment Standard

The containment standard is related to the level of certainty that is to be applied to a system in relation to the possible occurrence of overflows. For instance a 5 year containment standard means that only a single overflow in a 5 year period is acceptable for the specified storm event. The results of the containment standard sensitivity analysis are shown in Figure 5.9: Containment Sensitivity; the results show PWWF ranges from 70.3 L/s to 120.6 L/s for containment standards from 1 year to 50 years respectively.

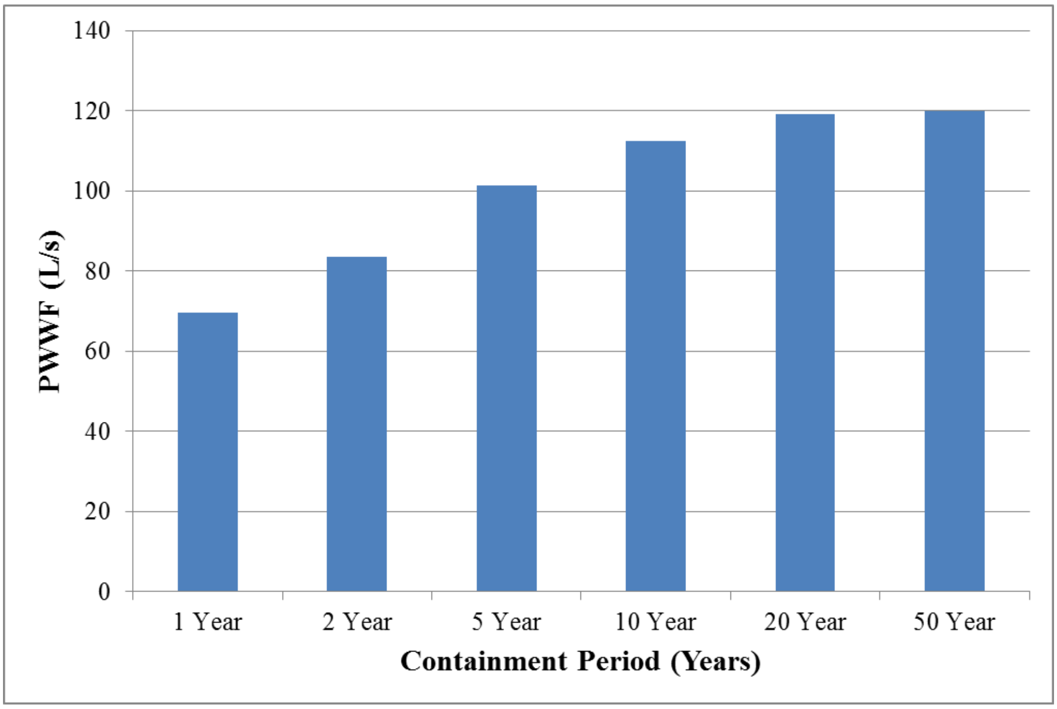

Figure 5.9: Containment Sensitivity

The largest increase in PWWF occurs between the 2 year and 5 year containment periods; there is only minimal increase between a 20 year and 50 year containment period. In effect the containment factor is a scale factor that is applied to the PWWF.

Storm Duration

The results of the storm duration sensitivity analysis are provided in Figure 5.10: Storm Duration Sensitivity, the results indicate a rapid increase in the calculated PWWF for storms ranging from 1 hour to 24 hours. This is a direct result of a rapid increase in the rainfall dependent I/I.

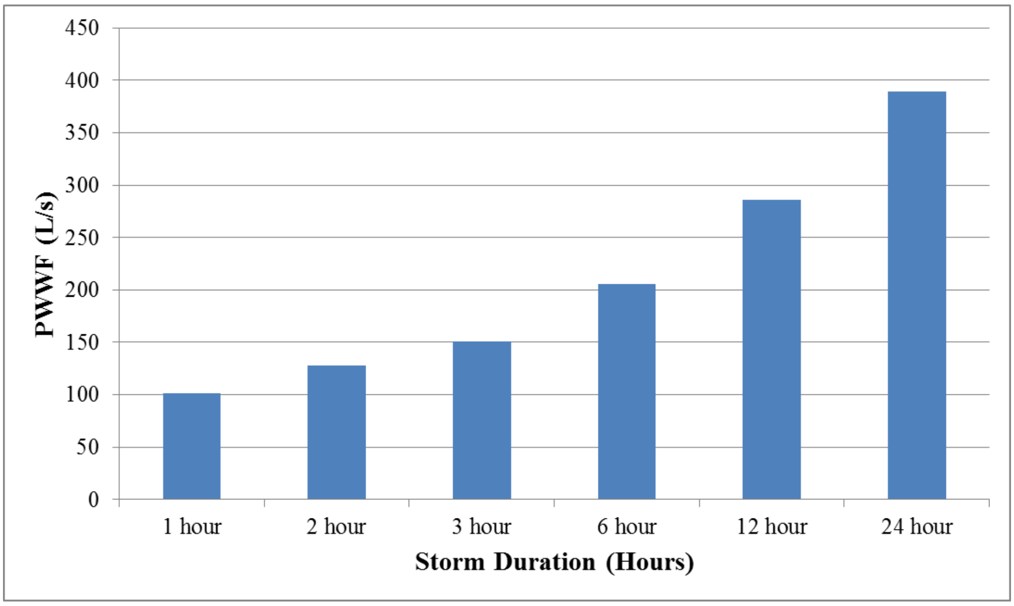

Figure 5.10: Storm Duration Sensitivity

Rainfall Event Occurrence

The results of the rainfall event occurrence sensitivity analysis are provided in Figure 5.11: Rainfall Event Occurrence Sensitivity. The sensitivity of the rainfall event occurrence was not as significant as the sensitivity of the storm duration.

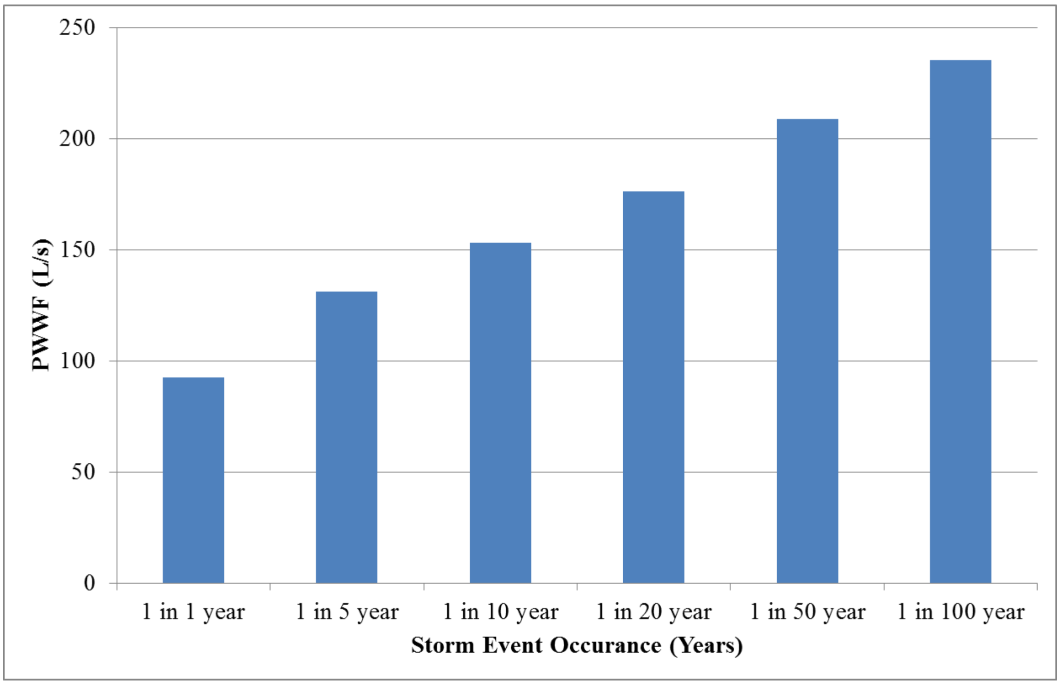

Figure 5.11: Rainfall Event Occurrence Sensitivity

The PWWF ranges from 93.3 L/s for a 1 year event up to 236.3 L/s for a 1 in 100 year event based on a 1 hour event. The tabulated calculations for all 4 sensitivity analyses are provided in Appendix L – WSAA Sensitivity Analysis.

SPS PWWF

The PWWF for all SPS's was calculated adopting similar values to those used for the sensitivity analysis except for the leakage severity. For SPS 26 and 29 the network is new however the soil aspect is high, whilst for SPS 3, 15, 21 and 23 the network is aging and the soil aspect is high. It was adopted that a leakage severity coefficient equal to 1 would be adopted for SPS 26 and 29 and for all other catchments a value of 1.6 would be used. The containment standard was set to 5 years for a 1 hour storm event with a 1 in 2 occurrence interval. The ADWF and peaking factor as calculated using the Statistical expanded data set was adopted. The results are shown in Table 5.13: SPS PWWF and indicate that the gravity portion of Catchment 3 has a PWWF of 97.2 L/s which is 22.9 L/s greater than the high speed flow rate of the pump at SPS 3. SPS 15 and 21 also have a PWWF that is greater than the pump flow rate whilst for SPS 29 the design pump flow rate is adequate after operational checks removed blockages as a result of gravel in the system. The pump flow rate for SPS 29 has increased to 11.3 L/s. SPS 23 has a calculated PWWF of 7.0 L/s however regular peak flows of up to 18L/s are recorded at this site during dry weather.

Table 5.13: SPS PWWF

Criteria	ADWF (L/s)	Peaking Factor	PDWF (L/s)	PWWF (L/s)
SPS 3	12.0	2.08	25.0	147.5
SPS 3 Gravity	7.16	2.11	15.1	97.2
SPS 15	0.41	2.91	1.19	20.3
SPS 21	1.28	2.53	3.24	17.1
SPS 23	0.49	6.54	3.20	9.3
SPS 26	0.15	2.19	0.33	1.2
SPS 29	2.62	2.1	5.50	10.2

The sum of PWWF values for the upstream SPSs combined with the gravity portion of catchment 3 equals 155.3 L/s which is approximately 5% greater than the PWWF calculated for SPS 3. The combined pump flow rate from the upstream catchments equals 57 L/s however this is now increased to 62.3 L/s with the pump problems at SPS 29 rectified. The high speed pump flow rate at SPS 3 is 74.3 L/s resulting in only 12 L/s capacity for SPS 3 gravity catchment during a wet weather event. It is not possible at SPS 3 to operate dual pumps at high speed at SPS 3 as this result in rising main breaks occurring.

5.10. Historical Rain Events

To determine suitable rain events for the analysis the 5 minute rainfall data for Nowra RAN Air Station AWS rain gauge (Station No. 068072) was used. The data was for a period from 1st December 2011 to 1st December 2013.

The data set had 3% of records missing for a two year period; however the majority of the missing records were for a short period of time on dry days. These missing records appeared to be a result of servicing of the weather station as they occurred every 2 – 3 days for 1 hour. No missing data was present for the 20 highest ranked rain events. Figure 5.12: Daily Rainfall Nowra is the rainfall that was recorded for the 2 year period.

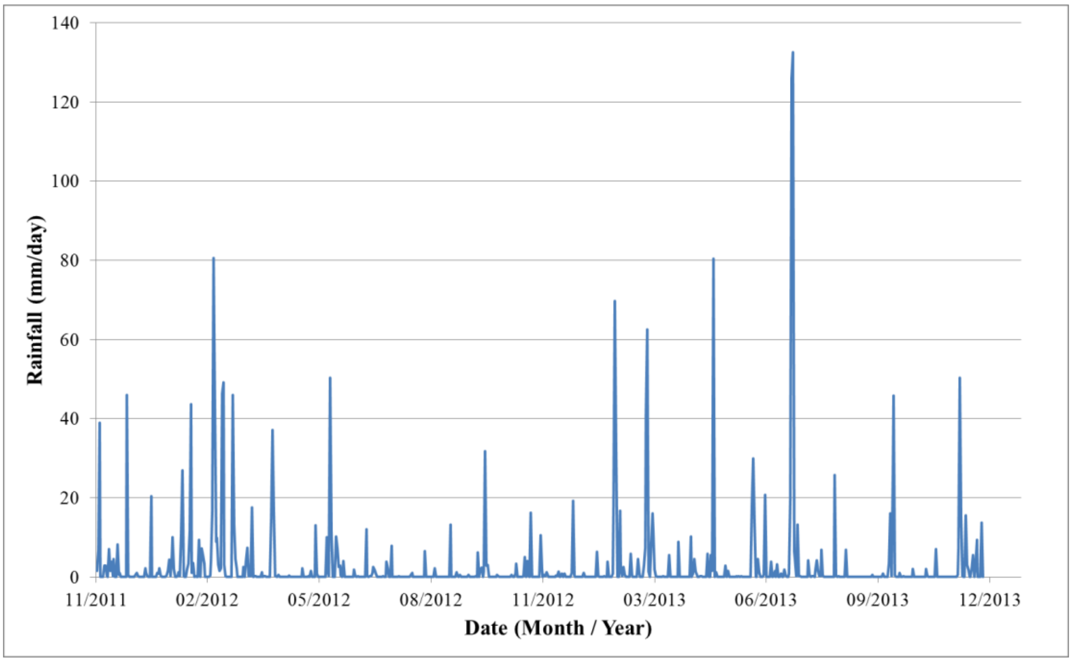

Figure 5.12: Daily Rainfall Nowra

There were 20 rainfall events that occurred with rainfall of 20mm or more during the 2 year period. Using the 2013 Nowra IFD chart each rain event was compared against the values to estimate the frequency and duration of each of the 20 wet weather events. A copy of the IFD charts is provided in Appendix M: Nowra IFD Charts.

The rain events were ranked from highest to lowest and the peak rainfall intensity for storm durations ranging from 5 minutes to 7 days used to determine the ARI event rating. Table 5.14: Storm Event Rating and Duration summaries the results.

Table 5.14: Storm Event Rating and Duration

Total Ranfall (mm)	Length of Storm (Hrs)	ARI Event Rating	Event Range	Start Date / Time	Overflow Duration (Hrs)
48.6	2.2	Between 1 in 10 and 1 in 20 year event	0.5 to 1 hour	24/02/2013 0:50	6.34
302	153	Between 1 in 5 and 1 in 10 year event	48 to 96 hours	23/06/2013 3:45	44.7
55.2	17.1	Between 1 in 2 and 1 in 5 year Event	10 to 30 minutes	16/03/2012 15:15	N/A
47.5	23.9	Between 1 in 2 and 1 in 5 year Event	0.5 to 1 hour	19/12/2011 0:00	N/A
43.4	13.7	Between 1 in 2 and 1 in 5 year Event	15 minutes to 1 hour	10/02/2012 8:25	N/A
80	14.4	Between 1 in 1 and 1 in 2 year event	2 to 6 hours	19/04/2013 21:35	8.47
65.8	11.7	Between 1 in 1 and 1 in 2 year event	2 to 6 hours	16/09/2013 14:55	unknown
25.8	7.7	Between 1 in 1 and 1 in 2 year event	15 minute	30/07/2013 16:55	N/A
131.8	46.7	Less than 1 in 1 year Event	N/A	28/02/2012 13:20	0.12
119	54.7	Less Than 1 in 1 year Event	N/A	26/01/2013 22:45	unknown
89.4	22.5	Less Than 1 in 1 year Event	N/A	7/03/2012 9:20	unknown
69.2	45.6	Less than 1 in 1 year Event	N/A	10/11/2013 14:25	N/A
54.6	40.8	Less than 1 in 1 year Event	N/A	22/05/2013 17:10	N/A
51.2	24.2	Less than 1 in 1 year Event	N/A	17/04/2012 18:00	N/A
48	19.8	Less than 1 in 1 year Event	N/A	5/06/2012 11:30	N/A
39	10.4	Less Than 1 in 1 year Event	N/A	26/11/2011 0:00	N/A
31.6	8.8	Less than 1 in 1 year Event	N/A	12/10/2012 2:25	N/A
27	21.3	Less than 1 in 1 year Event	N/A	3/02/2012 1:20	N/A
20.8	11.4	Less than 1 in 1 year Event	N/A	2/06/2013 0:00	N/A

Within the analysis period there were 2 storms of significance with one storm of 2.2hrs in length resulting in 37mm of rainfall falling in 30 minutes. This storm was classified as being between a 1 in 10 and 1:20 year 1/2hr event. A second storm that occurred in June 2013 was 6.5 days in length and was classified as being between a 1 in 5 and 1 in 10 year event with 260mm of rainfall in a 48hr period.

The SCADA historical records were used to determine the length of time that the overflow occurred. Errors within the SCADA records were found as the recorded timestamp for the overflow event did not appear correct. For the 2.2hr storm the SCADA time stamp indicates that the overflow commenced before the rainfall. There are two possible reasons for this, the first the storm event occurred over the catchment before the rain gauge or the time settings in the SCADA system were incorrect. Further investigation into this problem is required however initial investigations indicate the time settings for the SCADA overflow records were not set correctly.

The lack of reliability in the overflow timestamps makes it difficult to calculate the quantity of total flow that was pumped prior to an overflow occurring.

5.11. Field Analysis

Prior to field inspections being carried out (during a rain event) a 340kL overflow storage tank for SPS 3 was commissioned. This overflow tank increased the SPS and gravity main storage from approximately 160kL to 500kL. It was decided for the initial field work that a small portion of catchment 3 would be used for field investigations.

Dry Weather Field Investigations

A portion of catchment 3 which had previous limited flow monitoring undertaken was selected for field investigations. Copies of extracts of SW gravity sewer plans are provided, for reference in Appendix N – Field work Catchment Plans.

Rain events in 2013 had resulted in overflows occurring from the 1st manhole (MH 3E/2) upstream of where the flow monitoring (MH 3E/1) had previously been undertaken. Site inspection indicated that both manholes should have overflowed as MH 3E/1 was approximately 1m lower in elevation. CCTV of the joining pipework was undertaken and it found that a large portion of the pipe, approximately 50%, was blocked with a fatty residue. This blockage was removed using high pressure jet washing and it appeared that the partial blockage had been in the line for considerable period of time. This partial blockage would have resulted in the flow monitoring recording high velocities and subsequently higher flows. The overflow from manhole 3E/2 was attributed to the partial blockage and the data from the flow monitoring was regarded as not suitable for wet weather analysis.

6 manholes were selected for the initial wet weather trial, these manholes were all located upstream of MH 3E/2. An additional manhole 3ED/5 located on a side branch of the system from manhole 3E/2 was also selected as the upstream properties included two restaurants, a service station and a villa complex. All

properties within the field trial area were estimated to have been built between 1960 and 1980. The intent was to measure the flow using a mobile flow device whilst suitable rainfall was occurring.

Only two rainfall events occurred during the project period and both resulted in overflows occurring, after the commissioning of the overflow storage tank. The first rain event commenced at 8pm Saturday 16th August 2014 and limited field work was able to be undertaken. This event resulted in 104.6mm of rainfall and was rated as less than a 1 in 1 year event. A second rain event commenced at 8.20pm on the 25th August and limited field work was able to be undertaken. This event resulted in 54mm of rainfall and was rated as less than a 1 in 1 year event.

Rain Event 1 - 16th to 18th August 2014

Overflow at the site commenced at 3:22am Monday 18th August 2014. The total rainfall prior to overflow occurring was 50.4mm, in the 12 hours prior to overflow 16mm of rainfall fell with 10mm of this rainfall occurring in the 2 hours prior to the overflow commencing. Figure 5.13: SPS 3 Overflow Event 16th to 18th August shows the flow profile for the rain event.

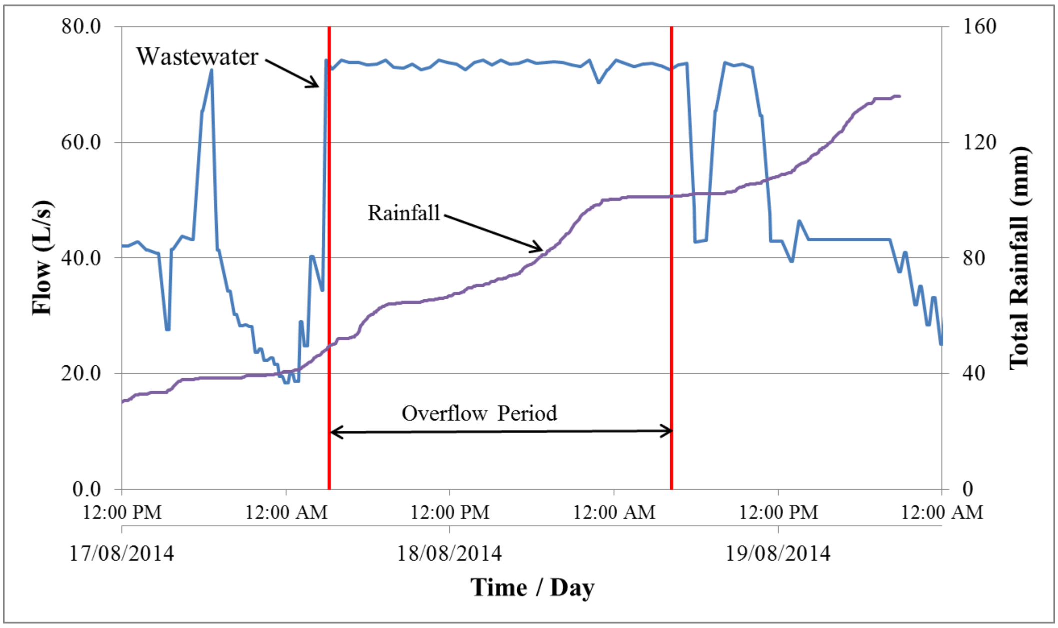

Figure 5.13: SPS 3 Overflow Event 16th to 18th August 2014

At 2:43am on Monday the 18th the low speed pump switch on indicating that the SPS well reached operating level with 3.1kL in the SPS well. At this point the pumps commenced transferring flow at 49 L/s, at 2:55am the pumps moved to high speed and continued to transfer flow at 74.3L/s. At 3.11am the overflow commenced, indicating that the total of pumped flow and storage had been exceeded in approximately 1620 seconds. This rapid rate would indicate a flow in excess of 360L/s. The estimated PWWF during this period was 125L/s. The calculated inflow does not appear consistent with the rainfall occurring prior or during the overflow event. If the estimated flow is correct it indicates that there is significant inflow to the system occurring.

During the daytime of the 18th August the 6 manholes were checked along with a number of other manholes randomly selected. Site visits to SPS 3 were also undertaken, it was noted that there was a build-up of effluent in the system however no large flows were detected within the system. No rainfall of significant intensity occurred whilst undertaking the field investigations. The timing of this rainfall event highlight the need to have long term flow monitoring installed for a number of sub catchment areas.

It was decided on completion of the field inspections if time permitted and a second rainfall event occurred that the field work would involve locating sources of direct inflow to the system.

Rain Event 2 - 25th to 26th August 2014

This rain event commenced in the late evening, field investigations were again undertaken the following day. It was decided to recheck the previous manholes however visual results indicate that no large flows were evident. A total of 16 manholes were checked twice between 8:30am and 3:30pm. No rainfall of significant intensity occurred till after field work investigations had ceased.

During the field investigations evidence of sources of infiltration was detected, these include a manhole under pooled water and a yard gully which showed signs of inflow having occurred. The storage tank was also checked and found to be approximately 1/2 full at 1:30pm and 2/3 full at 3:30pm.

Overflow at the site commenced at 4:05pm on Tuesday 26th August 2014, based on the live SCADA system. The SCADA historical results indicate that overflowed commenced at 3:17pm, this is not correct as I was on site at 3:30pm and no overflow was occurring. Again this highlighted the need to further investigate the accuracy of the SCADA historical records for overflow events. Figure 5.14: SPS 3 Overflow Event 25th to 26th August 2014 details the rainfall and the SCADA flow profile for the rain event.

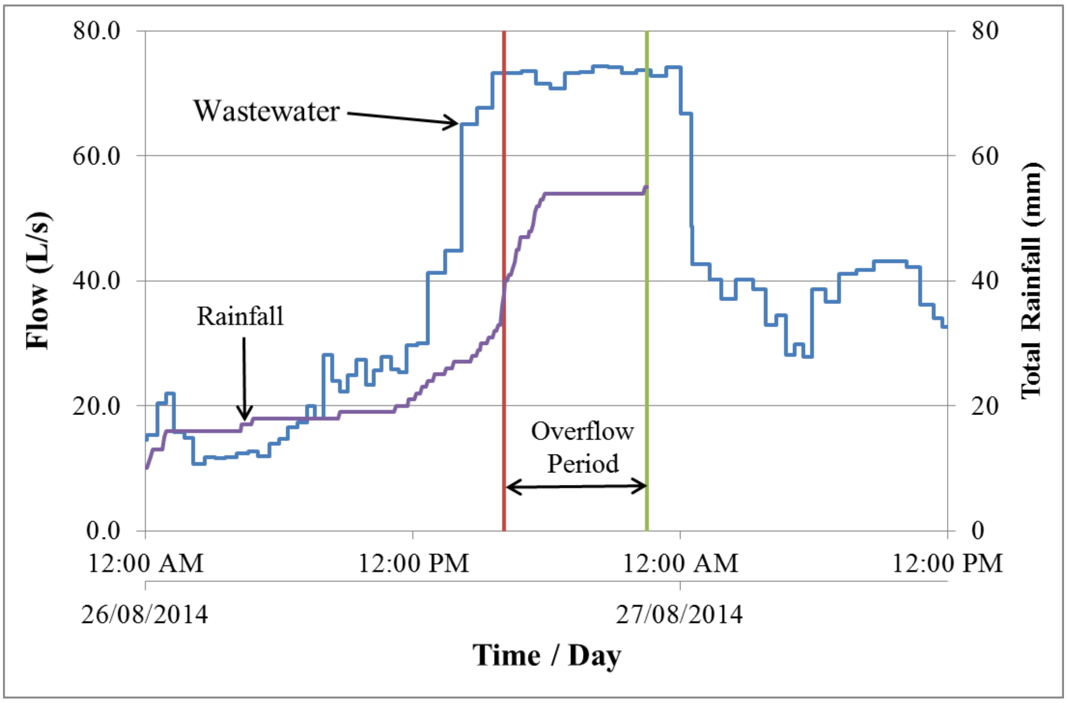

Figure 5.14: SPS 3 Overflow Event 25th to 26th August 2014

It can be seen that 10mm of rainfall occurred in a short period of time (30 minutes) and overflow commenced shortly after this high intensity rainfall started. The system however was observed to be nearing capacity prior to the higher intensity rainfall occurring.

<u>Field Observations</u>

Valuable knowledge of the system was gained by undertaking the field inspections.

In one vacant lot it was noticed that rainfall did not pool for long periods of time even though at nearby locations the rain puddles remained for several hours after the rain event had finished. Figure 5.15: Submerged Manhole shows a manhole that was located by Shoalhaven Water employees Chris Button and Nathan Wood and is an example of a potential source of I/I.

Figure 5.15: Submerged Manhole

(Source: Wady, I (26/8/2014) Submerged Manhole.jpg)

This manhole was raised and a new lid and surround installed. Visual observations during a major rain event in October 2014 was the water remained pooled on the property for a 2 day period thus the manhole was a source of I/I. A number of other manholes have recently been located within public reserves in other catchments which have been damaged by tractors using grass slashers. In one case 25% of the manhole lid and surround was removed, these cases are not only sources of I/I but also a safety issue for the public. All have now been repaired.

Another source of I/I that was discovered during the field work was broken inspection openings (IO). Figure 5.16: Broken inspection opening is an example of one IO located 15m upstream from the manhole in Figure 5.15. During periods of wet weather surface flow is able to directly enter the sewer system via this open IO.

Figure 5.16: Broken inspection opening

(Source: Wady, I (26/8/2014) Broken inspection opening.jpg)

Poor property drainage can also result in inflow into yard gullies. Hard surface areas in properties are regularly attached to the dwelling and the yard gully is often located within the hard surface area. As a result of surface flow during a rain event the rainwater enters the properties internal sewer network directly.

Figure 5.17: Yard gully inundation is an example of a yard gully in the South Nowra industrial estate. It can be seen that the rim of the yard gully is wet and the grated cover is not in place. The building is located downhill from the road reserve and extensive amount of pooled water can be seen between the base of the embankment and building.

Figure 5.17: Yard gully inundation

(Source: Wady, I (26/8/2014) Yard Gully Inundation.jpg)

Roof drainage connections are also a source of I/I, Figure 5.18: Roof drainage connection is an example of a potential connection to the wastewater system. This requires further investigation i.e. dye testing to determine if it is connected to the sewer system.

Figure 5.18: Roof drainage connection

(Source: Wady, I (26/8/2014) Roof drainage connection.jpg)

Properties located adjacent to water courses are potential hot spots for infiltration. Within the St Georges Basin area Shoalhaven Water employee Mr Gavin Phillips located a manhole that gets inundated by the basin during large tides. In addition to this a deck had been constructed by the property owner restricting access to the manhole. Figure 5.19: Manhole impacted by tidal inundation clearly shows a build-up of seaweed. Whilst the manhole has a gatic lid installed the manhole rim is cracked and extensive of rust is present.

Figure 5.19: Manhole impacted by tidal inundation

(Source: Phillips, G (August, 2014) Manhole inundation.jpg)

Mr Gavin Phillips also located vertical risers that were located along the St Georges Basin foreshore that become inundated during tidal events. In addition he also commented that erosion to the foreshore was occurring as these vertical risers use to be located in grassed areas. Figure 5.20: Vertical riser subject to tidal inundation is one of the effected connection points. Whilst the lid is sealed there is a high potential that the vertical shaft is allowing I/I into the sewerage system.

Figure 5.20: Vertical Riser impact by tidal inundation

(Source: Phillips, G (August, 2014) Vertical Riser inundation.jpg)

Whilst Figure 5.19 and 5.20 show potential source of I/I the I/I is not a direct result of rainfall I/I. It does however demonstrate that field observations can provide valuable information towards determining the source of increased flows in sewer systems.

5.12. Conclusion

The ADWF is an average value and deviation from this value will occur daily. All 3 design methods used estimated similar ADWF's and the land use resulted in the largest deviation. Over a period of time land uses in a commercial and industrial area will change and thus so will the ADWF.

The 3 design methods used to estimate PWWF provide a substantial range of flows, with the Queensland traditional method resulting in the lowest PWWF and the NSW Public Works method resulting in the highest PWWF. The WSAA method was the only design method that provided a range of PWWF's. The leakage severity coefficient takes into account both aging network infrastructure and the impact that the soil aspect has on I/I. In addition it was the only method that incorporates actual rainfall into the design parameters.

The Shoalhaven Water reduction in Storm Allowance from 0.058 L/s to 0.030 L/s for new sewer systems needs to be reviewed. For Coastal locations i.e. Sussex Inlet this reduction is justified as the soil type in this area is sand and thus a low impact soil. Reducing the storm allowance value in subdivisions with a clay soil type may result in under calculation of the PWWF in the long term.

In order to be able to recognise, investigate and rectify I/I to the sewer system a multi facet approach needs to be undertaken. This includes having a SCADA system that is “Fit for Purpose” and a data base that easily summaries catchment characteristics such as pipe age, lengths and soil aspect. In addition long term sub catchment flow monitoring can be used to identify higher areas of I/I and also to measure the success of rectification measures. The impact of private properties also warrants further investigation, whilst reduction of I/I can be achieved on the network infrastructure there is substantial private networks contributing to the I/I issue.

Chapter 6: Discussion

The process followed enabled a review of 3 current design methods and the restrictions to identifying I/I to the sewerage system. To enable management of I/I a number of improvements to the current system have to be implemented, these improvements include the implementation of a SCADA system that is fit for purpose and annual review of all sewer catchments. Adoption of the WSAA design methodology incorporating local climatic conditions is also highly recommended. This will enable sewer networks to be monitored in the long term and also to set a containment standard that is based on best practice engineering guidelines.

6.1.Flow Analysis

The flow analysis highlighted several important factors that need to be taken into consideration. These factors include the variation to diurnal flow for different land uses and the fact that the current NSW Publics Works design method does not take into account the local climatic conditions or the soil aspect. A water utility needs to be able to review the system performance on a regular basis to detect changes in land use and to also enable forward planning of future upgrades.

The current SCADA system does not allow for engineering or management requirements and this restricts the ability to regularly review catchment performance. Substantial improvement to the SCADA system can be made, it is estimated with the present reporting system that up to 20 hours per SPS is required to establish an ADWF that is representative of the catchment. The current reporting that is available is highly inaccurate.

There also other benefits that can be achieved by having a suitable SCADA system. These benefits include customisation of maximum inflow times to allow early detection of network blockages and also allowing the pump performance to be monitored daily. This is able to be done with the knowledge of the period when minimum flow occurs and benchmarking the pump performance against the pump flows at these times.

Estimating the ADWF from 16 random dry days produced accurate results for residential areas however uncertainty still exists for industrial land uses that have intermittent irregular high discharges. Knowing the standard deviation of the ADWF also provides valuable operational information for when maintenance or emergency work is undertaken.

The peaking factor using the 3 design methods was consistent with the average peaking factor calculated for SPS 3 catchment. The variation in diurnal flow did however result in higher peaking factors being possible and it is important for operators of sewerage systems to know that the peaking factor is an average.

The adopted PWWF for a catchment is by far the most important component of the design and subsequent system operation. The traditional Queensland method resulted in the lowest estimation of PWWF whilst the NSW Public Works method resulted in the highest PWWF. Neither method took into account actual local rainfall or the soil aspect of the location. In addition they also did not set a containment standard based on the environmental criticality of the location. Whilst Shoalhaven Water has reduced the storm allowance for new systems this allowance has no real relationship to either rainfall or soil aspect. In locations with clay or rock stratum this may result in an under estimation of PWWF in the long term considering typical system design parameters range from 15 to 30 years.

The most efficient method for reviewing system performance was using the visual method provided by WSAA (Figure 4.4). This method, using Excel, provided the user with a visual interpretation of the diurnal curve and allowed easy identification of suspect data to be identified. It also showed the variability in the diurnal curve and readily identified the period of minimum flow and periods of irregular peak flow.

It is common knowledge that Australia experiences prolonged periods of dry weather. The current inability to regular review system performance may result in the hydraulic capacity of a catchment being exceeded prior to a wet weather event occurring. The high degree of variability in Australia’s climate may also mean that several wet weather events occur in a short space of time followed by an extended period of dry weather. By setting a containment standard the water utility will be able to justify exceedance of hydraulic capacity resulting in overflows during wet weather events for those events which do not occur regularly.

6.2.I/I Detection

There are a number of methods that can be used to detect I/I into the sewerage system. They include dye testing, CCTV, smoke testing, flow monitoring and manhole inspections. A combination of these methods needs to be used to detect sources of I/I. Each method has its advantages and disadvantages, for example smoke testing will locate illegal storm connections if there is no water seal in the private drainage lines.

For flow monitoring to be effective it needs to be installed for a long period of time to ensure that initially it identifies the problem areas but also to ensure that rectification measures have resolved the I/I source.

Manhole inspections can readily identify damaged components however it is important that field staff also take into account the potential for rainwater to pool over the top of the manhole lid.

CCTV can be effective in identifying defects with the network pipeline however it may not detect illegal stormwater connections if the connection occurs upstream of the sewer junction.

Dye testing can be effective but time consuming, there is only a limited amount of dye that can be used on stormwater lines before impacting on the visual aspect of the receiving discharge point i.e. waterway.

To ensure that the detection of I/I is effective and efficient a decision matrix needs to be developed to ensure that detection methods are customised to the catchment characteristics and a consistent methodology is followed. For example a new catchment may have rainwater tank connections that are causing the issue whereas for an old catchment the issue may be related to aging public and private infrastructure.

6.3.I/I Rectification

The majority of current rectification measures focus on relining network pipelines using various technologies or rehabilitation of manholes. They do not however address the fact that private drainage lines can account for a far large portion of the total network.

Sewer mains are typically laid to service 2 or more properties thus for 20m of sewer network there may be 2 or more junction connections. For each of these connections there is private pipework to the household. If each household has 20m of private drainage line then this means for 2 connection points the network sewer main only accounts for 33% of the total sewer network. Table 2.5 - Reduction of RDII for public sewer rehabilitation indicates for a 100% rehabilitation of the public network only a 60% reduction of I/I is achieved.

Private sewer lines are typically laid shallower and to a lower standard of construction. There also do not readily get repaired by the property owner until there is a blockage. The repair of the blockage, using an electric eel, can also result in further damage being done to the pipes integrity.

Yard gullies are a potential point source of direct inflow into sewer systems, whilst the yard gully is required to be 75mm above the surrounding ground this is often not the case. Often paved areas are made flush with yard gullies or the surrounding ground is slowly built up over time. As the purpose of a yard gully is to allow effluent to surcharge outside of the household, the yard gullies are open with only a grate to prevent debris from entering the sewer system.

A solution to the problem of inflow via yard gullies is a device known as an overflow relief cap (ORC). These are approved for use by AS 3500 – Plumbing and Drainage Code and can resulting in a substantial reduction in inflow to the sewerage system especially for low lying flood prone areas. Figure 6.1: Overflow Relief Cap shows that actual OCR and a yard gully with it installed. If the wastewater system needs to surcharge to cap lifts and pops out of the yard gully.

Figure 6.1: Overflow Relief Cap
(Source: ORC Technology, 2013 http://www.orctechnology.com.au)

Wide Bay Water Corporation in Queensland is currently working with Fraser Coast Regional Council to mandate the installation of ORC's for all new developments. In addition they are currently installing ORC's for all properties within the Toogoon Sewerage Scheme. (Wide Bay Water Corporation, 2013)

6.4. Resource Requirements and System Improvements

At present no efficient system is in place for reviewing the performance of sewer catchments on an annual basis. Sewer catchment performance needs to be regularly reviewed and ranked to ensure that resources are focused on those catchments most impacted by I/I. There may be existing catchments which are highly impacted by I/I however are not readily identified as the hydraulic capacity has not yet been exceeded i.e. the storm allowance for PWWF is set to high or the catchment is new and is not fully developed.

The current SCADA system is not "Fit for Purpose" for engineering and management needs. The SCADA system has the capability to provide the required functionality however this has not been implemented. A high level review of the SCADA system is currently being undertaken, a separate review of current staff skill sets needs to also be undertaken. This will ensure that implementation of changes to the SCADA system is done correctly and effectively. During the investigative stage of the report the comment was often made "*Management does not tell us what they want*". After 6 months of trying to get basic changes implemented it has become apparent that the SCADA staff either do not have the skill sets required to implement the changes or the resources to ensure that it is undertaken in an efficient and effective manner. A culture change from the way it has always been done needs to occur to enable improvement to the SCADA system to occur.

The current operational maintenance system works in a reactive fashion, thus as problems arise they are rectified. The development of a long term maintenance strategy will help focus the work that is being undertaken and ensure that catchments are identified that are utilising more resources than others for rectification works. A review of this system of operation needs to be undertaken to ensure that resources are being used effectively and the long term maintenance of the system is being done proactively.

The adoption of the WSAA design methodology will enable short and long term modelling of the catchment performance to be undertaken. It will also ensure that a suitable containment standard is set based on environmental criticality of the catchment. Furthermore classification of catchments based on soil aspect will ensure that the design PWWF is suitable for the catchment area.

Development of a catchment data base will enable a more efficient review of the catchment characteristics to be undertaken. At present considerable time is required to research the properties of each catchment i.e. pipe age, lengths, number of manholes, soil type and catchment area.

A pump station operational methodology needs to be implemented to ensure that each SPS is operating as per the design methodology. This also needs to be directly linked to the asset maintenance and the SCADA system. The development of a central database of pump performance, settings, maintenance and operational changes will ensure that a consistent approach is taken by both operational and planning staff. At present there is at least 10 separate excel spreadsheets which staff rely upon for operational setup, maintenance and upgrade works.

Implementation of a flow monitoring strategy will enable a more detailed review of catchment performance to be undertaken. At present relying on the flow data from the SPS highlights that issues are present. For large catchments it does not enable efficient and effective use of resources to undertake identification and rectification measures. In addition flow monitoring can provide the utility with a baseline to measure the success of rectification measures.

A private property strategy needs to be developed to enable the installation of overflow relief caps on private properties. This should also consider the requirement for mandatory installation for new homes. Furthermore the strategy should also consider options for rectification of private household drainage lines. Whilst the past focus for many water utilities has been on maintaining their network private properties may be a significant contributor to the overall issue of I/I. With the Nowra sewerage system originally being commissioned in 1937 a number of properties internal drainage line may be more than 80 years of age. In other regions of Australia these internal drainage lines may be more than 100 years of age. There is no mandatory requirement for internal drainage lines to be upgraded at present.

Installation of suitable weather stations and direct access to Council's rain gauges would assist live monitoring of the system. At present rainfall data needs to be either purchased from the BOM for each rain event or is required to be obtained from the Council Flood monitoring system which is only accessible by certain staff members. Rain gauges at the sewer treatment plants only record daily rainfall totals and the rain gauge at the South Nowra Operations depot was found to be out of calibration during the project period. This information needs to be readily available for both operational and planning staff.

6.5. Improvements to Design Guidelines

The current design guidelines can be improved by incorporating the local climatic conditions into the design process. The current NSW Public Works methodology has no relationship to rainfall. The results indicate that the calculated PWWF for SPS 3 was comparable to the maximum PWWF calculated using the WSAA method. For other areas whereby rainfall intensity is lower or higher the results would be different. Customising the local climatic conditions for each sewerage scheme will ensure that the design methodology follows best practice guidelines. In the Ulladulla region two storm events in the past 18 months have resulted in substantial rainfall in a short period of time, at the same time these rain events have been of considerable lower intensity at Culburra and Shoalhaven Heads.

The soil aspect was shown to have a considerable impact on the calculated PWWF, further refinement of this factor could be undertaken to identify between the different soils stratums i.e. sand to clay to rock. This will enable further customisation of the design PWWF.

6.6. Conclusion

The largest current restraint to identifying I/I is the SCADA system, improvements to this system are required to ensure that information is readily accessible and that the data is reliable.

Customisation of design values to local climatic conditions will ensure that the system is designed to the conditions of the local catchment. Having a detailed record of all system characteristics will ensure that long term monitoring of system performance can be readily undertaken. These system characteristics need to include the soil aspect and the environmental criticality of the local catchment. This will ensure that the design PWWF is appropriate and that a suitable containment standard is incorporated into the system design.

The impact that private properties have on the sewerage system during wet weather events needs to be further investigated. Whilst a number of improvements to the current design methodology, SCADA system and associated asset data bases have been identified the impact may be minimal without consideration the aging private infrastructure.

Chapter 7: Recommendations for Further Study

The customisation of local climatic conditions has many benefits and a comparison of other sewerage schemes within the Shoalhaven region and other regions within NSW needs to be undertaken to demonstrate the effectiveness of this design method.

The impact of inflow via yard gullies and infiltration to private household drainage lines also needs to be further investigated to determine the potential contribution to wet weather flow.

Whilst a lot of research has been undertaken to I/I into authority network systems, these systems are often installed to a higher standard. The impact of aging private drainage lines warrants further study and investigation. A cost benefit analysis into the remediation of private infrastructure and installation of inflow prevention devices such as the over relief cap will enable an informed decision to be made to the advantages / disadvantages of undertaking the works.

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Appendix A: Project Specification

ENG 4111/4112 Engineering Research Project
PROJECT SPECIFICATION

FOR: **IVAN WADY**

TOPIC: INFLOW / INFILTRATION STRATEGIC MANAGEMENT PROJECT

SUPERVISORS: Dr Vasantha Aravinthan
Carmel Krogh, Shoalhaven Water
Andrew McVey, Shoalhaven Water

ENROLMENT: ENG 4111 – S1, 2014
ENG 4112 – S2, 2014

PROJECT AIM: To develop a strategy to enable a proactive approach to identify and address the issue of inflow / infiltration into the sewerage network.

SPONSORSHIP: Shoalhaven Water

PROGRAMME: Issue A 10th March 2014

1. Conduct extensive literature review on the design guidelines from various states to identify how the Infiltration / Inflow (I/I) values are incorporated into the sewer design in practice and summarize them.
2. Research if any studies were conducted to customize the I/I values adopted in the sewer design that suit the local conditions
3. Identify a problematic catchment within Shoalhaven wastewater network and gather baseline date of ADWF during dry days, rainfall events and corresponding PWWF for the selected catchment
4. Develop methodology for field analysis of a known problematic catchment,
5. Trial / conduct an investigation during a wet weather event (subject to wet weather) using the developed methodology in 4.
6. Analyse field results to determine if strategy was successful in identifying the cause of pump station / manhole overflow during wet weather as inflow or infiltration
7. Development rectification measures to mitigate inflow / infiltration.
8. Suggest improvement to the design guidelines based on the gather data and field investigation.
9. Submit an academic dissertation on the findings.

As time permits

10. Trial strategy on an additional catchment.
11. Undertake mitigation measures on problematic catchment and rerun field analysis (subject to extent of problem and wet weather events)

AGREED:

Ivan Wady (Student) , Dr Vasantha Aravinthan (Supervisor)

Appendix B: Overview of Nowra Sewerage Scheme

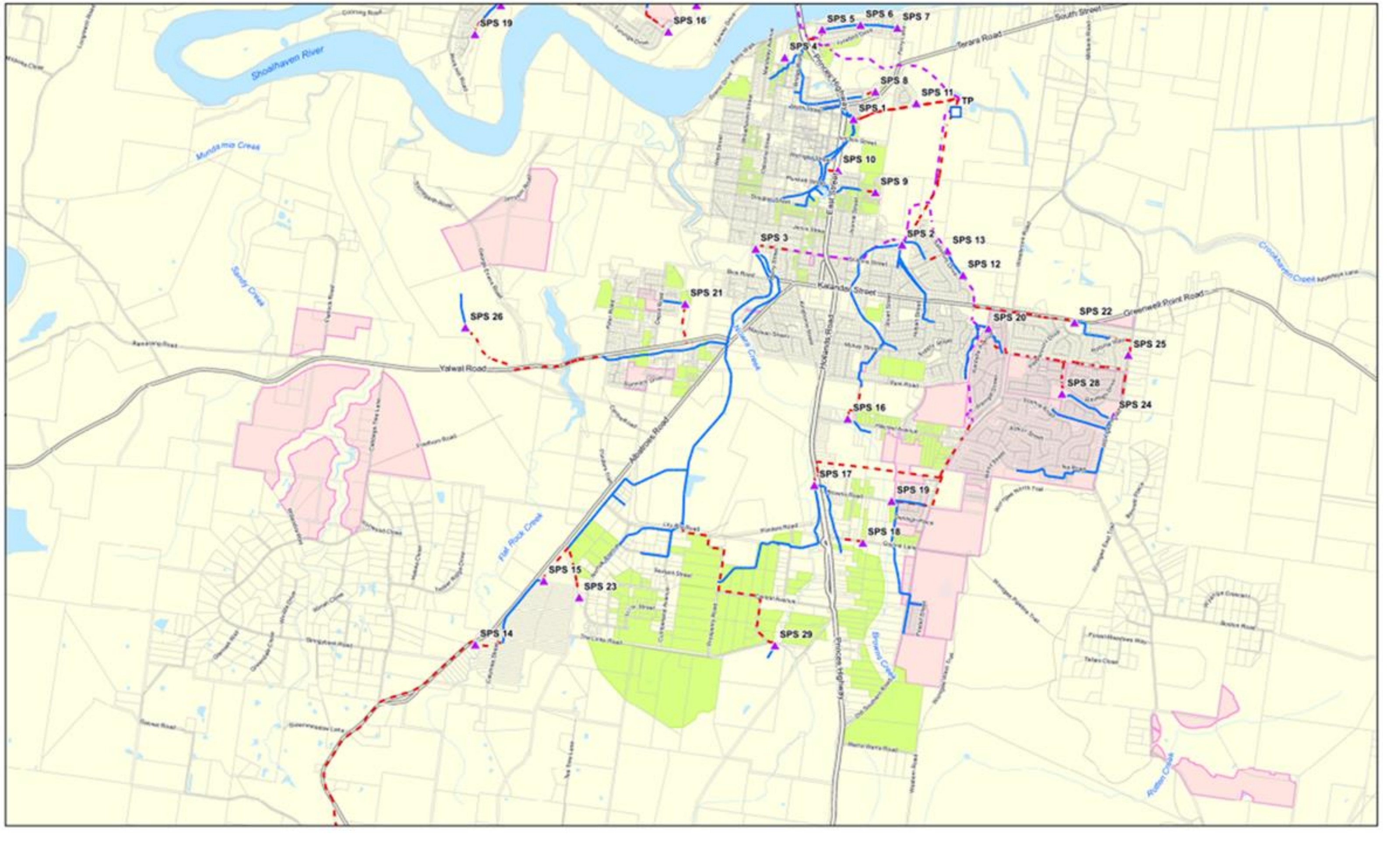


Figure B.1: Overview of Nowra Sewerage Scheme (Source: Shoalhaven Water, 2014)

Appendix C: Water Utility and Design Standard

Table C.1: Water Utility and Design Standard Summary

Authority	State	Standard Used			Reference	Website
Aulbury City Council	NSW	WSAA			(Albury City Council, 2014)	http://www.alburycity.nsw.gov.au
Baron Water	VIC	WSAA - MRWA			(Baron Water, 2014)	http://www.barwonwater.vic.gov.au
Bega Valley Shire Council	NSW	WSAA			(Bega Valley Shire Council, 2013)	http://www.begavalley.nsw.gov.au
Byron Bay Shire Council	NSW	WSAA	NSW Public Works	Aus Spec	(Byron Shire Council, 2013)	http://www.byron.nsw.gov.au
Calliope Shire Council	QLD	WSAA	QLD Traditional Method		(Calliope Shire Council, 2012)	http://www.gladstone.qld.gov.au
Central Highlands Water	VIC	WSAA - MRWA			(Central Highlands Water, 2014)	http://www.chw.net.au
City West Water	VIC	WSAA - MRWA			(City West Water, 2014)	http://www.citywestwater.com.au
Coffs Harbour City Council	NSW	NSW Public Works	Aus - SPEC		(Coffs Harbour City Council, 2013)	http://www.coffsharbour.nsw.gov.au
Coliban Water	VIC	WSA			(Coliban Water, 2011)	http://www.coliban.com.au
East Gippsland Water	VIC	WSAA - MRWA			(East Gippsland Water, 2011)	http://www.egwater.vic.gov.au
Gippsland Water	VIC	WSAA - MRWA			(Gippsland Water, 2007)	http://www.gippswater.com.au
Goulburn Valley Water	VIC	WSAA - MRWA			(Goulburn Valley Water, 2014)	http://www.gvwater.vic.gov.au
Port Macquarie Hastings Council	NSW	WSAA	NSW public Works	AUS SPEC	(Hastings Council, 2005)	http://www.pmhc.nsw.gov.au
Hunter Water	NSW	WSAA -Hunter Water Edition			(WSAA, 2014)	https://www.wsaa.asn.au
Mackay City Council	NSW	QLD Traditional (Modified)			(Mackay City Council, 2008)	http://www.mackay.qld.gov.au
Midcoast Water	NSW	NSW Public Works			(Midcoast Water, 2006)	http://www.midcoastwater.com.au
Pine Rivers Shire Council	QLD	Queensland Traditonal Method			(Pines River Shire Council, 2014)	https://www.moretonbay.qld.gov.au
Power And Water Northern Territory	NT	MWSDB (1979)			(PowerWater, 2014)	http://www.powerwater.com.au
SA Water	SA	WSA			(SA Water, 2010)	http://www.sawater.com.au
SEQ	QLD	WSA			(SEQ, 2013)	http://www.seqcode.com.au
Shoalhaven Water	NSW	WSA			(Shoalhaven Water, 2011)	http://www.shoalwater.nsw.gov.au
South East Water	VIC	WSAA - MRWA			(South East Water, 2014)	http://southeastwater.com.au
Sydney Water	NSW	WSAA - Sydney Water Edition			(WSAA, 2014)	https://www.wsaa.asn.au
Tas Water	Tas	WSAA - MRWA			(Tas Water, 2013)	http://www.taswater.com.au
Toowoomba Regional Council	QLD	WSAA			(Toowoomba Regional Council, 2014)	http://www.toowoombarc.qld.gov.au
Tweed Shire Council	NSW	AUS-Spec			(Tweed Shire Council, 2010)	http://www.tweed.nsw.gov.au
Wagga Wagga City Council	NSW	WSAA			(Wagga Wagga City Council, 2013)	http://www.wagga.nsw.gov.au
Wannonwater	VIC	WSAA - MRWA			(Wannonwater, 2010)	http://www.wannonwater.com.au
Western Water	VIC	WSAA - MRWA			(Western Water, 2013)	http://www.westernwater.com.au
Water Corporation	WA	No Information Available			(Water Corporation, 2014)	http://watercorporation.com.au

Appendix D: Sewer Design Code Equivalent Populations for Synchronous discharges

(Sources WSAA, 2002)

TABLE A1 EQUIVALENT POPULATIONS FOR SYNCHRONOUS* DISCHARGES

***Peaks coinciding with normal residential occupancies**

Classification	Unit	EP per Unit	Remarks
Residential			
Single occupancy lots	**Lot**	**3.5**	To be used for single occupancy lots down to 300 m^2
Single lot 1000m^2	Gross hectare	25	Approx 70% net which takes roads, parks etc into consideration
Single lot 500m^2	Gross hectare	50	
Single lot 300m^2	Gross hectare	80	
Multiple occupancy lots			
Single occupancy medium density dwelling units	**Dwelling unit**	**3.0**	To be used for multiple occupancy lots down to 300 m^2
Medium density (Group housing)	Gross hectare	120	Density of 40 dwelling units/ gross ha
Medium density e.g. 3 storey walk-up flats	Gross hectare	210	Density of 70 dwelling units/gross ha
Single occupancy high density dwelling units	**Dwelling unit**	**2.5**	
High density multi storey apartments	Gross hectare	375–4500	Depends on locality e.g. CBD in small capital city, CBD in Sydney, strip development along Gold Coast
Commercial/ Special Cases			
High density commercial	Gross lettable floor space, 10,000 m^2	500–800	Typical for capital city CBD
Local commercial	Gross hectare	75	
Educational institutions	Student	0.2	Includes teaching staff. Treat residential colleges and boarding houses as medium density dwelling units
General public entertainment facilities	Visitor	0.05	Shows, race crowds, etc
Clubs	Occupant	0.25	Use the maximum number of occupants for which the club facilities were designed
Hospitals and nursing homes	Available beds	3.4	Includes staff quarters
Parks / gardens / reserves	Gross hectare	20	
Golf courses	Gross hectare	10	Treat club houses as above
Future industrial areas	Gross hectare	150	To be used only when the future types of industry are unknown otherwise use Table A2

TABLE A2 NON-SYNCHRONOUS DISCHARGES
LIST OF INDUSTRIES AND THEIR EP CLASSIFICATIONS

INDUSTRY			EP Classification	Notes
Meal preparation	Non-Residential	Restaurant	8	
		Cafeteria	8	
		Canteen	8	
		Caterers	8	
Food manufacture	Dairy	Milk	2	
		Cheese, butter and yoghurt	4	
		Ice cream	6	
	Fruit and vegetable	Cannery	5	
		Condiments and sauces	5	
	Meat	Abattoir	5	
		Rendering tallow	6	
		Gelatine and glue	4	
		Poultry	3	1
		Small-goods	5	
	Grain	Flour milling	10	
		Starch	4	1
		Edible oils and fats	3	
		Cereals	7	
		Bakery	10	
		Biscuits and cakes	7	1
	Beverages	Beer	5	1
		Soft drinks and cordials	6	
	Others	Yeast	3	
		Confectionery	8	
		Salt	6	
Textile and leather	Tannery and hides		5	1
	Wool	Wool scour	3	1
		Felt and carpet	6	
		Dyeing and spinning	6	1
	Cotton and synthetics	Dyeing and spinning	5	1

continued

INDUSTRY			EP Classification	Notes
Chemical	Petrochemical	Oil refinery	10	(2)
	Pharmaceutical		7	
	Organic	Liquids	6	
		Resins, polymers and plastics	6	
		Adhesives	6	
	Others	Soaps and detergents	7	
		Paint manufacture	8	
Metal processing	Metal finishing	Electroplating	6	1
		Anodising	6	
		Galvanising	6	
	Battery manufacture	Dry cell	7	
		Wet cell (lead acid)	7	
	Engineering	Machine Shops	7	1
		Sheet Metal	7	1
		Foundry	7	1
		Rolling	5	1
		Extrusion	7	1
Manufacture —Non-Metallic	Paper		8	
	Plastics		6	
	Wood		8	
	Mining (Earth)	Glass	8	
		Fibre cement	9	
		Concrete products	5	
Services	Laboratories	Industrial and research	5	
	Laundries	Industrial	1	
	Hotels, motels etc.		6	3
	Others	Film Processing	6	1
Future Unknown				4

NOTES:

1 Some industries may have discharges much larger than usual. Discharges should be reviewed against available information for the development.

2 Building area not applicable. Total property area is used.

3 For high-rise city hotels and motels use Table A1 – High-density commercial areas.

4 Where the type of future industry is unknown use Table A1 – Future industrial areas.

Appendix E: Water Directorate NSW Standard ET

7 STANDARD ET FIGURES – RESIDENTIAL USER CATEGORIES

Table 1: Standard ET Figures - Residential User Categories

CATEGORY	STANDARD UNIT	SUGGESTED VALUES	
		WATER ET	**SEWER ET**
Single Residential Lots (House)			
Standard Residential Lot (450m^2 - 2000m^2)	Lot	**1.00**	**1.00**
Small Residential Lot (< 450m2)	Lot	**Use Units**	
Large Residential Lot (> 2000m2)	Lot	**1.20**	**1.00**
Multi-Residential Lots (Medium Density 1 - 2 Storey)			
Dual Occ - 1 bedroom	Dwelling	Use Units for Lot Size of < 450m2 / dwelling or Standard Residential Lot for Lot Size > or = 450m2 / dwelling	
Dual Occ - 2 bedroom	Dwelling		
Dual Occ - 3 bedroom (or more)	Dwelling		
Duplex - 1 bedroom	Dwelling		
Duplex - 2 bedroom	Dwelling		
Duplex - 3 bedroom (or more)	Dwelling		
Units - 1 bedroom	Dwelling	**0.40**	**0.50**
Units - 2 bedroom	Dwelling	**0.60**	**0.75**
Units - 3 bedroom (or more)	Dwelling	**0.80**	**1.00**
Multi-Residential Lots (High Density)			
Multi Storey Apartments (1 bedroom)	Dwelling	**0.33**	**0.50**
Multi Storey Apartments (2 bedroom)	Dwelling	**0.50**	**0.75**
Multi Storey Apartments (3 or more bedroom)	Dwelling	**0.67**	**1.00**

Notes
1 Standard ET = Town Water Usage of 230 kL/a & Sewage Loading of 140 kL/a

9 STANDARD ET FIGURES – INDUSTRIAL USER CATEGORIES (GENERAL)

Table 3: Standard ET Figures - Industrial User Categories*

CATEGORY	STANDARD UNIT	SUGGESTED VALUES	
		WATER ET	**SEWER ET****
Industrial General			
Light Industrial	Gross Ha	**15**	**15**
Future Unknown - Light	Gross Ha	**15**	**15**
Future Unknown - Med	Gross Ha	**30**	**30**
Future Unknown - Heavy	Gross Ha	**50**	**50**
* For industrial categories that have process water, it is suggested that a non-typical development ET is determined based on the methodology in Section 6.3 of the guidelines.			
** Additional ET figures for sewer have been included on Table 4, for a large range of detailed subcategories. These figures are provided for background information and should only be used as a general guide or where actual consumption data cannot be obtained.			

Notes
1 Standard ET = Town Water Usage of 230 kL/a & Sewage Loading of 140 kL/a
Assumed Residential Standard Discharge Factor: 60%
Gross Ha = Total land area of zone

8 STANDARD ET FIGURES – COMMERCIAL USER CATEGORIES

Table 2: Standard ET Figures - Commercial User Categories

CATEGORY	STANDARD UNIT	SUGGESTED VALUES		COMMENTS
		WATER ET	SEWER ET	
Accommodation (Permanent)				
Nursing Home / Special Care Home	Bed	**0.50**	**0.75**	*Limited medical facilities, communal kitchen / laundry*
Self Care Retirement Units / Villas	-	**Use Residential Units**		*Internal kitchen / laundry facilities*
Self Care Retirement - Serviced Unit (Onsite)	-	**Use Nursing Homes**		*No internal kitchen / laundry facilities*
Self Care Retirement - Serviced Unit (Offsite)	Bed	**0.30**	**0.45**	*No internal kitchen / laundry facilities*
Boarding House	Bed	**0.33**	**0.50**	*Communal kitchen / laundry*
Caravan / Mobile Home Park (1 br)	Van	**0.40**	**0.50**	
Caravan / Mobile Home Park (2 br)	Van	**0.60**	**0.75**	*Use if number of rooms unknown*
Caravan / Mobile Home Park (3 br)	Van	**0.80**	**1.00**	
Accommodation (Short Term)				**Peak week loading - use peak occupancy**
Caravan Park				
Camping Site (temporary)	Site	**0.50**	**0.63**	*Site approx. equivalent to average caravan site*
Caravan / Cabin Site (temporary)	Site	**0.50**	**0.63**	*Also use for on-site caravans / cabins*
Bed & Breakfast / Guest House	Room	**0.40**	**0.50**	*House based with communal kitchen / laundry*
Motel / Hotel / Resort Room	Room	**0.30**	**0.45**	*Consider food prep, entertainment & sporting separately*
Backpackers / Hostel	Bed	**0.15**	**0.23**	*Communal kitchen, small laundry, not serviced*
Serviced / Unserviced Apartments	-	**Use multi-res lots (high density)**		*Self contained (if not use motel)*
Accommodation (Medical Care)				
Hospital	Bed	**0.90**	**1.43**	*Includes medical facilities*
Hostel (Medical)	Bed	**0.70**	**1.11**	*Includes some medical facilities*
Business (Excluding Food Preparation)				
General				
Single Retail Shop	Floor Area m^2	**0.00**	**0.00**	
Supermarket	Floor Area m^2	**0.00**	**0.00**	*Includes minor food preparation*
Shopping Centre	-	**Insufficient Data**		*Consider amenities, food prep & specific bus. separately*
Offices	Floor Area m^2	**0.00**	**0.01**	
Specific				
Hairdresser / Beauty Salon	Basin	**0.50**	**0.79**	
Laundromat	Machine	**0.45**	**0.71**	
Medical Centre	Room	**0.40**	**0.63**	*Based on number of consultation rooms*
Plant Nursery	-	**Insufficient Data**		*Consider case by case*
Car Yard / Showroom	Floor Area m^2	**0.00**	**0.00**	
Service Station	Lane	**0.60**	**0.90**	
Car Wash	Lane	**5.70**	**9.03**	
Escort Agency	Room	**0.40**	**0.50**	

CATEGORY	STANDARD UNIT	SUGGESTED VALUES		COMMENTS
		WATER ET	SEWER ET	
Animal Boarding	-	Insufficient Data		*Consider case by case*
Self Storage	Floor Area m^2	0.00	0.01	*Consider office area only*
Food Preparation				
General				
Restaurant / Café	Floor Area m^2	0.01	0.01	
Take Away / Fast Food (no amenities)	Floor Area m^2	0.02	0.02	*Also use for general food preparation*
Take Away / Fast Food (including amenities)	Floor Area m^2	0.03	0.05	
Catering	Floor Area m^2	0.02	0.02	
Specific				
Bakery	-	Insufficient Data		*Use Take Away / Fast Food (non amenities)*
Butcher	-	Insufficient Data		*Use Take Away / Fast Food (non amenities)*
Fishing Co-op	-	Insufficient Data		*Use Take Away / Fast Food (non amenities)*
Entertainment				
Licensed Club	Floor Area m^2	Insufficient Data		*Separate into Food Preparation, Entertainment, Amenities*
Pub / Bar	Floor Area m^2	0.03	0.05	*Consider food preparation area separately*
Cinema / Theatre / Public Entertainment	-	Insufficient Data		*Use Food Preparation & Amenities*
Function / Conference Centre	-	Insufficient Data		*Use Food Preparation & Amenities*
Marina	Berth	0.60	0.90	
Sporting / Spectator Facilities				
General				
Amenities & Indoor Facilities	-	Insufficient Data		*Use Food Preparation & Amenities*
Specific				
Hockey Field (artificial surface)	-	Insufficient Data		*Consider case by case*
Bowling Alley	Lane	0.35	0.55	
Bowling Green	-	Insufficient Data		*Separate into Food Preparation, Amenities, Irrigation*
Swimming Pool - Indoor	ML	Insufficient Data		*Consider case by case*
Swimming Pool - Outdoor	ML	Insufficient Data		*Consider case by case*
Community Facilities				
Child Care Centre / Pre-school	Person	0.06	0.10	
Education - School (primary & secondary)	Person	0.03	0.05	
Education - College, University (tertiary)	Person	0.02	0.02	*Consider Food Preparation separately*
Correctional Centre	Person	0.50	0.75	
Church / Place of Worship	-	Insufficient Data		*Use Food Preparation & Amenities*
Community Centre / Hall	-	Insufficient Data		*Use Food Preparation & Amenities*
Parks / Gardens / Reserves (Irrigation)	-	Insufficient Data		*Consider case by case*
Public Amenities Block (per shower)	Shower	0.40	0.63	
Public Amenities Block (per wc)	wc	0.40	0.63	

Notes
1 Standard ET = Town Water Usage of 230 kL/a & Sewage Loading of 140 kL/a
Assumed Residential Standard Discharge Factor: 60%

Appendix F: SPS Gravity Pipeline Summary

F.1. Catchment 15

The pipe network for catchment 15 includes the gravity pipelines for upstream catchments 14 and 27. Catchment 27 represents the industrial component of HMAS Albatross with pipes in this catchment being up to 15 years of age. Approximately 60% of the pipework within catchment 15 is Asbestos Cement class C.

Table F.1: Catchment 15 pipeline summary shows 1105m of UPVC – White Class 12 have been installed in this catchment. This specific class of pipework has been problematic in the Shoalhaven Region with longitudinal breaks occurring in sections of water mains.

Table F.1: Catchment 15 Pipeline Summary

Catchment 15	Pipe Size (mm)	Pipe Length (m) / Age (Yr's)										Total
		5	10	15	20	25	30	35	40	45	55	Length (m)
AC/C	150				4				2072			2076
UPVC	150		355									355
UPVC-White/12	150			1105								1105
Total (m)		0	355	1105	4	0	0	0	2072	0	0	3536

F.2. Catchment 21

The pipe network for catchment 21 ranges in age from less than 5 years up to 35 years with approximately 80% of the pipework being 20 years or older. In the past 5 years the network has grown 13%. Table F.2: Catchment 21 Pipeline Summary details the varying age of the infrastructure in this catchment.

Table F.2: Catchment 21 Pipeline Summary

Catchment 21	Pipe Size (mm)	Pipe Length (m) / Age (Yr's)										Total
		5	10	15	20	25	30	35	40	45	55	Length (m)
PVCP	150		15		1414	497	1245	867				4038
	225							237				237
UPVC	150	658	87									745
Total (m)		658	102	0	1414	497	1245	1104	0	0	0	5020

F.3. Catchment 23

The pipe network for catchment 23 services the South Nowra industrial precinct. Only minimal land expansion has occurred within this network in the past 25 years. Development of the parcels of land along with changes in land use will continue to occur. Table F.3: - Catchment 23 Pipeline Summary provides details of the infrastructure servicing this catchment.

Table F.3: Catchment 23 Pipeline Summary

Catchment 23	Pipe Size (mm)	Pipe Length (m) / Age (Yr's)										Total
		5	10	15	20	25	30	35	40	45	55	Length (m)
PVCP	150					1837						1837
UPVC	150		101	19								120
Total (m)		0	101	19	0	1837	0	0	0	0	0	1957

F.4. Catchment 26

The pipe network for catchment 26 is relatively new with 66% of the pipeline being 15 years of age and 33% up to 5 years in age. The is potential for this catchment to experience further growth in the future both from expansion of the University campus and from planned residential growth. Table F.4: Catchment 26 Pipeline Summary details the relative new nature of the infrastructure in this catchment.

Table F.4: Catchment 26 Pipeline Summary

Catchment 26	Pipe Size (mm)	Pipe Length (m) / Age (Yr's)										Total
		5	10	15	20	25	30	35	40	45	55	Length (m)
UPVC	150	296		320								616
	225	296										296
Total (m)		592	0	320	0	0	0	0	0	0	0	912

F.5. Catchment 29

The pipeline network for catchment 29 was installed to service the South Coast correctional facility. Further expansion of this network will occur if a proposed industrial subdivision occurs. At present there is 135m of 300mm UPVC less than 5 years of age. There are extensive private drainage lines servicing the facility.

Appendix G: SPS Pump Performance and Identifier

The pump performance for each SPS is provided in Table G.1: Pump Performance and Identifier. The pump identifier is used in the Excel processing of the data.

A review of the pump performance also identified issues with SPS 29, the design flow rates of the pumps is 12 L/s. This required further investigation which revealed the pump impellors were warn and a large quantity of gravel was also found in the base of the SPS well. Further investigation revealed that gravel was partially choking the discharge pipework. The gravel had been in the system for a considerable amount of time and resembled river stone.

These issues have now been rectified however it reinforces the importance of having a SCADA system that allows regular review of SPS performance to be monitored. The development of a standard SPS operating methodology would also ensure that SPS's are set to operate as per the hydraulic design i.e. dual pump mode has no benefit with long rising mains and low flow pumps.

Table G.1: Pump Performance and Identifier

SPS	Pump Performance	Flow (L/s)	Identifier	Comment
3	Pump 1 Low Speed	49.3	PS3P1LS	Low Speed Pump 1
	Pump 2 Low Speed	49.3	PS3P2LS	Low Speed Pump 2
	Pump 1 High Speed	74.3	PS3P1HS	High Speed Pump 1
	Pump 2 High Speed	74.3	PS3P2HS	High Speed Pump 2
15	Pump 1	4.6	PS15P1	Pump 1
	Pump 2	4.6	PS15P2	Pump 2
	Pump 1 and 2	7.1	PS15P12	Pump 1 & 2 Combined
21	Pump 1	12.2	PS21P1	Pump 1
	Pump 2	12.2	PS21P2	Pump 2
	Pump 1 and 2	13.4	PS21P12	Pump 1 & 2 Combined
23	Pump 1	12.1	PS23P1	Pump 1
	Pump 2	12.8	PS23P2	Pump 2
	Pump 1 and 2	18.3	PS23P12	Pump 1 & 2 Combined
26	Pump 1 Low Speed	8.8	PS26P1	Pump 1
	Pump 2 Low Speed	8.8	PS26P2	Pump 2
	Pump 1 and 2	11.5	PS26P12	Pump 1 & 2 Combined
29	Pump 1 Low Speed	6.7	PS29P1	Pump 1
	Pump 2 Low Speed	6.7	PS29P2	Pump 2
	Pump 1 and 2	6.7	PS29P12	Pump 1 & 2 Combined

Appendix H: SPS SCADA Graphs

H.1. SPS 15

The SCADA data for SPS 15 was graphed for the entire period. Figure H.1: SPS 15 SCADA Flow Data illustrates a period of suspect data. This period is consistent with the suspect period identified for SPS 3. This period of data was excluded from the data set.

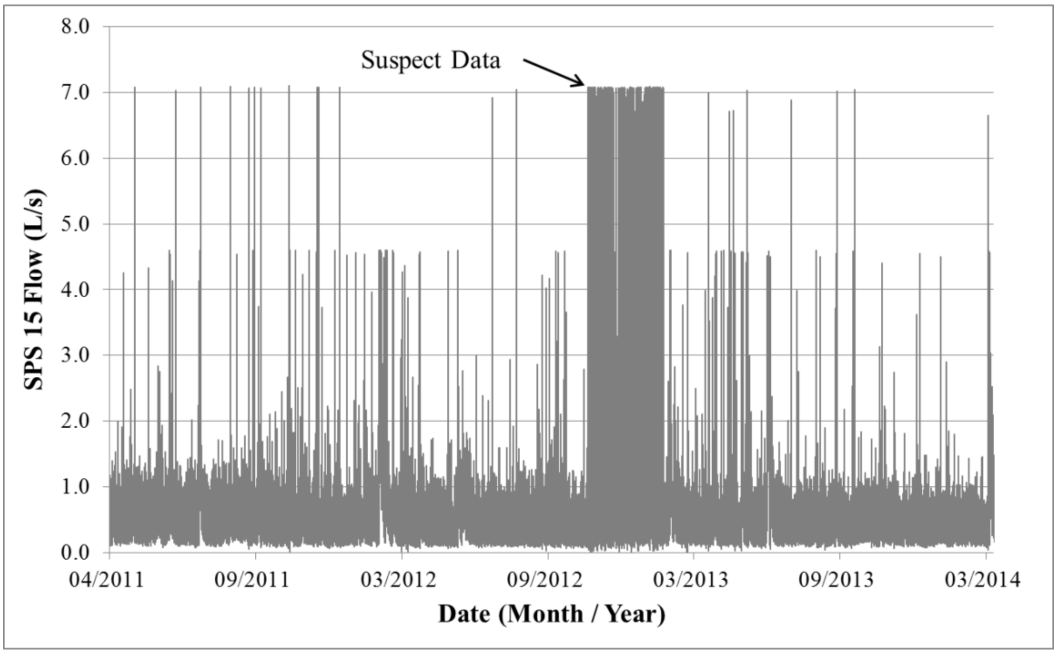


Figure H.1: SPS 15 SCADA Flow Data

H.2. SPS 21

The SCADA data for SPS 21 was graphed for the entire period. Figure H.2: SPS 21 SCADA Flow Data illustrates a period of suspect data. This period is consistent with the suspect period identified for SPS 3. This period of data was excluded from the data set.

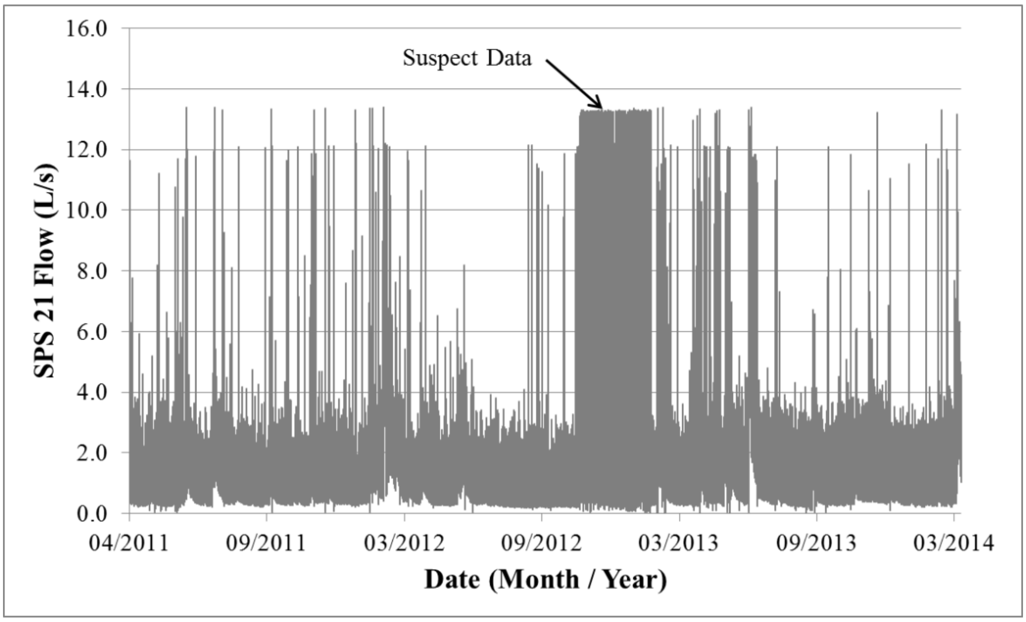


Figure H.2: SPS 21 SCADA Flow Data

H.3. SPS 23

The SCADA data for SPS 23 was graphed for the entire period. Figure H.3: SPS 23 SCADA Flow Data illustrates a period of suspect data. This period is consistent with the suspect period identified for SPS 3. This period of data was excluded from the data set. An additional period was also identified of missing data from 1st July to 8th July 2013. This period of missing data was also excluded from the SPS 3 data set.

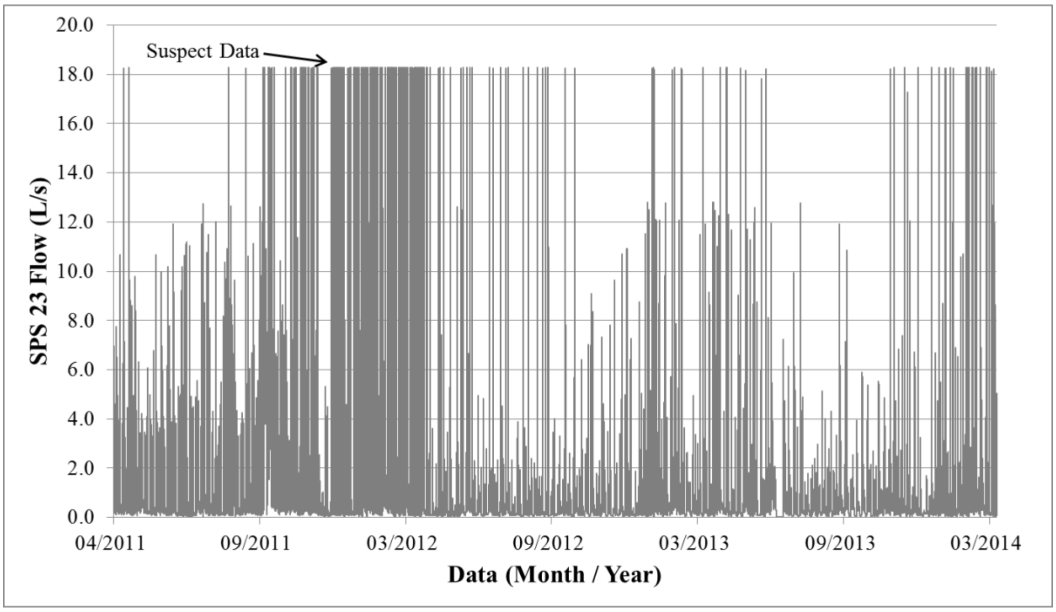


Figure H.3: SPS 23 SCADA Flow Data

H.4. SPS 26

The SCADA data for SPS 26 was graphed for the entire period. Figure H4: SPS 26 SCADA Flow Data illustrates a period of suspect data. This period is consistent with the suspect period identified for SPS 3. This period of data was excluded from the data set. It is also observed that the average flow is less than 1 L/s with regular peaks of up to 8 L/s.

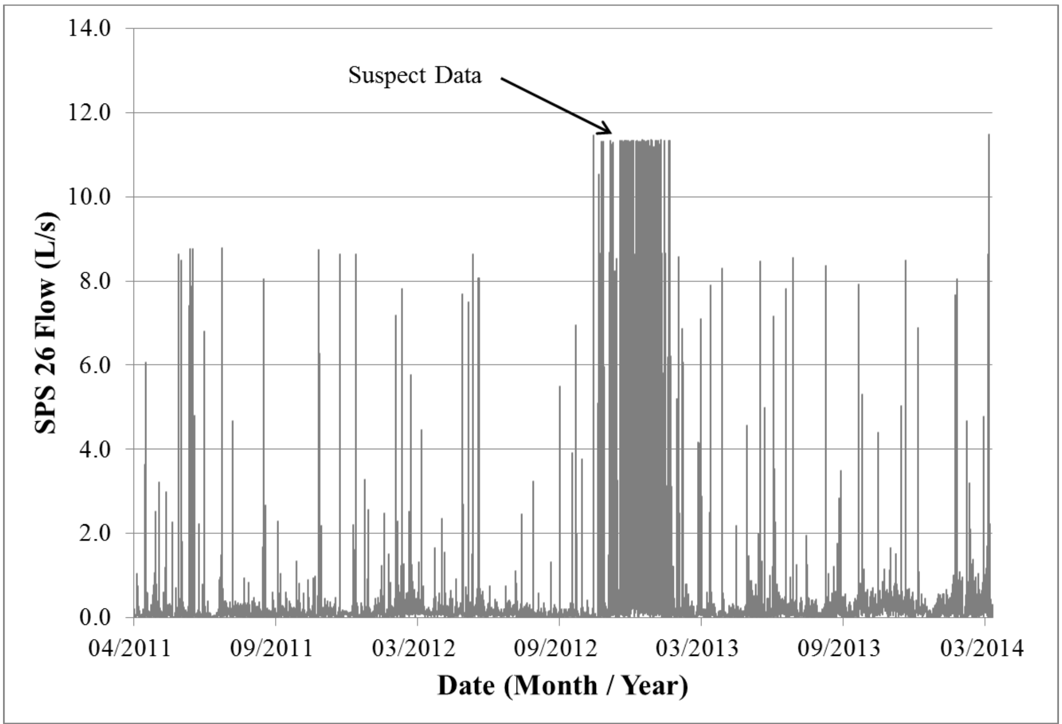


Figure H.4: SPS 26 SCADA Flow Data

H.5. SPS 29

The SCADA data for SPS 29 was graphed for the entire period. Figure H.5: SPS 29 SCADA Flow Data illustrates a period of suspect data. This period is consistent with the suspect period identified for SPS 3. This period of data was excluded from the data set. It is also observed that there is a gradual increase in peak flows during the analysis period.

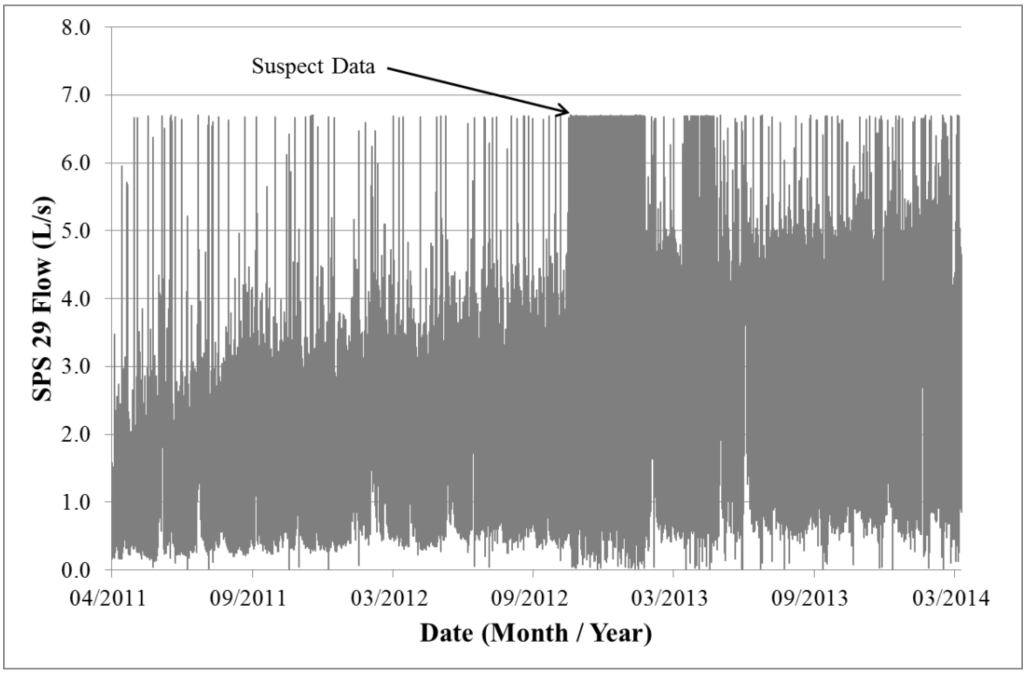


Figure H.5: SPS 29 SCADA Flow Data

Appendix I: Rainfall Data

Daily Rainfall (millimetres)

NOWRA RAN AIR STATION AWS

Station Number: 068072 · State: NSW · Opened: 2000 · Status: Open · Latitude: 34.95°S · Longitude: 150.54°E · Elevation: 109 m

2011	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	*0*	*0*	*1.4*	*0*	*1.0*	*4.8*	*0*	*0*	*0*	*0*	*0.2*	*2.6*
2nd	*0*	*0*	*0*	*0*	*0.2*	*2.0*	*0*	*0*	*0*	*28.0*	*0*	*3.6*
3rd	*1.6*	*15.8*	*0*	*0*	*8.0*	*0.4*	*0.2*	*0*	*0*	*16.4*	*32.2*	*0*
4th	*1.6*	*0.2*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*2.2*	*6.8*
5th	*0*	*6.0*	*0.2*	*11.0*	*0*	*1.8*	*0*	*0*	*0.2*	*0*	*0*	*0.2*
6th	*1.8*	*0*	*0.8*	*18.2*	*0*	*0*	*0*	*0*	*0.2*	*0*	*0*	*3.6*
7th	*0*	*3.2*	*0*	*0.2*	*0*	*0.2*	*0*	*5.4*	*4.6*	*8.0*	*0*	*1.8*
8th	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*7.6*	*0.2*	*3.8*
9th	*27.2*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1.4*	*0.2*	*10.0*	*1.0*
10th	*31.0*	*6.4*	*0*	*0*	*1.6*	*0*	*0*	*0*	*0.4*	*0*	*8.6*	*0*
11th	*11.4*	*0*	*0.6*	*0*	*0*	*1.4*	*0*	*0*	*0*	*0*	*1.0*	*0*
12th	*1.6*	*11.2*	*0*	*0*	*0*	*2.8*	*0*	*10.6*	*4.4*	*1.6*	*0*	*8.4*
13th	*0*	*0*	*0*	*0*	*0*	*3.4*	*0*	*0.2*	*0*	*0.4*	*8.6*	*1.6*
14th	*0*	*0.2*	*0*	*0*	*0.6*	*36.8*	*0*	*0*	*0*	*0*	*0*	*0*
15th	*0*	*0*	*2.8*	*0*	*0*	*24.2*	*0*	*0.2*	*0*	*0.6*	*0*	*0*
16th	*0*	*1.0*	*0*	*9.4*	*0*	*0.4*	*0.4*	*0*	*0*	*0*	*0*	*0*
17th	*0*	*6.6*	*2.6*	*0.4*	*0*	*0.2*	*1.4*	*0*	*0*	*0*	*1.2*	*0*
18th	*0*	*7.6*	*0*	*0*	*0*	*0*	*0*	*17.4*	*0*	*0*	*0*	*0*
19th	*0*	*0*	*15.6*	*0*	*0*	*0*	*1.4*	*4.2*	*0*	*0*	*0*	*3.6*
20th	*0.4*	*0*	*57.4*	*0*	*0*	*0*	*48.2*	*6.0*	*0*	*0*	*0*	*43.8*
21st	*0*	*0.4*	*45.6*	*0*	*0*	*0*	*21.6*	*5.2*	*0*	*0*	*0.4*	*0.6*
22nd	*5.6*	*0*	*37.2*	*0*	*0*	*0*	*47.6*	*2.2*	*0*	*0*	*0*	*0*
23rd	*0*	*1.4*	*0*	*7.4*	*0*	*0*	*20.8*	*0*	*0*	*0*	*9.6*	*0*
24th	*0*	*0*	*0*	*0.2*	*0*	*0*	*0.2*	*0.2*	*0.4*	*0*	*4.0*	*0*
25th	*0*	*0.2*	*0*	*0.2*	*5.4*	*0*	*0.4*	*0*	*21.0*	*4.8*		*0*
26th	*0*	*0*	*0*	*7.8*	*0*	*0*	*0*	*0*	*19.8*	*13.6*		*0*
27th	*0*	*1.2*	*0*	*0.6*	*0*	*0*	*0.2*	*0*	*0*	*2.2*	*2.2*	*0.6*
28th	*0*	*3.8*	*2.2*	*1.0*	*0*	*0*	*0*	*0*	*0*	*0*	*0*	*1.2*
29th	*0*		*0*	*1.4*	*0*	*0*	*0*	*0*	*2.2*	*0*	*0*	*0*
30th	*0*		*0*	*9.4*	*11.4*	*0.6*	*0*	*0*	*0.4*	*5.8*	*0.2*	*0*
31st	*0*		*0*		*24.0*		*0*	*0*		*0*		*0*
Highest daily	*31.0*	*15.8*	*57.4*	*18.2*	*24.0*	*36.8*	*48.2*	*17.4*	*21.0*	*28.0*	*32.2*	*43.8*
Monthly Total	*82.2*	*65.2*	*166.4*	*67.2*	*52.2*	*79.0*	*142.4*	*51.6*	*55.0*	*89.2*	*80.6*	*83.2*

Annual total for 2011 = *1014.2*mm

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Daily Rainfall (millimetres)

NOWRA RAN AIR STATION AWS

Station Number: 068072 · State: NSW · Opened: 2000 · Status: Open · Latitude: 34.95°S · Longitude: 150.54°E · Elevation: 109 m

2012	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	0	1.4	87.4	0		0	0	0	2.2	0	0	0.8
2nd	0	0.8	18.4	17.8	0	3.2	0	0.2	0	0	0	0
3rd	0	17.2	2.8	0.2	0.4	8.4	0	0	0	0	0	1.6
4th	2.2	20.4	9.0	0	0	0.4	0	0	0	0	0.6	0
5th	1.0	0	3.8	0.2	0	1.8	0	0	0	↓	0	0
6th	0.2	0	2.4	0	0	56.8	12.2	0	0	3.6	0	0
7th	0	0.6	1.4	0	0	4.0	0	0	0	2.8	3.2	0
8th	10.2	3.8	90.2	0	0	0	0.2	0	0	0	1.0	0
9th	10.2	2.0	6.2	0	0	0	0.2	0	0	4.4	0	0
10th	0	11.0	17.6	1.2	0	0.2	0	0	0	0	0	0.4
11th	0	43.4	0	0	0	10.2	3.6	0	0	1.4	0	0
12th	0	1.2	0	0.4	0	8.0	0	1.6	0	19.2	0	1.4
13th	0	3.4	0	0	0	1.8	2.6	0	0	12.6	0	0
14th	1.4	0	0	0	2.2	3.0	0	0	13.2	5.8	5.2	0
15th	0.6	0.2	0	0	0	0	0	0	0	0	0	0.2
16th	2.0	0	0	0	0	1.2	0	0	0	0	2.0	0.8
17th	0.4	0	58.4	0	0	3.6	0	0	0	0	2.2	0
18th		10.0	0.6	28.0	0.2	0	0	0	0	0	0	0.8
19th	0	0	4.6	37.0	0	0	0.2	0	1.8	0	2.2	0
20th	0	7.4	2.2	3.4	1.6	0	0	0	0	0	14.0	0
21st	0.4	8.6	0	0.2	0	0	0	0	0.6	0	0	0
22nd	1.0	0	0	0	0	0	2.8	0	0	0	0	0
23rd	3.6	0	0	0.2	0	0	3.2	0	0	0.8	0	0
24th	1.6	0	0	0.4	0	0	0	6.6	0	0	0	1.0
25th	1.0	0	2.6	0	13.8	0	0	0	0	0	0	19.2
26th	4.4	0.2	0	0	0	2.0	0	0	0	0	0	0.4
27th	10.0	0.2	0	0	0	0	7.8	0	0	0	0	0
28th	2.0	4.2	6.8	0	0	0.2	0	0	0	0	12.4	0
29th	0	44.4	5.0	0	0	0	0	0	0.8	0	0.2	0
30th	0.4		0.2		0	0	0	0	0	0	0	0
31st	0		0.2		0		0	0		0		0
Highest daily	10.2	44.4	90.2	37.0	13.8	56.8	12.2	6.6	13.2	19.2	14.0	19.2
Monthly Total	52.6	180.4	319.8	89.0	18.2	104.8	32.8	8.4	18.6	50.6	43.0	26.6

Annual total for 2012 = 944.8mm

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Daily Rainfall (millimetres)

NOWRA RAN AIR STATION AWS

Station Number: 068072 · State: NSW · Opened: 2000 · Status: Open · Latitude: 34.95°S · Longitude: 150.54°E · Elevation: 109 m

2013	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	0	0	30.6	10.2	0	0	0.4	0	0	0	0	0
2nd	0	18.4	2.2	0	0	19.4	0	0.2	0	0	0	0
3rd	1.0	0.2	1.2	0.8	1.6	2.0	0	0	0	1.0		0
4th	0	1.4	0	3.0	0	0	0	0	0	1.0		0
5th	0	2.0	0	1.6	0	0	0	0	0	0		5.4
6th	0	0	0	1.8	0	0	0	0	0	0	0	0
7th		0	0	0.2	0	0	0	0	0	0	0	0
8th		0	0	0	0	3.8	0	2.0	0.4	0	0	0
9th	0	0	0.2	0.2	0.2	0.2	4.8	4.8	0.4	0	0	0
10th	0	0	0	0.2	0	0	0	0	0	0	0.6	0
11th	0	6.2	0	0	0.2	1.6	0.2	0	0	0	30.0	0
12th	0	0	0	0	0	0.2	0.2	0	0	0	33.8	0
13th	0	0	0	0	0	3.0	0	0	0	0	9.0	0
14th	6.4	0	5.6	0	0.2	0	0	0	17.8	0	0	0
15th	0.2	0	0	0	0	0.8	1.8	0	2.6	2.2	0	7.2
16th	0	0.6	0	5.8	0	0	5.8	0	0	0	0.2	0.2
17th	0	4.2	0	0	0	0	0	0	73.2	0	16.0	1.6
18th	0	0.8	0	0	0	0	0.4	0	2.2	0	2.4	0
19th	0.2	0	0	7.0	0	1.8	0	0	0.4	0	1.8	0
20th	0	0	0	74.2	0	0.4	6.8	0	0	0	0	0
21st	0	1.4	0	8.2	0	0	0.4		0	0	0	0.2
22nd	0	3.0	0	1.2	0	0.2	0	0	0	1.6	6.6	1.0
23rd	3.8	13.8	8.8	0	36.4	14.2	0	0	1.0	5.4	1.2	0
24th	0	93.6	0	0	15.6	45.0	0	0	0	0	0	11.0
25th	0	14.2	0	0	9.6	159.0	0	0	0	0	4.0	0
26th	0	1.2	0	0	0	66.0	0	0	0	0	9.4	31.0
27th	17.6	0	0	0	0	3.2	0	0	0	0	0	5.2
28th	24.2	8.8	0	0	5.8	3.4	0	0	0	0	0	0
29th	77.2		0	0	0	10.4	0	0	0	0	1.4	0
30th	0.6		0	2.8	0	3.0	0	0.6	0	0	13.2	0
31st	0		0		0		26.2	0		0		0
Highest daily	77.2	93.6	30.6	74.2	36.4	159.0	26.2	4.8	73.2	5.4	33.8	31.0
Monthly Total	131.2	169.8	48.6	117.2	69.6	337.6	47.0	7.6	98.0	11.2	129.6	62.8

Annual total for 2013 = 1230.2mm

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Daily Rainfall (millimetres)

NOWRA RAN AIR STATION AWS

Station Number: 068072 · State: NSW · Opened: 2000 · Status: Open · Latitude: 34.95°S · Longitude: 150.54°E · Elevation: 109 m

2014	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1st	0	0	12.2	0	0	7.6	0	0	0			
2nd	0.6	0	5.8	0	0	25.6	0	0	0			
3rd	0	0	0	0.2	0.4	0	0	0	10.2			
4th	0	2.2	0	2.6	1.0	0	0	0	0			
5th	0	1.4	0	2.8	0	0	0	0	5.6			
6th	0	0	0	0	0	11.8	0	0	17.0			
7th	0	0	0	0.4	0	0	0	0	0			
8th	4.4	0	0	0.2	0	0	0	3.2	0			
9th	0.2	0	0.8	0	0	0	0	0	0			
10th	0	0	0.8	1.6	0	11.0	0.6	0	4.8			
11th	0	0	0	10.4	1.6	8.8	0	0	0.2			
12th	0	1.6	0	10.6	0.2	0	0	0	0			
13th	0	1.0	6.8	7.0	0	0.2	0.6	0	0			
14th	0		0	0	0.2	2.4	0	0	0.2			
15th	0		0	0.8	0	4.2	0.4	0	0			
16th	0		0	0.6	0	3.0	1.0	0.2	2.8			
17th	0		0	0	0	0	0.6	45.6	0			
18th	0		0	0	0	0	0	71.6				
19th	0		0	0	0	0.2	0	77.6				
20th	5.2		2.0	0	0	0	0	5.0				
21st	3.6	0	0	0	0	0	0	2.0				
22nd	1.2	0.6	0	0	0	0	0	0.8				
23rd	1.0	0.4	1.2	0	0	0	0	1.8				
24th	0	1.0	0	0	0	0	0	1.2				
25th	18.6	0.4	158.0	1.8	0	0	0.2	0.2				
26th	0	0	48.0	0	0	0	0	5.6				
27th	0	2.2	26.6	0	0	0	0	38.8				
28th	0	1.0	0.2	0	12.4	0	0	4.0				
29th	0		2.8	0	0	0	0	1.6				
30th	0		0	0.2	0	0	0	0.2				
31st	0		5.2		1.2		0	0				
Highest daily	18.6	2.2	158.0	10.6	12.4	25.6	1.0	77.6	17.0			
Monthly Total	34.8		270.4	39.2	17.0	74.8	3.4	259.4				

↓ This day is part of an accumulated total
Quality control: 12.3 Done & acceptable, *12.3* Not completed or unknown

Appendix J: Maximum Inflow Time

J.1. SPS 3

As there are no formal records of periods when a SPS is affected by power outage, maintenance work etc. it was adopted that a statistical analysis of the maximum inflow period would be used to quality check the data sets. For the purpose of the analysis the inflow period is between pump runs i.e. pump off to pump on.

The maximum inflow period for 16 random dry weekdays and weekend days was adopted as adequate or the analysis. The results of the analysis for SPS 3 are provided in Table J.1: SPS 3 Maximum Inflow Time. For SPS 3 it was determined that during a weekday an inflow period of up to 11,216 seconds was suitable whilst for a weekend this time frame reduced to 10,306 seconds. These time frames equate to approximately 3hrs with the majority of the maximum inflow periods occurring between 12am and 6am. The average maximum inflow period was calculated to be 1hr 45 minutes during weekends and 2hrs 10 minutes for weekdays.

Table J.1: SPS 3 Maximum Inflow Time

SPS 3				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 6:27	8800		21/05/2011 5:46	7508
27/06/2011 6:36	6356		25/06/2011 5:04	5706
21/09/2011 4:44	9418		31/07/2011 6:35	5422
10/08/2012 5:05	7541		17/09/2011 6:52	7550
20/08/2012 5:01	8329		23/10/2011 7:37	7869
2/10/2012 6:16	9310		15/04/2012 4:28	5429
26/10/2012 6:04	7789		28/04/2012 4:38	5519
21/03/2013 6:44	8955		23/06/2012 4:33	5228
10/05/2013 6:08	7879		18/08/2012 4:59	8243
20/05/2013 4:36	9097		21/10/2012 8:01	8657
23/08/2013 5:32	7576		13/04/2013 5:10	6725
27/09/2013 5:51	7787		17/08/2013 5:38	6224
9/10/2013 5:02	5821		28/09/2013 5:02	6004
12/12/2013 3:24	5789		13/10/2013 6:17	6176
15/01/2014 6:02	6803		18/01/2014 6:08	5302
19/02/2014 4:31	6804		8/02/2014 4:05	3353
Average	7753		Average	6307
Std Dev	1154		Std Dev	1333
3 x Std Dev	11216		3 x Std Dev	10306

The full data set was reviewed and 17 days were deemed not suitable for further analysis. Table J.2: SPS 3 Inflow Exceedance Days tabulates the results; two of the rejected days were classed as wet weather days. For weekdays 10 days were not suitable whilst for weekends 7 days were deemed not suitable.

Table J.2: SPS 3 Inflow Exceedance Days

SPS 3					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
12/05/2011 5:42	10655	3.0	Thursday	0	Dry Weekday
28/06/2011 5:11	15035	4.2	Tuesday	0	Dry Weekday
13/07/2011 5:35	10636	3.0	Wednesday	0	Dry Weekday
12/11/2011 7:42	18589	5.2	Saturday	0	Dry Weekend
4/01/2012 6:36	22061	6.1	Wednesday	2.2	Dry Weekday
5/01/2012 16:09	10324	2.9	Thursday	1	Dry Weekday
2/03/2012 8:29	30355	8.4	Friday	18.4	Wet weekday
7/10/2012 4:45	10916	3.0	Sunday	2.8	Dry Weekend
28/10/2012 9:15	17815	4.9	Sunday	0	Dry Weekend
15/01/2013 4:30	14510	4.0	Tuesday	0.2	Dry Weekday
23/03/2013 5:53	13347	3.7	Saturday	8.8	Wet weekend
24/03/2013 5:17	16189	4.5	Sunday	0	Dry Weekend
13/08/2013 8:12	19969	5.5	Tuesday	0	Dry Weekday
7/11/2013 4:57	21082	5.9	Thursday	0	Dry Weekday
8/11/2013 1:41	10548	2.9	Friday	0	Dry Weekday
11/01/2014 8:35	12882	3.6	Saturday	0	Dry Weekend
12/01/2014 7:23	18838	5.2	Sunday	0	Dry Weekend

There were 3 two day periods which were identified as exceeding the maximum allowable inflow time. Inflow exceedance days from the upstream SPSs were also removed from SPS 3 data set.

J.2.SPS 15

The maximum allowable inflow period adopted for SPS 15 was 21551 seconds and 23176 seconds for weekdays and weekend days respectively. This is approximately 6.5 hours for a weekday which is more than double the allowable time adopted for SPS 3. Table J3: SPS 15 Maximum Inflow Time used alternative random days to SPS 3 for weekdays to test the statistical methodology being used. The results for weekdays and weekends were similar with weekends having longer periods of inflow being acceptable. It can also be seen in Table J.3 that on the 2/10/2012 the maximum inflow occurred between 2.23pm and 6.06pm.

Table J.3: SPS 15 Maximum Inflow Time

SPS 15				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 3:55	8656		21/05/2011 22:48	6226
27/06/2011 5:56	13759		25/06/2011 4:47	15219
21/09/2011 5:25	14456		31/07/2011 5:43	17040
10/05/2012 4:22	17016		17/09/2011 2:46	8124
10/08/2012 3:39	15029		23/10/2011 5:30	14435
20/08/2012 5:17	9934		15/04/2012 2:45	12666
2/10/2012 18:06	13413		28/04/2012 1:18	4463
26/10/2012 4:44	12615		23/06/2012 3:35	12084
21/03/2013 6:39	12117		18/08/2012 5:25	13309
20/05/2013 3:23	12787		21/10/2012 7:06	15955
23/08/2013 6:02	9218		13/04/2013 4:43	12300
27/09/2013 6:20	18211		17/08/2013 4:56	15088
9/10/2013 4:10	13040		28/09/2013 6:41	7367
12/12/2013 3:07	8058		13/10/2013 4:00	13913
15/01/2014 6:15	16175		18/01/2014 6:40	9427
19/02/2014 6:08	13940		8/02/2014 5:17	15614
Average	13027		Average	12077
Std Dev	2842		Std Dev	3700
3 x Std Dev	21551		3 x Std Dev	23176

Table J.4: SPS 15 Inflow Exceedance days indicates only 8 days were deemed not suitable, with two of the days being wet weather days resulting the SPS being inhibited i.e. the SPS was turned off as a result of overflows already occurring at SPS 3. Two other days that were not suitable had inflow periods of approximately 11.5hrs, these periods are well in excess of normal inflow times adopted as being suitable.

Table J.4: SPS 15 Inflow Exceedance Days

SPS 15					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
31/12/2011 9:40	41741	11.6	Saturday	0	Dry Weekend
4/01/2012 15:07	41430	11.5	Wednesday	2.2	Dry Weekday
26/03/2012 7:46	24247	6.7	Monday	0	Dry Weekday
27/05/2012 6:48	23566	6.5	Sunday	0	Dry Weekend
4/10/2012 7:44	24493	6.8	Thursday	0	Dry Weekday
22/05/2013 6:31	21543	6.0	Wednesday	0	Dry Weekday
25/03/2014 7:42	35683	9.9	Tuesday	158	Station Inhibited
27/03/2014 3:58	21696	6.0	Thursday	26.6	Station Inhibited

J.3.SPS 21

SPS 21 had the lowest maximum inflow time of all the SPSs; the average maximum inflow time is 1.3 hours for weekdays and 1.2 hours for weekends. The standard deviation of the results was approximately 15 minutes and the maximum adopted inflow time was approximately 2 hours for both weekdays and weekends.

Table J.5: SPS 21 Maximum Inflow Time indicates that all random days used had similar maximum inflow times occurring predominately between 12am and 4am. These periods are well below that of similar catchments i.e. SPS 3 and 15 and indicate that consistent night time flow is occurring. This needs to be further investigated as the catchment is elevated in comparison to the other catchments and it is not expected that the inflow is a result of ground water infiltration.

Further investigation also needs to be undertaken into the Endeavour energy light industrial facility, potential the night times results are reflective of 24 hours operations.

Table J.5: SPS 21 Maximum Inflow Time

SPS 21				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 2:16	4962		21/05/2011 4:59	4146
27/06/2011 3:47	3469		25/06/2011 5:18	3005
21/09/2011 4:35	5253		31/07/2011 2:43	2406
10/08/2012 4:02	3288		17/09/2011 5:12	4435
20/08/2012 3:33	6044		23/10/2011 6:01	4431
2/10/2012 2:52	6286		15/04/2012 3:22	3844
26/10/2012 4:30	6144		28/04/2012 3:01	2848
21/03/2013 3:23	4623		23/06/2012 4:47	3197
10/05/2013 3:07	4096		18/08/2012 4:02	6094
20/05/2013 2:34	4337		21/10/2012 5:44	5546
23/08/2013 3:42	5188		13/04/2013 5:20	4790
27/09/2013 2:54	3926		17/08/2013 6:08	4061
9/10/2013 2:16	4776		28/09/2013 3:19	3975
12/12/2013 4:00	3572		13/10/2013 2:21	3953
15/01/2014 3:10	4736		18/01/2014 5:10	4752
19/02/2014 3:43	4004		8/02/2014 3:41	5073
Average	4669		Average	4160
Std Dev	914		Std Dev	956
3 x Std Dev	7412		3 x Std Dev	7029

Table J.6: SPS 21 Inflow Exceedance days indicates that the majority of the rejected days were a result of the station being inhibited during wet weather events. Of the 4 days that were not a result of wet weather events the exceedance occurred outside of normal operating hours. This is indicative that there were operational issues with the SPS that required on call staff to attend.

Table J.6: SPS 21 Inflow Exceedance days

SPS 21					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
14/12/2011 19:27	7580	2.1	Wednesday	0	
4/01/2012 6:42	13317	3.7	Wednesday	2.2	
11/02/2012 6:15	13651	3.8	Saturday	43.4	Station Inhibited
17/03/2012 0:55	10581	2.9	Saturday	58.4	Station Inhibited
7/09/2012 20:36	9517	2.6	Friday	0	
7/10/2012 5:15	12635	3.5	Sunday	2.8	
29/01/2013 6:58	10411	2.9	Tuesday	77.2	Station Inhibited
24/02/2013 9:10	19475	5.4	Sunday	93.6	Station Inhibited
20/04/2013 14:54	24084	6.7	Saturday	74.2	Station Inhibited
21/04/2013 10:23	54963	15.3	Sunday	8.2	Station Inhibited
24/05/2013 21:47	7460	2.1	Friday	15.6	Station Inhibited
25/05/2013 5:51	14563	4.0	Saturday	9.6	Station Inhibited
24/06/2013 6:00	7751	2.2	Monday	45	Station Inhibited
25/03/2014 8:16	16691	4.6	Tuesday	158	Station Inhibited
26/03/2014 7:58	11399	3.2	Wednesday	48	Station Inhibited
27/03/2014 3:58	25951	7.2	Thursday	26.6	Station Inhibited
8/04/2014 15:56	10636	3.0	Tuesday	5.2	Station Inhibited

J.4.SPS 23

The maximum inflow analysis for SPS 23 resulted in an allowable inflow time of 71643 seconds for weekdays and 107983 for weekends. This is approximately 19.9 hours and 30 hours for weekdays and weekends respectively. This was to be expected as the SPS services an industrial estate and has a single sporadic large discharger and minimal work occurs on weekends.

A single day (20/5/2013) from the 16 days for the random data was rejected on the basis that the maximum inflow period from the previous day had been exceeded and was greater than 24 hours. In addition a single day from the weekend random data was also rejected on the basis the SCADA data indicated the pump had run continuously for 36 hours.

Table J.7: SPS 23 Maximum Inflow Time shows the average inflow time for weekdays is approximately 7.5 hours whilst for weekends its 10.4 hours. The maximum adopted inflow times are nearly 3 times these values indicating the high degree of flow variability that occurs at this station.

Table J.7: SPS 23 Maximum Inflow Time

SPS 23				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 3:59	19654		21/05/2011 3:21	12844
27/06/2011 2:37	25627		25/06/2011 21:14	15606
21/09/2011 2:26	9844		31/07/2011 10:09	6531
10/05/2012 7:34	52452		17/09/2011 13:36	7851
10/08/2012 8:36	37504		23/10/2011 7:37	16989
20/08/2012 0:09	61120		15/04/2012 2:51	65847
2/10/2012 2:13	20820		28/04/2012 23:16	40728
26/10/2012 23:15	11695		23/06/2012 11:32	51667
21/03/2013 7:50	34861		18/08/2012 11:42	72843
20/05/2013 7:20	Rejected		21/10/2012 16:44	66334
23/08/2013 7:46	28779		13/04/2013 0:00	Rejected
27/09/2013 22:43	16099		17/08/2013 22:07	21376
9/10/2013 22:51	12713		28/09/2013 18:24	32809
12/12/2013 5:40	10984		13/10/2013 21:23	74671
15/01/2014 8:52	27094		18/01/2014 1:37	26240
19/02/2014 9:01	37659		8/02/2014 15:58	47226
Average	27127		Average	37304
Std Dev	14839		Std Dev	23560
3 x Std Dev	71643		3 x Std Dev	107983

From an operational standpoint this knowledge is extremely useful not only for maintenance work but also for identifying a SPS whereby septicity may be an issue.

An extensive number of days were required to be rejected from both SPS 23 and 3 data sets. Table J.8: SPS 23 Inflow Exceedance days has a total of 27 days rejected on the basis on exceeding maximum inflow time and a further 8 days as a result of missing SCADA data. The majority of the rejected days are Mondays; this highlights the importance of undertaking the analysis as the SPS flow data for Monday would have biased the final results due to not pumping effluent from the previous day. The rejected data also included seven occurrences of 2 day combinations, 6 of these events included Mondays.

This analysis also highlighted that minimal discharge occurs from the light industrial precinct on Sundays, thus the majority of flows can be attributed to I/I during a wet weather event that occurs on a Sunday.

Table J.8: SPS 23 Inflow Exceedance Days

SPS 23					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
30/06/2013 0:00	719070	199.7	Sunday	3	SCADA Failure
1/07/2013 0:00			Monday	0.4	
2/07/2013 0:00			Tuesday	0	
3/07/2013 0:00			Wednesday	0	
4/07/2013 0:00			Thursday	0	
5/07/2013 0:00			Friday	0	
6/07/2013 0:00			Saturday	0	
7/07/2013 0:00			Sunday	0	
8/07/2013 20:16			Monday	0	
19/05/2013 0:00	122847	34.1	Sunday	0	Multiple days
			Monday	0	
19/10/2013 0:00	109711	30.5	Saturday	0	Multiple days
			Sunday	0	
5/05/2013 0:00	109686	30.5	Sunday	0	Multiple days
			Monday	0	
27/07/2013 0:00	103544	29	Saturday	0	Multiple days
			Sunday	0	
7/10/2013 0:00	93773	26.0	Monday	0	Multiple days
			Tuesday	0	
9/03/2013 0:00	91758	25.5	Saturday	0.2	Multiple days
			Sunday	0	
13/05/2013 8:03	86364	24.0	Monday	0	Overlap Sunday
6/01/2013 0:00	86316	24	Sunday	0	Multiple days
			Monday	0	
7/05/2012 7:09	83332	23.1	Monday	0	Overlap Sunday
17/06/2013 7:29	82407	22.9	Monday	0	Overlap Sunday
24/12/2012 10:10	80503	22.4	Monday	1	Overlap Sunday
9/12/2013 0:00	80351	22	Sunday	0	Multiple days
			Monday	0.4	
4/11/2013 7:46	78733	21.9	Monday	0	Overlap Sunday
19/12/2011 9:11	76094	21.1	Monday	3.6	Overlap Sunday
16/09/2012 0:00	75178	21	Sunday	0	Multiple days
			Monday	0	
28/05/2012 7:22	74904	20.8	Monday	0	Overlap Sunday
2/07/2012 6:38	74740	20.8	Monday	0	Overlap Sunday
24/02/2014 6:08	73454	20.4	Monday	1	Overlap Sunday

J.5.SPS 26

The maximum inflow was calculated to be approximately 17 hours for weekdays and 32 hours for weekends. Table J.9: SPS 26 Maximum Inflow Time highlights the high degree of variability of the maximum inflow times. The random days selected for the analysis have inflow times ranging from 1.5 hours to in excess of 1 day.

This was expected as the university campus typical operating hours are 8am to 5pm weekdays and the facility is relatively minor. In addition to this as a result of holiday periods the campus has minimal attendance for extended periods of time.

Table J.9: SPS 26 Maximum Inflow Time

SPS 26				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 4:22	15720		21/05/2011 7:52	53498
27/06/2011 2:11	14700		25/06/2011 5:09	22591
21/09/2011 8:48	31680		31/07/2011 17:05	4182
10/08/2012 7:19	26340		17/09/2011 10:49	57392
20/08/2012 6:34	23640		23/10/2011 4:30	49928
2/10/2012 18:32	9709		15/04/2012 13:40	7695
26/10/2012 23:39	13907		28/04/2012 21:28	6369
21/03/2013 4:39	16740		23/06/2012 22:19	11154
10/05/2013 3:49	32173		18/08/2012 11:44	40035
20/05/2013 22:53	21574		21/10/2012 11:38	98807
23/08/2013 3:04	23726		13/04/2013 14:28	66664
27/09/2013 6:01	35423		17/08/2013 14:14	4386
9/10/2013 9:54	6368		28/09/2013 9:48	33830
12/12/2013 15:57	5399		13/10/2013 17:21	2858
15/01/2014 5:39	47622		18/01/2014 0:21	4503
19/02/2014 7:55	46854		8/02/2014 14:08	1548
Average	23223		Average	29090
Std Dev	12523		Std Dev	28356
3 x Std Dev	60793		3 x Std Dev	114159

This catchment also reflected similar results to that of SPS 23 whereby the wastewater discharge during weekends was minimal and thus any wet weather event that occurred on a weekend the majority of the flows are likely to be a result of I/I.

Table J.10: SPS 26 Inflow exceedance days shows that for weekends minimal wastewater discharge can be expected to occur. All of the rejected days were period of 2 to 3 days with the exception of the 29th March 2013 which was Easter Friday. This day is a public holiday and thus it is expected that no discharge from the campus would occur on this day.

Table J.10: SPS 26 Inflow Exceedance Days

SPS 26					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
22/05/2011 0:00	81462	22.6	Sunday	0	2 day period
			Monday	0	
21/01/2012 0:00	88759	24.7	Saturday	0.4	2 day period
			Sunday	1	
15/09/2012 0:00	108864	30.2	Saturday	0	2 day period
			Sunday	0	
22/09/2012 0:00	94568	26.3	Saturday	0	2 events 3 day period
			Sunday	0	
	74758	20.8	Monday	0	
20/10/2012 0:00	98807	27.4	Saturday	0	2 events 3 day period
			Sunday	0	
	70886	19.7	Monday	0	
24/11/2012 0:00	108025	30.0	Saturday	0	2 day period
			Sunday	0	
29/03/2013 18:25	62241	17.3	Friday	0	Good Friday
10/01/2014 0:00	82554	22.9	Saturday	0	2 day period
			Saturday	0	

J.6.SPS 29

SPS 29 services the South Correctional Facility and is a unique facility in comparison to other land uses in the Shoalhaven Region. The results shown in Table J.11: SPS 29 Maximum Inflow Time are similar to that of a residential area. The maximum inflow time for weekdays was 4.9 hours whilst for weekends it was 5.4 hours. The calculated standard deviation was approximately 1hr for weekdays demonstrating that the wastewater discharge from the site between 12am and 6am was consistent throughout the year.

Table J.11: SPS 29 Maximum Inflow Time

SPS 29				
Weekday			Weekend	
Date / Time Start	Time (s)		Date / Time Start	Time (s)
23/05/2011 5:27	16872		21/05/2011 6:24	15021
27/06/2011 5:17	10987		25/06/2011 4:45	9103
21/09/2011 6:40	12364		31/07/2011 6:09	9886
10/05/2012 4:06	11247		17/09/2011 6:15	15001
10/08/2012 5:18	9623		23/10/2011 6:58	14765
20/08/2012 6:12	8124		15/04/2012 5:51	7290
2/10/2012 4:51	8693		28/04/2012 5:09	6488
26/10/2012 4:57	8582		23/06/2012 6:10	7717
21/03/2013 5:34	8126		18/08/2012 6:37	7661
20/05/2013 4:36	5783		21/10/2012 7:40	7491
23/08/2013 4:50	7306		13/04/2013 5:51	7390
27/09/2013 4:39	10879		17/08/2013 4:30	6255
9/10/2013 4:49	10054		28/09/2013 4:13	4080
12/12/2013 4:00	5348		13/10/2013 4:25	5798
15/01/2014 5:19	9978		18/01/2014 6:47	9454
19/02/2014 3:43	5731		8/02/2014 3:55	1561
Average	9356		Average	8435
Std Dev	2798		Std Dev	3677
3 x Std Dev	17751		3 x Std Dev	19465

Only 6 days were rejected based on the analysis and 2 of these days was a result of the station operation being inhibited during wet weather events. Table J.12: SPS 29 Inflow Exceedance days shows only 4 days which exceeded the maximum adopted inflow time that was not a result of operational restriction i.e. station inhibited. In addition the maximum inflow time that was deemed not suitable was only 6.9 hours, indicating that inflow to the station is relatively consistent for all 7 days of the week. This is expected due to the nature of the facility, the residents are full time occupants.

Table J.12: SPS 29 Inflow Exceedance Days

SPS 29					
Date / Time Start	Time (s)	Hr's	Day	Rainfall (mm)	Comment
24/05/2011 7:06	19637	5.5	Tuesday	0	Weekday
26/05/2011 5:41	21655	6.0	Thursday	0	Weekday
28/05/2011 6:27	19384	5.4	Saturday	74.2	Station Inhibited
5/07/2011 22:02	24822	6.9	Tuesday	0	Weekday
19/11/2012 0:52	19091	5.3	Monday	0	Weekday
20/04/2013 14:00	20837	5.8	Saturday	2.2	Station Inhibited

Appendix K: Dry Day Flow

K.1. SPS 15

SPS 15 had 150 weekdays and 68 weekend days over the entire analysis period that were deemed suitable based on the criteria for analysis. The results indicate that there was a decrease in the ADWF over the 3 periods for both weekdays and weekend. Period 2 had the highest peaking factor for both weekdays and weekends whilst the minimum flow was the same for period 2 and 3 weekdays and period 3 weekends. The results are provided in Table K.1: SPS 15 Dry Day Flow. Based on the entire analysis period the peaking factor was the same for weekdays as weekends.

Table K.1: SPS 15 Dry Day Flow

SPS 15								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.50	0.55	0.46	0.52	0.41	0.43	0.45	0.49
Std Dev (L/s)	0.05	0.19	0.12	0.18	0.05	0.08	0.09	0.16
3 x Std Dev (L/s)	0.66	1.12	0.83	1.04	0.57	0.67	0.73	0.95
PDWF (L/s)	1.37	1.61	2.08	2.26	1.37	1.15	1.64	1.63
Std Dev (L/s)	0.66	1.44	2.06	2.32	1.11	0.51	1.52	1.63
3 x Std Dev (L/s)	3.35	5.93	8.25	9.23	4.69	2.69	6.19	6.52
Peaking Factor	2.76	2.94	4.51	4.39	3.32	2.66	3.63	3.35
Minimum Flow (L/s)	0.15	0.16	0.12	0.15	0.12	0.12	0.13	0.14
Std Dev (L/s)	0.04	0.07	0.05	0.09	0.03	0.04	0.04	0.07
3 x Std Dev (L/s)	0.26	0.37	0.26	0.43	0.22	0.23	0.25	0.35
No. of Dry Days	39	16	58	23	53	29	150	68
No. of Wet Days	219	87	201	81	205	75	625	243

The limited number of days did not provide a suitable range of data to determine the peak month of each period. Using the 3 times standard deviation parameter for ADWF, PDWF and minimum flow the data set was re-evaluated. The results are provided in Table K.2: SPS 15 Dry Day Flow Expanded Data Set

Table K.2: SPS 15 Dry Day Flow Expanded Data Set

Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.48	0.51	0.50	0.54	0.41	0.42	0.45	0.48
Std Dev (L/s)	0.05	0.09	0.14	0.17	0.04	0.05	0.07	0.10
PDWF (L/s)	1.28	1.43	2.39	2.49	1.20	1.08	1.33	1.44
Std Dev (L/s)	0.43	0.72	2.24	2.15	0.66	0.28	0.74	0.87
Peaking Factor	2.68	2.82	4.78	4.61	2.91	2.59	2.97	3.02
Minimum Flow (L/s)	0.14	0.16	0.13	0.14	0.13	0.12	0.13	0.14
Std Dev (L/s)	0.04	0.06	0.03	0.07	0.03	0.03	0.04	0.05
No. of Dry Days	205	92	219	102	223	86	625	274
No. of Wet Days	53	11	40	2	36	17	150	37

(Table heading: SPS 15)

The expanded data set has 625 weekdays and 274 weekend days suitable for evaluation. The results are similar to the previous data set. There is a noticeable decrease in the PDWF, for all data, of 0.3L/s for weekdays which reduced the peaking factor from 3.63 to 2.97. The same effect also occurred on weekends with a reduction of PDWF of 0.2L/s this also decreased the peaking factor from 3.35 to 3.02.The minimum flow values had only minor variations. This data was adopted as being suitable to determine the peak month of each period. The results are provided in Table K.3: SPS 15 Monthly ADWF.

Table K.3: SPS 15 Monthly ADWF

Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	0.49	0.46	0.43	0.47	0.50	0.48	0.45	0.49
May	0.50	0.44	0.42	0.46	0.52	0.43	0.40	0.47
June	0.45	0.46	0.40	0.45	0.50	0.52	0.42	0.49
July	0.49	0.42	0.43	0.44	0.51	0.41	0.40	0.44
August	0.50	0.42	0.41	0.44	0.55	0.43	0.43	0.46
September	0.53	0.45	0.41	0.47	0.60	0.44	0.41	0.48
October	0.48	0.40	0.44	0.44	0.50	0.42	0.45	0.46
November	0.45	0.60	0.42	0.43	0.47	0.57	0.46	0.50
December	0.45	0.77	0.40	0.44	0.46	0.82	0.39	0.49
January	0.42	0.71	0.42	0.43	0.44	0.76	0.40	0.45
February	0.48	0.63	0.39	0.45	0.53	0.76	0.42	0.51
March	0.50	0.46	0.38	0.45	0.59	0.47	0.39	0.48
Annual	0.48	0.50	0.41	0.45	0.51	0.54	0.42	0.48

(Table heading: SPS 15 (L/s))

No month was the peak month for all 3 periods. November to March in period 2 exhibited an increase in flows which was against the overall trend of a decrease in flows over the 3 year period. September and April had the highest flow based on the entire analysis period, in comparison to SPS 3 whereby September had the lowest flows.

The WSAA method was then used as a check on the monthly calculated values (Table K.3) and to establish a typical diurnal curve based on the entire data period. The results are provided in Table K.4: SPS 15 WSAA Dry Day Flow

Table K.4: SPS 15 WSAA Dry Day Flow

SPS 15								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	0.48	0.47	1.01	1.05	2.10	2.23	0.15	0.14
May, 2011	0.50	0.52	0.96	1.12	1.93	2.18	0.16	0.16
Jun, 2011	0.45	0.50	0.99	1.04	2.19	2.09	0.13	0.18
Jul, 2011	0.48	0.47	1.00	1.00	2.09	2.12	0.14	0.12
Aug, 2011	0.49	0.55	0.92	1.17	1.88	2.15	0.13	0.19
Sep, 2011	0.52	0.56	1.02	1.13	1.95	2.01	0.17	0.18
Oct, 2011	0.47	0.50	0.95	1.16	2.01	2.33	0.13	0.13
Nov, 2011	0.43	0.42	1.05	1.03	2.45	2.47	0.12	0.12
Dec, 2011	0.42	0.43	0.90	0.86	2.12	1.98	0.12	0.14
Jan, 2012	0.41	0.42	0.67	0.89	1.63	2.10	0.11	0.11
Feb, 2012	0.45	0.48	0.79	1.08	1.78	2.26	0.16	0.15
Mar, 2012	0.51	0.52	0.88	0.96	1.75	1.84	0.19	0.21
Apr, 2012	0.46	0.43	0.89	1.05	1.96	2.42	0.14	0.12
May, 2012	0.42	0.42	0.81	0.91	1.90	2.16	0.11	0.10
Jun, 2012	0.43	0.47	0.86	1.00	1.99	2.14	0.14	0.14
Jul, 2012	0.40	0.41	0.79	0.89	1.97	2.15	0.12	0.11
Aug, 2012	0.40	0.43	0.88	0.98	2.17	2.28	0.11	0.12
Sep, 2012	0.40	0.43	0.86	1.09	2.16	2.52	0.11	0.10
Oct, 2012	0.38	0.41	0.76	0.87	2.01	2.11	0.10	0.11
Nov, 2012	0.38	0.41	0.80	0.88	2.08	2.14	0.10	0.11
Dec, 2012								
Jan, 2013								
Feb, 2013	0.50	0.49	0.99	1.01	1.95	2.07	0.17	0.14
Mar, 2013	0.45	0.46	0.95	0.98	2.13	2.14	0.13	0.13
Apr, 2013	0.41	0.45	0.74	0.90	1.81	2.01	0.13	0.14
May, 2013	0.39	0.40	0.79	0.89	2.05	2.21	0.12	0.13
Jun, 2013	0.38	0.42	0.75	0.89	1.95	2.14	0.11	0.10
Jul, 2013	0.42	0.41	0.73	0.86	1.77	2.13	0.13	0.11
Aug, 2013	0.41	0.41	0.83	0.97	2.02	2.36	0.12	0.13
Sep, 2013	0.40	0.41	0.78	0.90	1.97	2.21	0.13	0.12
Oct, 2013	0.44	0.46	0.82	0.89	1.87	1.96	0.15	0.14
Nov, 2013	0.40	0.42	0.89	1.04	2.22	2.48	0.12	0.11
Dec, 2013	0.39	0.37	0.79	0.70	2.02	1.89	0.13	0.10
Jan, 2014	0.40	0.39	0.74	0.82	1.85	2.07	0.12	0.11
Feb, 2014	0.38	0.41	0.74	0.83	1.94	2.04	0.13	0.13
Mar, 2014	0.38	0.39	0.69	0.88	1.82	2.23	0.12	0.12
Average	0.43	0.44	0.85	0.96	1.99	2.17	0.13	0.13
Std Dev	0.04	0.05	0.10	0.11	0.16	0.16	0.02	0.03

The results using the WSAA method were similar to the statistical method.

Figure K.1: SPS 15 Diurnal Curve is the flow characteristic for period 3. The weekend peak occurred later in the morning and was higher than the weekday peak. The minimum flow for both weekdays and weekends was the same.

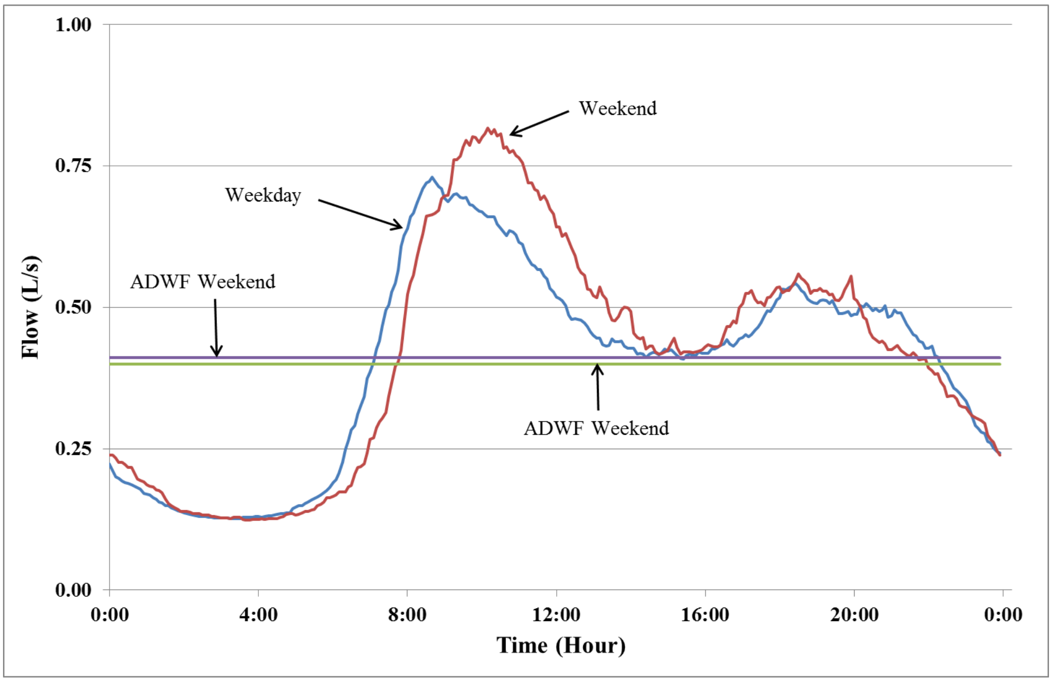


Figure K.1: SPS 15 Diurnal Curve

The flow variability for SPS 15 shown in Figure K.2: SPS 15 Diurnal Flow Variability. It indicates that the maximum flow is approximately double the PDWF, there is also a high degree of flow variability in the early hours of the day i.e. 1am to 3am, and this may be a result of this catchment having upstream SPSs discharging into it.

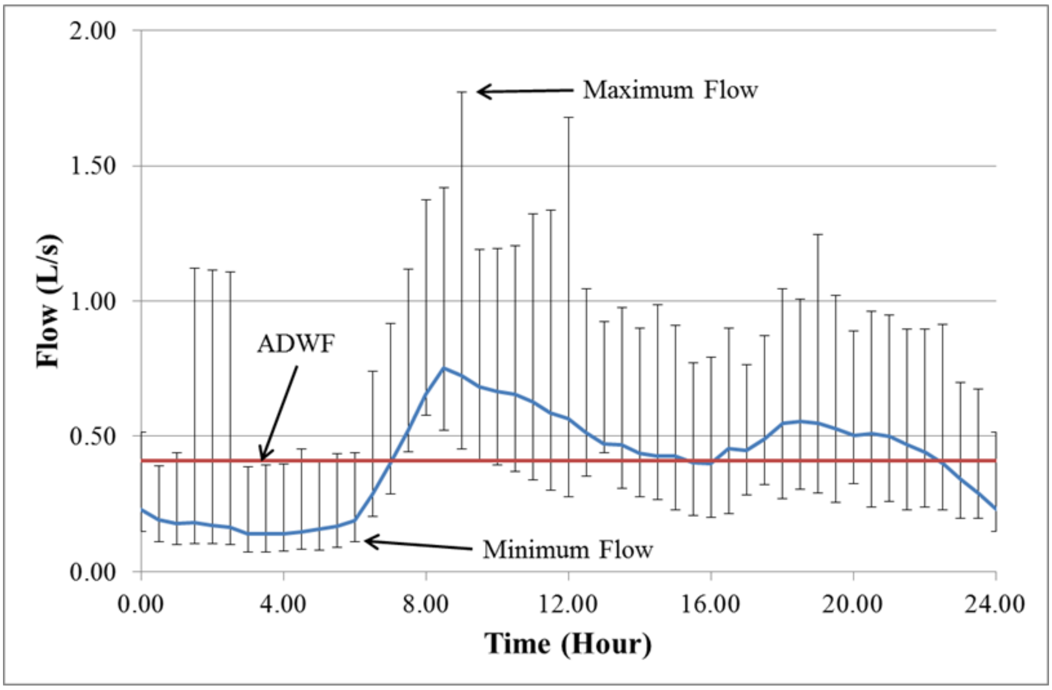


Figure K.2: SPS 15 Diurnal Flow Variability

K.2. SPS 21

SPS 21 had 150 weekdays and 68 weekend days over the entire analysis period that were deemed suitable based on the criteria for analysis. The results indicate that there was an increase in the ADWF over the 3 periods for both weekdays and weekend. Period 2 had the highest peaking factor for both weekdays and weekends whilst the minimum flow was higher for weekends compared to weekends. Based on the entire analysis period the peaking factor was the same for weekdays as weekends. The results are provided in Table K.5: SPS 21 Dry Day Flow.

Table K.5: SPS 21 Dry Day Flow

SPS 21								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	1.12	1.25	1.18	1.29	1.28	1.39	1.20	1.32
Std Dev (L/s)	0.13	0.14	0.39	0.40	0.43	0.23	0.36	0.29
3 x Std Dev (L/s)	1.52	1.67	2.35	2.48	2.56	2.09	2.29	2.20
PDWF (L/s)	3.64	3.67	4.65	5.15	3.22	3.99	3.88	4.32
Std Dev (L/s)	1.92	0.76	3.73	3.87	1.24	1.47	2.70	2.55
3 x Std Dev (L/s)	9.39	5.93	15.83	16.77	6.95	8.39	11.97	11.96
Peaking Factor	3.26	2.94	3.94	4.00	2.51	2.86	3.23	3.27
Minimum Flow (L/s)	0.31	0.36	0.28	0.29	0.29	0.34	0.29	0.33
Std Dev (L/s)	0.09	0.09	0.08	0.08	0.05	0.10	0.08	0.10
3 x Std Dev (L/s)	0.59	0.64	0.51	0.53	0.44	0.63	0.52	0.62
No. of Dry Days	39	17	58	23	53	29	150	69
No. of Wet Days	219	85	199	80	203	73	621	243

The limited number of days did not provide a suitable range of data to determine the peak month of each period. Using the 3 times standard deviation parameter for ADWF, PDWF and minimum flow the data set was re-evaluated. The results are provided in Table K.6: SPS 21 Dry Day Flow Expanded Data Set.

Table K.6: SPS 21 Dry Day Flow Expanded Data Set

SPS 21								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	1.13	1.20	1.29	1.35	1.28	1.38	1.18	1.27
Std Dev (L/s)	0.11	0.12	0.43	0.41	0.15	0.17	0.21	0.23
PDWF (L/s)	3.26	3.51	5.71	5.85	3.23	3.80	3.42	3.75
Std Dev (L/s)	1.25	1.03	4.43	4.25	0.61	0.84	1.64	1.49
Peaking Factor	2.89	2.92	4.41	4.33	2.53	2.76	2.89	2.95
Minimum Flow (L/s)	0.33	0.36	0.29	0.30	0.31	0.35	0.31	0.34
Std Dev (L/s)	0.08	0.08	0.07	0.09	0.07	0.10	0.08	0.10
No. of Dry Days	196	83	219	95	181	90	566	254
No. of Wet Days	62	19	38	8	75	12	55	53

The expanded data set showed a jump in ADWF from period 1 to 2, whilst the flow was similar for periods 2 and 3. The PDWF was higher in period 2, whilst the minimum flow remained similar across all 3 periods. This data was adopted as being suitable to determine the peak month of each period.

December was the peak month for all three weekday periods and for 2 of the 3 weekend periods. It was near the peak weekend month for period 1 with only 0.03L/s difference. The results are provided in Table K.7: SPS 21 Monthly ADWF.

Table K.7: SPS 21 Monthly ADWF

SPS 21 (L/s)								
Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	1.11	1.19	1.27	1.22	1.10	1.29	1.54	1.29
May	1.08	1.01	1.24	1.08	1.24	1.06	1.34	1.22
June	1.24	1.14	1.24	1.20	1.24	1.19	1.43	1.34
July	1.08	1.04	1.33	1.16	1.16	1.11	1.43	1.24
August	1.16	0.99	1.21	1.11	1.24	1.05	1.29	1.20
September	1.04	0.98	1.22	1.08	1.13	1.05	1.32	1.16
October	1.14	0.97	1.24	1.11	1.22	1.01	1.27	1.21
November	1.07	1.55	1.34	1.25	1.15	1.50	1.40	1.29
December	1.28	1.94	1.53	1.45	1.31	1.93	1.60	1.45
January	1.15	1.84	1.26	1.24	1.16	1.88	1.29	1.26
February	1.10	1.73	1.20	1.19	1.34	1.79	1.24	1.35
March	1.13	1.21	1.30	1.28	1.26	1.34	1.41	1.36
Annual	1.13	1.29	1.28	1.18	1.20	1.35	1.38	1.27

The WSAA method was used as a check on the above monthly calculated values and to establish a typical diurnal curve based on the entire data period.

The results are provided in Table K.8: SPS21 WSAA Dry Day Flow. The results indicate that the WSAA method resulted in slightly lower ADWF for both weekdays and weekends based on the entire analysis period.

Table K.8: SPS 21 WSAA Dry Day Flow

SPS 21								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	1.06	1.07	2.13	2.49	2.00	2.33	0.31	0.32
May, 2011	1.07	1.14	2.62	2.98	2.44	2.61	0.28	0.29
Jun, 2011	1.21	1.22	2.46	2.87	2.04	2.35	0.43	0.42
Jul, 2011	1.05	1.11	2.07	2.84	1.97	2.56	0.28	0.30
Aug, 2011	1.13	1.20	2.63	2.93	2.32	2.44	0.35	0.38
Sep, 2011	1.04	1.06	2.20	2.36	2.12	2.22	0.28	0.27
Oct, 2011	1.11	1.26	2.82	2.84	2.54	2.26	0.33	0.37
Nov, 2011	1.02	1.12	2.30	2.45	2.25	2.19	0.30	0.32
Dec, 2011	1.25	1.27	2.52	2.80	2.02	2.20	0.37	0.39
Jan, 2012	1.13	1.10	2.25	2.49	1.99	2.27	0.34	0.33
Feb, 2012	1.03	1.34	2.23	3.28	2.17	2.44	0.33	0.39
Mar, 2012	1.12	1.26	2.23	3.14	2.00	2.50	0.41	0.49
Apr, 2012	1.09	1.12	2.16	2.56	1.98	2.29	0.38	0.38
May, 2012	0.99	1.00	2.19	2.41	2.21	2.40	0.29	0.28
Jun, 2012	1.03	1.08	2.41	2.37	2.34	2.19	0.35	0.33
Jul, 2012	1.03	1.10	2.07	2.82	2.00	2.56	0.30	0.31
Aug, 2012	0.96	1.01	2.41	2.65	2.51	2.61	0.23	0.24
Sep, 2012	0.97	1.04	2.36	2.60	2.43	2.51	0.23	0.22
Oct, 2012	0.92	1.04	2.21	2.31	2.40	2.23	0.23	0.25
Nov, 2012	0.96	1.07	2.53	2.58	2.64	2.42	0.21	0.22
Dec, 2012								
Jan, 2013								
Feb, 2013	1.13	1.20	2.56	2.99	2.27	2.49	0.30	0.35
Mar, 2013	1.16	1.11	2.56	2.50	2.20	2.26	0.33	0.30
Apr, 2013	1.15	1.32	2.39	3.40	2.08	2.58	0.30	0.38
May, 2013	1.21	1.31	2.78	3.06	2.29	2.33	0.30	0.35
Jun, 2013	1.24	1.37	2.89	3.52	2.32	2.57	0.37	0.42
Jul, 2013	1.36	1.37	2.64	3.36	1.95	2.46	0.43	0.39
Aug, 2013	1.20	1.30	2.79	3.31	2.32	2.55	0.29	0.36
Sep, 2013	1.16	1.21	2.73	3.08	2.36	2.54	0.25	0.21
Oct, 2013	1.23	1.25	2.63	3.04	2.13	2.43	0.29	0.29
Nov, 2013	1.36	1.37	3.01	3.71	2.21	2.71	0.39	0.34
Dec, 2013	1.48	1.45	3.14	3.03	2.12	2.09	0.41	0.45
Jan, 2014	1.25	1.32	2.20	2.77	1.76	2.10	0.33	0.36
Feb, 2014	1.18	1.21	2.56	2.92	2.17	2.42	0.27	0.30
Mar, 2014	1.26	1.33	3.02	3.32	2.39	2.50	0.28	0.30
Average	1.13	1.20	2.49	2.88	2.20	2.40	0.32	0.33
Std Dev	0.13	0.12	0.29	0.36	0.20	0.16	0.06	0.07

Figure K.3: SPS 21 Diurnal Curve is the flow characteristic for period 3. The weekend peak occurred later in the morning and was higher than the weekday peak. The weekend afternoon peak was lower for weekends. The minimum flow for both weekdays and weekends was similar.

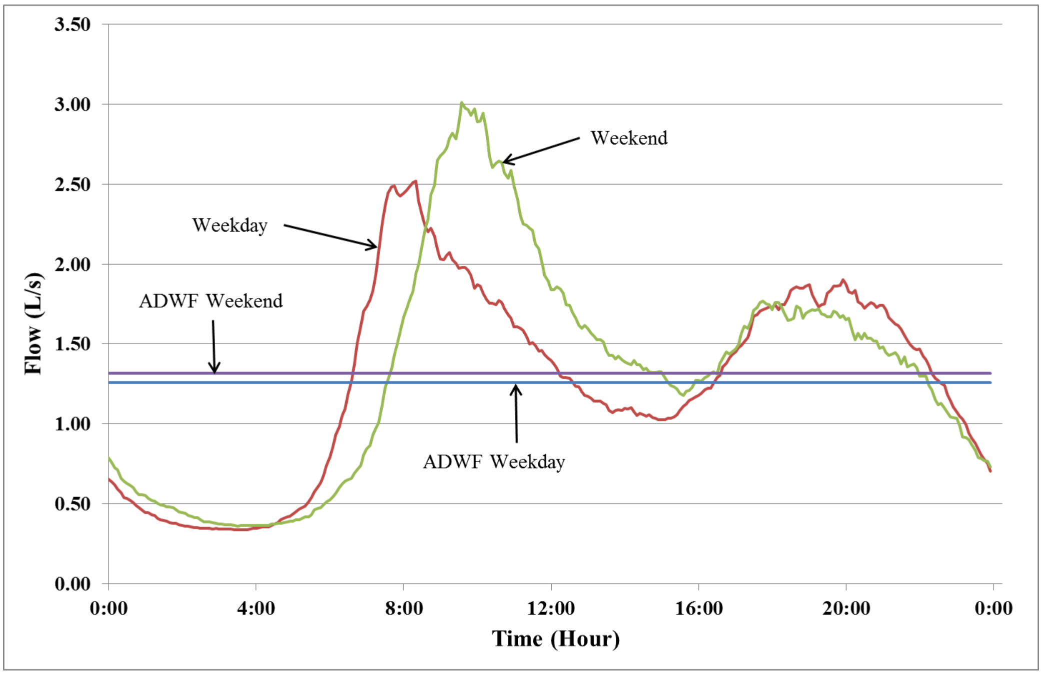


Figure K.3: SPS 21 Diurnal Curve

The flow variability for SPS 21 shown in Figure K.4: SPS 21 Diurnal Flow Variability. It indicates that the maximum flow is more than double the PDWF. There is also a high degree of flow variability occurring during the afternoon peak period.

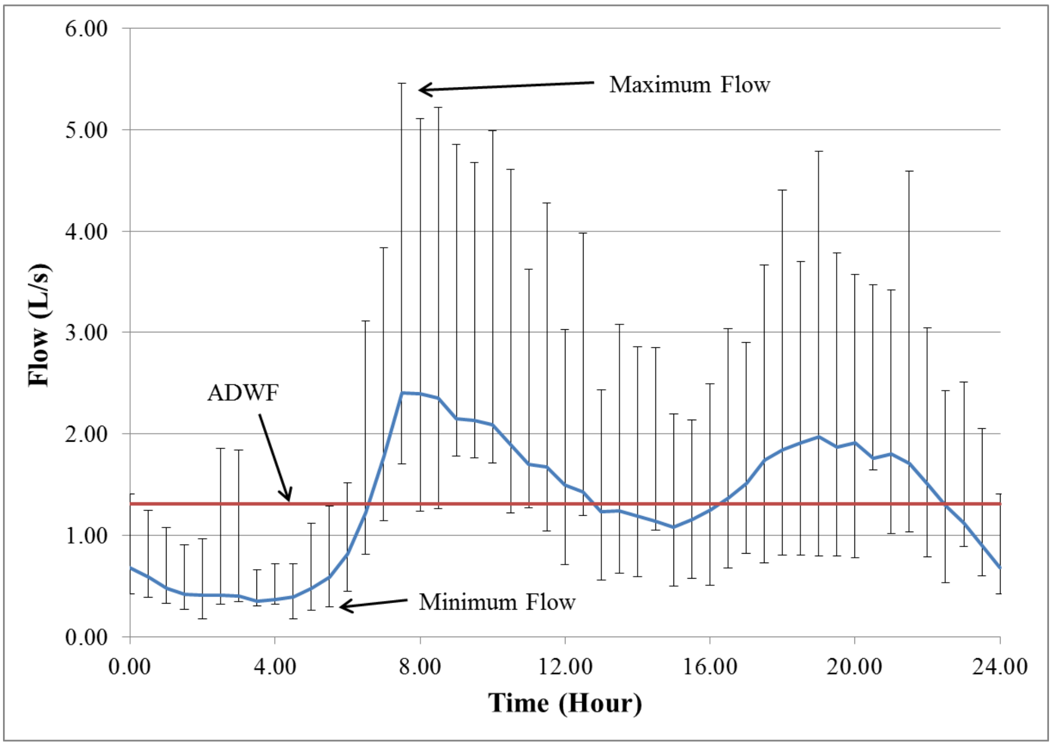


Figure K.4: SPS 21 Diurnal Flow Variability

K.3. SPS 23

SPS 23 had 143 weekdays and 61 weekend days over the entire analysis period that were deemed suitable based on the criteria for analysis. The results indicate that there was a large decrease in the ADWF from period 1 to 2 and then a minor increase in period 3. The PDWF and peaking factor was higher for weekdays compared to weekends whilst the minimum flow was higher on weekends. The results are provided in Table K.9: SPS 23 Dry Day Flow.

Table K.9: SPS 23 Dry Day Flow

SPS 23								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.71	0.55	0.40	0.14	0.49	0.43	0.51	0.36
Std Dev (L/s)	0.47	1.13	0.21	0.10	0.25	1.20	0.34	0.95
3 x Std Dev (L/s)	2.11	3.94	1.04	0.44	1.23	4.02	1.52	3.21
PDWF (L/s)	5.98	3.04	3.83	1.19	3.18	2.54	4.20	2.19
Std Dev (L/s)	3.73	4.69	5.09	0.66	3.96	5.63	4.53	4.83
3 x Std Dev (L/s)	17.19	17.10	19.09	3.18	15.07	19.44	17.78	16.68
Peaking Factor	8.43	5.51	9.57	8.67	6.54	5.85	8.18	6.09
Minimum Flow (L/s)	0.28	0.37	0.13	0.11	0.22	0.69	0.20	0.39
Std Dev (L/s)	0.57	0.85	0.07	0.07	0.11	2.64	0.31	1.67
3 x Std Dev (L/s)	1.98	2.92	0.34	0.32	0.56	8.62	1.14	5.40
No. of Dry Days	39	17	56	22	48	22	143	61
No. of Wet Days	220	85	198	79	197	73	615	239

The limited number of days did not provide a suitable range of data to determine the peak month of each period. Using the 3 times standard deviation parameter for ADWF, PDWF and minimum flow the data set was re-evaluated. The results are provided in Table K.10: SPS 23 Dry Day Flow Expanded Data Set.

Table K.10: SPS 23 Dry Day Flow Expanded Data Set

SPS 23								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.65	0.43	0.45	0.14	0.38	0.16	0.49	0.26
Std Dev (L/s)	0.36	0.51	0.21	0.09	0.22	0.19	0.24	0.35
PDWF (L/s)	5.33	2.14	4.86	0.40	2.02	0.97	3.50	1.11
Std Dev (L/s)	2.93	2.82	6.11	0.54	3.13	2.96	3.04	2.08
Peaking Factor	8.15	5.02	10.89	2.87	5.36	6.10	7.21	4.25
Minimum Flow (L/s)	0.20	0.26	0.14	0.10	0.08	0.12	0.18	0.18
Std Dev (L/s)	0.15	0.29	0.07	0.05	0.02	0.15	0.10	0.20
No. of Dry Days	173	82	217	84	300	46	581	261
No. of Wet Days	86	22	37	17	40	0	177	39

The results also showed a reduction of ADWF, PDWF, Peaking Factor and minimum flow over the three periods. The most significant difference was for the PDWF and subsequent calculated peaking factor for period 2 weekends.

No month was the peak month for all three periods. April was the peak weekday month, whilst February was the peak weekend month based on the 3 years. The results are provided in Table K.11: SPS 23 Monthly ADWF.

Table K.11: SPS 23 Monthly ADWF

SPS 23(L/s)								
Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	0.81	0.52	0.37	0.58	0.35	0.15	0.13	0.32
May	0.63	0.42	0.38	0.47	0.26	0.10	0.12	0.18
June	0.53	0.52	0.43	0.49	0.19	0.13	0.51	0.25
July	0.91	0.46	0.47	0.53	0.22	0.11	0.27	0.20
August	0.68	0.36	0.33	0.43	0.62	0.09	0.16	0.30
September	0.60	0.39	0.41	0.49	0.34	0.12	0.12	0.21
October	0.97	0.40	0.37	0.53	1.11	0.18	0.06	0.52
November	0.72	0.49	0.39	0.52	0.69	0.15	0.14	0.26
December	0.35	0.42	0.32	0.40	0.19	0.08	0.09	0.13
January	0.44	0.37	0.35	0.49	0.24	0.23	0.19	0.21
February	0.42	0.61	0.35	0.56	0.70	0.28	0.07	0.35
March	0.34	0.45	0.38	0.45	0.47	0.15	0.11	0.25
Annual	0.65	0.45	0.38	0.49	0.43	0.14	0.16	0.26

The WSAA method was used as a check on the above monthly calculated values and to establish a typical diurnal curve based on the entire data period. The results indicate there is a significant drop in peak flow during period 2 consistent with the results in Table K.10. This may be a result of changes to land use within the catchment. The results are provided in Table K.12 – SPS23 WSAA Flow Analysis

Table K.12: SPS 23 WSAA Dry Day Flow

SPS 23								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	0.68	0.20	2.25	0.39	3.30	1.95	0.21	0.13
May, 2011	0.45	0.18	1.38	0.23	3.08	1.28	0.19	0.12
Jun, 2011	0.49	0.13	1.63	0.19	3.35	1.50	0.16	0.08
Jul, 2011	0.42	0.16	1.63	0.21	3.88	1.29	0.10	0.12
Aug, 2011	0.30	0.14	2.51	0.19	8.29	1.31	0.12	0.12
Sep, 2011	0.52	0.21	1.93	0.35	3.69	1.64	0.16	0.14
Oct, 2011	0.71	0.59	5.45	4.03	7.72	6.77	0.24	0.34
Nov, 2011	0.39		1.24		3.16		0.09	
Dec, 2011	0.21	0.18	0.54	0.58	2.54	3.20	0.07	0.08
Jan, 2012	0.47	0.24	2.56	1.78	5.39	7.57	0.11	0.10
Feb, 2012	0.26	0.70	1.04	2.41	3.96	3.43	0.10	0.37
Mar, 2012	0.39	0.37	0.99	2.86	2.55	7.64	0.21	0.21
Apr, 2012	0.34	0.15	1.29	0.40	3.80	2.72	0.09	0.10
May, 2012	0.16	0.07	0.31	0.10	1.93	1.47	0.09	0.05
Jun, 2012	0.24	0.10	1.35	0.12	5.67	1.24	0.08	0.07
Jul, 2012	0.39	0.10	0.93	0.12	2.36	1.18	0.11	0.08
Aug, 2012	0.25	0.08	0.49	0.09	1.97	1.17	0.14	0.07
Sep, 2012	0.29	0.08	0.69	0.10	2.41	1.15	0.10	0.07
Oct, 2012	0.30	0.14	0.61	0.18	2.06	1.26	0.18	0.10
Nov, 2012	0.35	0.10	1.02	0.14	2.88	1.39	0.14	0.07
Dec, 2012	0.23	0.07	0.36	0.07	1.56	1.10	0.13	0.06
Jan, 2013	0.15	0.11	0.32	0.47	2.08	4.42	0.11	0.06
Feb, 2013	0.48	0.35	1.13	0.59	2.35	1.68	0.24	0.20
Mar, 2013	0.32	0.08	0.68	0.10	2.12	1.33	0.19	0.07
Apr, 2013	0.35	0.14	0.60	0.20	1.70	1.39	0.21	0.09
May, 2013	0.32	0.10	0.83	0.13	2.57	1.28	0.17	0.06
Jun, 2013	0.39	0.13	0.90	0.21	2.31	1.64	0.16	0.06
Jul, 2013	0.18	0.07	0.51	0.07	2.92	1.10	0.07	0.06
Aug, 2013	0.34	0.13	0.72	0.18	2.09	1.34	0.21	0.09
Sep, 2013	0.39	0.11	0.72	0.17	1.83	1.57	0.21	0.08
Oct, 2013	0.38	0.06	0.68	0.09	1.76	1.43	0.24	0.05
Nov, 2013	0.35	0.10	0.70	0.15	2.03	1.47	0.17	0.07
Dec, 2013	0.34	0.13	0.51	0.17	1.49	1.27	0.23	0.08
Jan, 2014	0.32	0.13	0.74	0.18	2.34	1.42	0.15	0.09
Feb, 2014	0.36	0.07	1.16	0.09	3.18	1.25	0.15	0.06
Mar, 2014	0.44	1.33	1.10	3.32	2.50	2.50	0.21	0.09
Average	0.36	0.20	1.15	0.59	3.02	2.15	0.15	0.11
Std Dev	0.12	0.24	0.92	0.99	1.53	1.74	0.05	0.07

Figure K.5: SPS 23 Diurnal Curve is the flow characteristic for period 3. The figure indicates that during weekdays there is a peak period occurring during the middle of the day i.e. lunch time and minimal activity within the catchment on weekends. Further analysis of the weekend period to differentiate between Saturdays and Sundays would highlight the extent of activity that occurs in the industrial estate on weekends. It is expected that higher flows would occur on the Saturday and no flows on the Sunday. This was evident in the maximum inflow analysis that was undertaken.

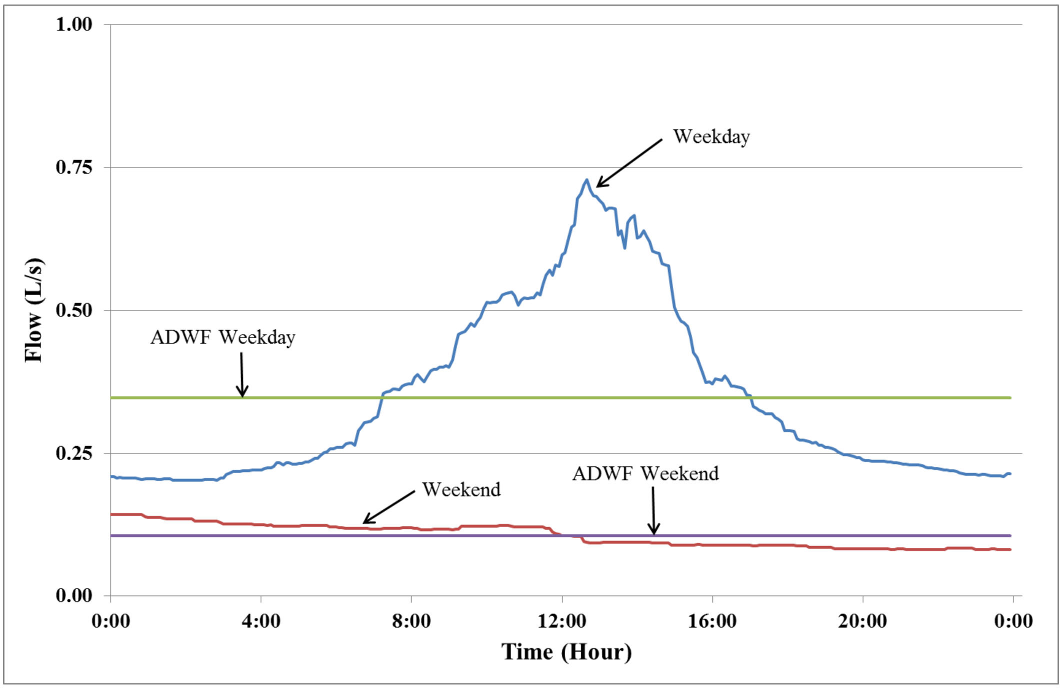


Figure K.5: SPS 23 Diurnal Curve

The results of the flow variability for SPS 23 have been discussed previously in the report (Figure 5.7). It is noted that the maximum flow was approximately 10 L/s and that the diurnal curve in Figure K.5 shows a peak of 0.75 L/s. It appears that this is representative of the catchment as a whole however, as previously noted, does not illustrate the irregular high peak discharge of the liquid treatment facility.

K.4. SPS 26

SPS 26 had 149 weekdays and 67 weekend days over the entire analysis period that were deemed suitable based on the criteria for analysis. The results indicate that there was a large increase in the ADWF and PDWF from period 1 to 2 and then a decrease in period 3 to flows similar to the average for the entire analysis period. The minimum flow increased from period 1 to 3 and was similar for both weekdays and weekends. The results are provided in Table K.13: SPS 26 Dry Day Flow.

Table K.13: SPS 26 Dry Day Flow

SPS 26								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.06	0.05	0.20	0.16	0.14	0.21	0.14	0.16
Std Dev (L/s)	0.03	0.05	0.30	0.32	0.08	0.15	0.20	0.22
3 x Std Dev (L/s)	0.15	0.20	1.10	1.11	0.38	0.66	0.74	0.81
PDWF (L/s)	0.41	0.09	1.72	0.90	0.46	0.37	0.93	0.50
Std Dev (L/s)	0.82	0.07	3.01	2.49	0.56	0.27	2.05	1.47
3 x Std Dev (L/s)	2.86	0.31	10.73	8.38	2.14	1.19	7.07	4.92
Peaking Factor	6.59	1.71	8.68	5.72	3.34	1.75	6.61	3.19
Minimum Flow (L/s)	0.03	0.04	0.08	0.07	0.13	0.15	0.09	0.08
Std Dev (L/s)	0.03	0.05	0.10	0.07	0.12	0.13	0.11	0.11
3 x Std Dev (L/s)	0.14	0.19	0.39	0.28	0.50	0.54	0.40	0.41
No. of Dry Days	38	16	58	22	53	29	149	67
No. of Wet Days	221	85	199	77	207	74	627	236

The limited number of days did not provide a suitable range of data to determine the peak month of each period. Using the 3 times standard deviation parameter for ADWF, PDWF and minimum flow the data set was re-evaluated. The results are provided in Table K.14: SPS 26 Dry Day Flow Expanded Data Set.

Table K.14: SPS 26 Dry Day Flow Expanded Data Set

SPS 26								
Value	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	0.07	0.06	0.20	0.14	0.15	0.20	0.14	0.14
Std Dev (L/s)	0.03	0.04	0.23	0.17	0.09	0.13	0.11	0.12
PDWF (L/s)	0.28	0.12	1.72	0.81	0.34	0.41	0.67	0.36
Std Dev (L/s)	0.33	0.08	2.75	1.85	0.31	0.25	1.05	0.52
Peaking Factor	3.81	2.09	8.51	5.96	2.19	2.01	4.74	2.67
Minimum Flow (L/s)	0.04	0.04	0.09	0.07	0.03	0.15	0.10	0.09
Std Dev (L/s)	0.03	0.04	0.09	0.06	0.03	0.12	0.10	0.09
No. of Dry Days	172	73	236	85	228	99	714	277
No. of Wet Days	87	28	21	14	32	4	62	26

This data was adopted as being suitable to determine the peak month of each period.

December and January in period 2 were the peak months; this is usually the holiday period for the university campus and may be a result of other courses being undertaken during this period. The results are provided in Table K.15: SPS 26 Monthly ADWF.

Table K.15: SPS 26 Monthly ADWF

SPS 26 (L/s)								
Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	0.06	0.17	0.09	0.13	0.04	0.13	0.10	0.11
May	0.07	0.13	0.11	0.11	0.04	0.09	0.10	0.08
June	0.07	0.20	0.13	0.15	0.04	0.16	0.19	0.14
July	0.05	0.11	0.16	0.12	0.07	0.12	0.18	0.13
August	0.10	0.07	0.08	0.09	0.07	0.06	0.11	0.08
September	0.06	0.07	0.13	0.09	0.03	0.02	0.14	0.09
October	0.08	0.07	0.17	0.12	0.07	0.06	0.32	0.16
November	0.08	0.20	0.27	0.20	0.04	0.04	0.28	0.14
December	0.08	0.68	0.22	0.17	0.09	0.25	0.19	0.15
January	0.06	0.62	0.13	0.15	0.03	0.74	0.22	0.15
February	0.09	0.26	0.17	0.18	0.06	0.46	0.28	0.16
March		0.13	0.23	0.23	0.12	0.10	0.33	0.23
Annual	0.07	0.20	0.15	0.14	0.06	0.14	0.20	0.14

The WSAA method was used as a check on the above monthly calculated values and to establish a typical diurnal curve based on the entire data period. The results were similar to those in Table K.15. The results are provided in Table K.16: SPS26 WSAA Dry Day Flow.

Table K.16: SPS 26 WSAA Dry Day Flow

SPS 26								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	0.05	0.07	0.10	0.11	1.87	1.61	0.02	0.04
May, 2011	0.05	0.01	0.13	0.02	2.59	1.70	0.02	0.01
Jul, 2011	0.06	0.04	0.13	0.06	2.25	1.47	0.02	0.03
Jul, 2011	0.02	0.02	0.04	0.03	1.62	1.57	0.01	0.01
Aug, 2011	0.09	0.04	0.21	0.06	2.20	1.56	0.05	0.02
Sep, 2011	0.05	0.02	0.12	0.06	2.28	2.47	0.02	0.02
Oct, 2011	0.07	0.06	0.14	0.12	1.96	1.97	0.03	0.03
Nov, 2011	0.07	0.02	0.13	0.04	1.82	1.83	0.03	0.01
Dec, 2011	0.06	0.05	0.09	0.09	1.46	1.67	0.04	0.02
Jan, 2012	0.05	0.01	0.07	0.01	1.55	1.21	0.03	0.01
Feb, 2012	0.10	0.02	0.16	0.04	1.56	1.63	0.06	0.02
Mar, 2012	0.19	0.12	0.32	0.15	1.65	1.23	0.13	0.06
Apr, 2012	0.14	0.11	0.19	0.15	1.37	1.31	0.10	0.08
May, 2012	0.12	0.07	0.26	0.10	2.25	1.40	0.06	0.05
Jun, 2012	0.10	0.11	0.18	0.16	1.85	1.43	0.06	0.07
Jul, 2012	0.10	0.06	0.15	0.09	1.49	1.45	0.07	0.04
Aug, 2012	0.07	0.02	0.17	0.02	2.50	1.11	0.02	0.01
Sep, 2012	0.05	0.01	0.13	0.01	2.73	1.16	0.02	0.01
Oct, 2012	0.04	0.01	0.11	0.02	2.57	1.61	0.01	0.01
Nov, 2012	0.04	0.01	0.09	0.02	2.55	1.74	0.01	0.01
Dec, 2012		0.02		0.11		4.63		0.01
Jan, 2013								
Feb, 2013	0.05		0.17		3.63		0.02	
Mar, 2013	0.06	0.02	0.14	0.03	2.38	1.32	0.02	0.02
Apr, 2013	0.05	0.02	0.10	0.05	1.97	1.95	0.02	0.02
May, 2013	0.06	0.05	0.15	0.11	2.34	2.23	0.02	0.03
Jun, 2013	0.07	0.04	0.19	0.11	2.67	2.76	0.03	0.02
Jul, 2013	0.08	0.10	0.17	0.14	2.19	1.43	0.03	0.05
Aug, 2013	0.05	0.02	0.11	0.03	1.97	1.69	0.02	0.01
Sep, 2013	0.06	0.06	0.17	0.16	2.72	2.99	0.02	0.02
Oct, 2013	0.04	0.15	0.13	0.26	3.46	1.79	0.02	0.06
Nov, 2013	0.25	0.21	0.39	0.34	1.53	1.60	0.13	0.11
Dec, 2013	0.22	0.10	0.27	0.14	1.26	1.40	0.16	0.06
Jan, 2014	0.05	0.01	0.09	0.02	1.93	1.95	0.02	0.01
Feb, 2014	0.16	0.02	0.30	0.03	1.88	1.78	0.06	0.01
Mar, 2014	0.09	0.19	0.18	0.32	2.06	1.63	0.03	0.30
Average	0.08	0.06	0.16	0.09	2.12	1.77	0.04	0.04
Std Dev	0.05	0.05	0.07	0.08	0.54	0.65	0.04	0.05

Figure K.6 - SPS 26 Diurnal Curve is the flow characteristic for period 3. The weekend flow was minimal and the minimum flow on weekends occurs in the middle of the day. Due to the level of low flows it is difficult to make a conclusion based on the weekend diurnal flow. The weekday peak occurred during lunch time which was expected.

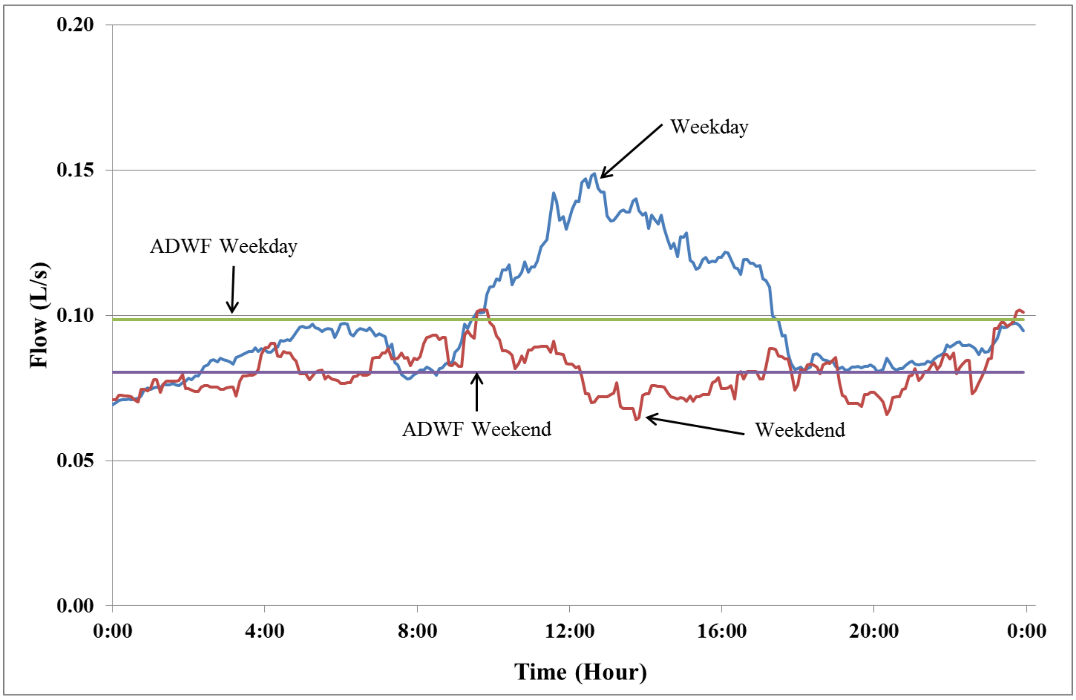


Figure K.6: SPS 26 Diurnal Flow

The flow variability for SPS 26 shown in Figure K.7: SPS 26 Diurnal Flow Variability. It indicates that the maximum flow is up to 6 times ADWF and follows a consistent flow profile to the ADWF. This may be a result of short courses being undertaken at the university.

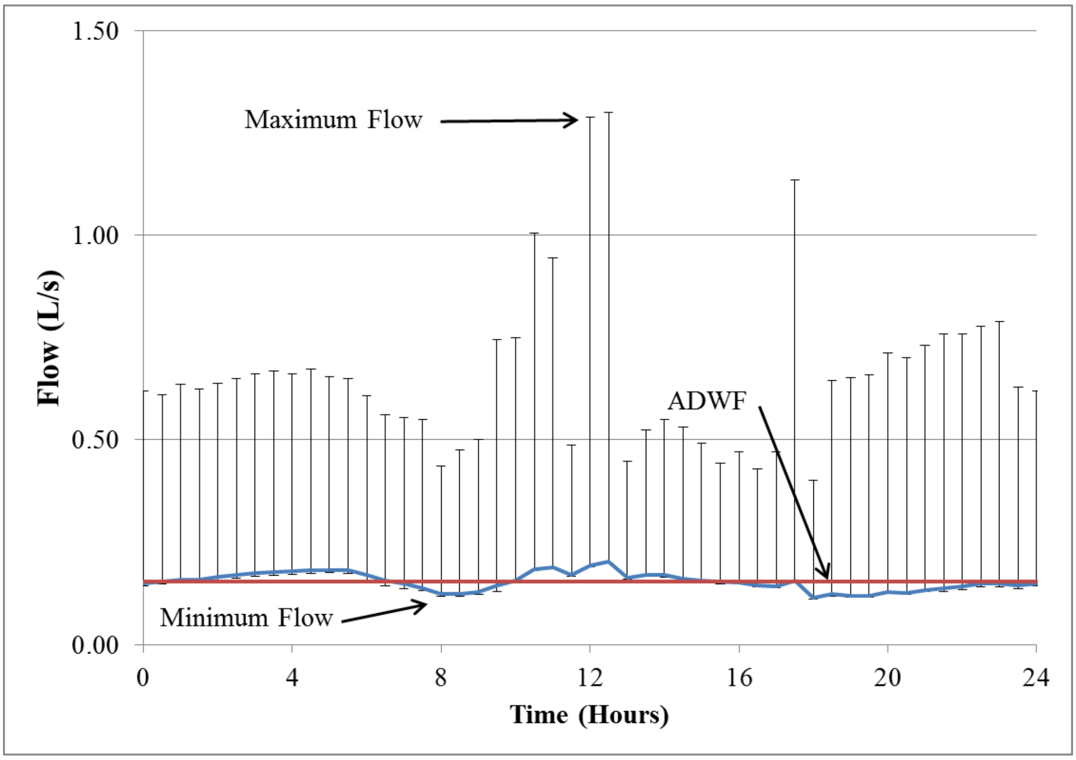


Figure K.7: SPS 26 Diurnal Flow Variability

K.5. SPS 29

SPS 29 had 151 weekdays and 69 weekend days over the entire analysis period that were deemed suitable based on the criteria for analysis. The results indicate that there was a large increase in the ADWF and PDWF from period 1 to 3. The minimum flow and PDWF also increased over the analysis period. The peaking factor remained similar for all periods. There was minimal difference between weekdays and weekends which would be expected for a jail. The results are provided in Table K.17: SPS 29 Dry Day Flow.

Table K.17: SPS 29 Dry Day Flow

Value	SPS 29							
	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	1.24	1.18	2.00	1.96	2.63	2.68	2.02	1.95
Std Dev (L/s)	0.20	0.20	0.42	0.44	0.29	0.77	0.63	0.83
3 x Std Dev (L/s)	1.85	1.79	3.27	3.28	3.51	4.98	3.92	4.43
PDWF (L/s)	3.64	2.87	4.82	4.38	5.31	5.36	4.69	4.32
Std Dev (L/s)	1.32	0.75	1.39	1.29	0.72	0.70	1.35	1.38
3 x Std Dev (L/s)	7.60	5.12	8.98	8.25	7.48	7.47	8.73	8.44
Peaking Factor	2.94	2.43	2.41	2.23	2.02	2.00	2.32	2.21
Minimum Flow (L/s)	0.31	0.29	0.51	0.51	0.69	0.85	0.52	0.53
Std Dev (L/s)	0.26	0.07	0.17	0.20	0.45	0.54	0.35	0.43
3 x Std Dev (L/s)	1.10	0.50	1.03	1.11	2.04	2.47	1.57	1.83
No. of Dry Days	39	17	59	23	53	29	151	69
No. of Wet Days	218	86	200	82	208	74	626	242

The results are provided in Table K.18: SPS 29 Dry Day Flow Expanded Data Set. The results indicate that the flow increased over the three periods for both weekdays and weekends. This flow increase was also consistent with the overall increase in flows that occurred at SPS 3.

Table K.18: SPS 29 Dry Day Flow Expanded Data Set Analysis

Value	SPS 29							
	Period 1		Period 2		Period 3		All Data	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
ADWF (L/s)	1.39	1.24	2.14	2.10	2.62	2.55	2.10	2.03
Std Dev (L/s)	0.24	0.21	0.45	0.45	0.32	0.38	0.62	0.61
PDWF (L/s)	3.59	2.89	5.00	4.62	5.50	5.22	4.77	4.34
Std Dev (L/s)	1.13	0.70	1.33	1.25	0.84	0.69	1.38	1.29
Peaking Factor	2.57	2.33	2.34	2.20	2.10	2.05	2.26	2.14
Minimum Flow (L/s)	0.39	0.33	0.55	0.54	0.73	0.75	0.60	0.59
Std Dev (L/s)	0.13	0.09	0.19	0.20	0.28	0.31	0.31	0.28
No. of Dry Days	220	69	233	102	244	100	751	303
No. of Wet Days	37	34	26	3	17	3	26	8

This data was adopted as being suitable to determine the peak month of each period.

There was no consistent peak month, December to March was the peak period based on all periods however this trend was not evident for the weekends. The results are provided in Table K.19: SPS 29 Monthly ADWF.

Table K.19: SPS 29 Monthly ADWF

SPS 29 (L/s)								
Month	Weekday				Weekend			
	Period 1	Period 2	Period 3	All Periods	Period 1	Period 2	Period 3	All Periods
April	1.03	1.74	2.30	1.74	1.43	2.72	2.58	2.25
May	1.14	1.71	2.42	1.78		2.61	2.77	2.40
June	1.18	1.95	2.49	1.95	1.63	2.27	2.69	2.36
July	1.25	1.86	2.63	2.03	0.97	1.66	2.39	1.56
August	1.35	1.94	2.57	1.94	0.98	1.59	2.19	1.56
September	1.42	1.96	2.55	1.98	1.13	1.88	2.64	2.00
October	1.48	1.84	2.58	2.03	1.13	1.97	2.36	1.82
November	1.58	2.44	2.86	2.26	1.21	1.85	2.37	1.86
December	1.51	2.79	2.70	2.36	1.35	1.87	2.53	1.93
January	1.51	2.83	2.68	2.40	1.42	1.86	2.45	1.90
February	1.70	2.67	2.93	2.47	1.46	2.13	3.05	2.36
March	1.74	2.35	2.82	2.38	1.41	2.87	2.49	2.28
Annual	1.39	2.14	2.62	2.10	1.24	2.10	2.55	2.03

The WSAA method was used as a check on the above monthly calculated values and to establish a typical diurnal curve based on the entire data period. The results proved to be similar to those calculated using the statistical method. The results are provided in Table K.20: SPS29 WSAA Dry Day Flow.

Table K.20: SPS 29 WSAA Dry Day Flow

SPS 29								
Month / Year	ADWF (L/s)		Peak Flow (L/s)		Peaking Factor		Minimum Flow (L/s)	
	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend	Weekday	Weekend
Apr, 2011	1.01	1.05	2.08	2.07	2.06	1.96	0.29	0.36
May, 2011	1.02	0.98	2.59	2.14	2.53	2.19	0.23	0.24
Jul, 2011	1.08	1.11	2.49	2.32	2.31	2.08	0.27	0.36
Jul, 2011	1.16	1.08	2.69	2.28	2.31	2.10	0.29	0.29
Aug, 2011	1.35	1.15	3.02	2.74	2.24	2.39	0.37	0.32
Sep, 2011	1.38	1.21	2.98	2.76	2.16	2.27	0.33	0.29
Oct, 2011	1.49	1.37	3.01	3.18	2.01	2.32	0.42	0.33
Nov, 2011	1.52	1.51	3.46	3.36	2.27	2.22	0.45	0.46
Dec, 2011	1.46	1.38	3.23	2.94	2.22	2.12	0.40	0.38
Jan, 2012	1.49	1.41	3.36	3.00	2.26	2.13	0.40	0.32
Feb, 2012	1.73	1.70	3.40	3.17	1.96	1.87	0.55	0.61
Mar, 2012	1.80	1.68	3.46	3.79	1.92	2.26	0.62	0.54
Apr, 2012	1.65	1.59	3.38	3.12	2.05	1.96	0.43	0.43
May, 2012	1.59	1.53	3.28	3.33	2.06	2.18	0.40	0.39
Jun, 2012	1.75	1.69	3.54	3.48	2.02	2.06	0.50	0.47
Jul, 2012	1.74	1.84	3.39	3.79	1.95	2.07	0.47	0.55
Aug, 2012	1.82	1.70	3.64	3.47	2.00	2.04	0.46	0.47
Sep, 2012	1.82	1.80	3.69	3.80	2.02	2.11	0.58	0.51
Oct, 2012	1.77	1.80	3.31	3.91	1.87	2.17	0.45	0.43
Nov, 2012	1.84	1.84	4.30	4.17	2.34	2.26	0.41	0.53
Dec, 2012								
Jan, 2013								
Feb, 2013		2.09		4.15		1.99		0.42
Mar, 2013	2.23	2.12	4.55	4.27	2.04	2.02	0.62	0.66
Apr, 2013	2.47	2.15	4.40	4.00	1.78	1.86	0.75	0.44
May, 2013	2.24	1.97	4.43	4.34	1.98	2.20	0.64	0.45
Jun, 2013	2.26	2.13	4.06	4.43	1.79	2.08	0.63	0.64
Jul, 2013	2.65	2.23	4.77	4.52	1.80	2.03	0.94	0.66
Aug, 2013	2.47	2.36	4.67	4.97	1.89	2.11	0.71	0.71
Sep, 2013	2.41	2.40	4.25	4.80	1.76	2.00	0.61	0.72
Oct, 2013	2.46	2.46	4.49	5.00	1.82	2.03	0.60	0.69
Nov, 2013	2.57	2.66	4.58	4.93	1.78	1.85	0.73	0.66
Dec, 2013	2.50	2.41	4.48	4.75	1.79	1.97	0.61	0.82
Jan, 2014	2.64	2.48	4.85	4.76	1.84	1.92	0.69	0.58
Feb, 2014	2.67	2.77	5.07	5.30	1.90	1.91	0.71	0.86
Mar, 2014	2.05	2.29	3.79	4.77	1.85	2.08	0.49	0.79
Average	1.88	1.82	3.72	3.76	2.02	2.08	0.52	0.51
Std Dev	0.50	0.49	0.75	0.90	0.20	0.13	0.16	0.16

Figure K.8: SPS 29 Diurnal Curve is the flow characteristic for period 3. The weekend and weekday flows are similar and this is to be expected as the residents do not get out much.

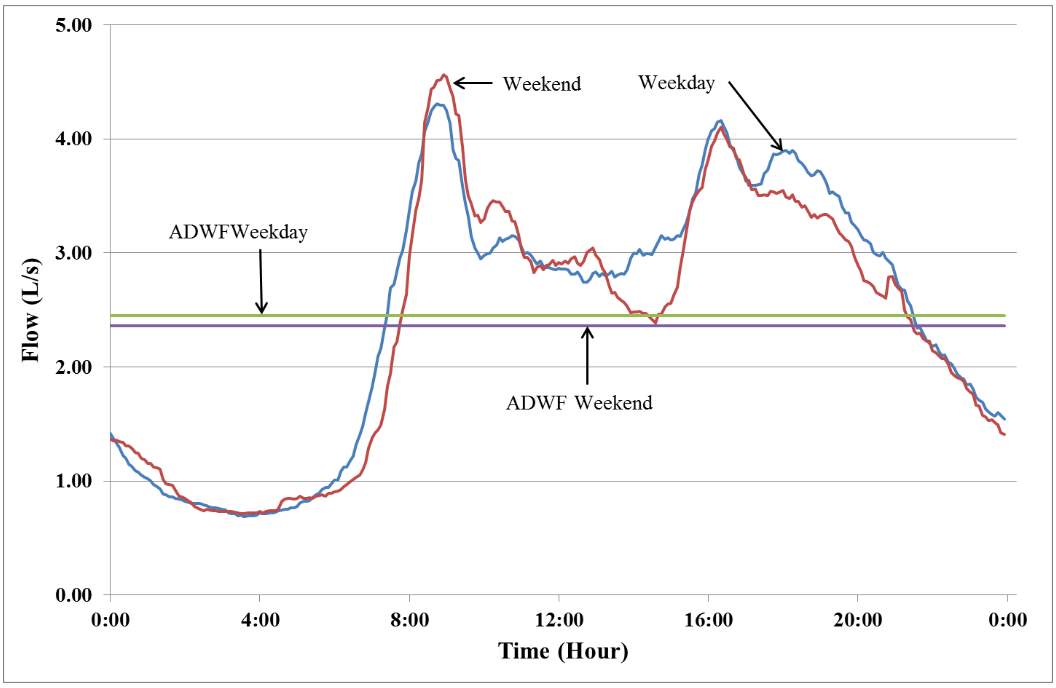


Figure K.8: SPS 29 Diurnal Curve

The flow variability for SPS 29 shown in Figure K.9: SPS 29 Diurnal Flow Variability. It indicates that the maximum flow is approximately 1.75 times the PDWF, there is also a high degree of flow variability in the early hours of the day i.e. 12am to 6am, and this may be a result of the occupants being full time residents.

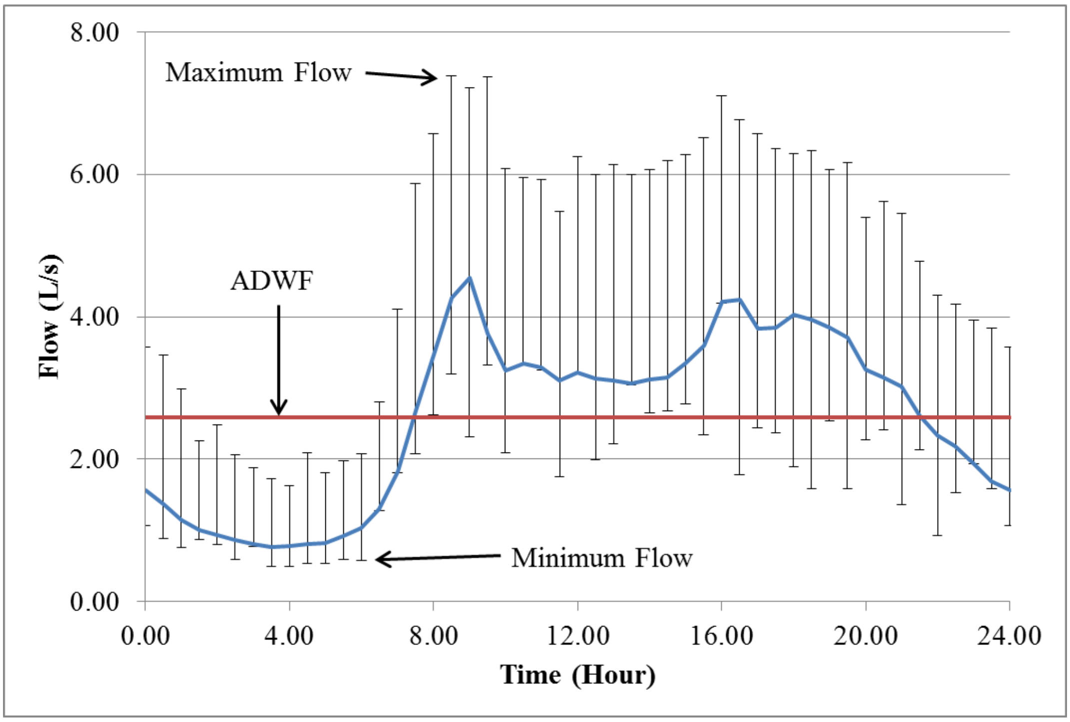


Figure K.9: SPS 29 Diurnal Flow Variability

Appendix L: WSAA Sensitivity Analysis

Table L.1: SPS 3 Diurnal Flow Values

Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor	Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor
0:00	6.84	2.14	8.98	1.31	4:00	2.77	2.37	5.14	1.85
0:05	6.62	2.13	8.74	1.32	4:05	2.78	2.37	5.15	1.85
0:10	6.41	1.87	8.28	1.29	4:10	2.78	2.37	5.15	1.85
0:15	6.17	1.79	7.96	1.29	4:15	2.77	2.37	5.14	1.86
0:20	6.35	3.84	10.19	1.60	4:20	2.78	2.36	5.15	1.85
0:25	5.86	1.80	7.66	1.31	4:25	2.82	2.36	5.18	1.84
0:30	5.60	1.48	7.08	1.26	4:30	2.83	2.36	5.19	1.83
0:35	5.39	1.33	6.73	1.25	4:35	2.83	2.36	5.19	1.83
0:40	5.25	1.29	6.54	1.25	4:40	2.84	2.36	5.20	1.83
0:45	5.17	1.29	6.46	1.25	4:45	2.86	2.36	5.22	1.82
0:50	4.99	1.27	6.26	1.25	4:50	2.89	2.36	5.24	1.82
0:55	4.87	1.23	6.10	1.25	4:55	2.91	2.35	5.26	1.81
1:00	4.73	1.18	5.91	1.25	5:00	2.92	2.36	5.28	1.81
1:05	4.62	1.10	5.72	1.24	5:05	2.96	2.36	5.32	1.79
1:10	4.55	1.09	5.64	1.24	5:10	3.04	2.38	5.41	1.78
1:15	4.44	1.09	5.53	1.25	5:15	3.10	2.38	5.48	1.77
1:20	4.41	1.10	5.51	1.25	5:20	3.12	2.38	5.50	1.76
1:25	4.36	1.09	5.45	1.25	5:25	3.21	2.38	5.59	1.74
1:30	4.21	0.97	5.18	1.23	5:30	3.09	0.86	3.95	1.28
1:35	4.28	2.53	6.81	1.59	5:35	3.14	0.88	4.03	1.28
1:40	4.19	2.55	6.74	1.61	5:40	3.24	0.91	4.15	1.28
1:45	4.06	2.55	6.61	1.63	5:45	3.42	1.09	4.51	1.32
1:50	3.91	2.55	6.45	1.65	5:50	3.53	1.09	4.62	1.31
1:55	3.78	2.55	6.33	1.67	5:55	3.65	1.18	4.83	1.32
2:00	3.74	2.55	6.29	1.68	6:00	3.83	1.29	5.12	1.34
2:05	3.62	2.55	6.16	1.70	6:05	3.98	1.31	5.30	1.33
2:10	3.75	3.49	7.24	1.93	6:10	4.16	1.47	5.62	1.35
2:15	3.66	3.49	7.15	1.96	6:15	4.21	1.45	5.66	1.34
2:20	3.58	3.49	7.07	1.97	6:20	4.49	1.57	6.06	1.35
2:25	3.53	3.50	7.02	1.99	6:25	4.60	1.60	6.20	1.35
2:30	3.50	3.50	7.00	2.00	6:30	4.75	1.67	6.42	1.35
2:35	3.47	3.51	6.97	2.01	6:35	5.07	1.80	6.86	1.35
2:40	3.44	3.51	6.95	2.02	6:40	5.40	2.11	7.51	1.39
2:45	3.38	3.50	6.88	2.04	6:45	5.75	2.15	7.90	1.37
2:50	3.34	3.51	6.85	2.05	6:50	6.11	2.42	8.53	1.40
2:55	3.28	3.51	6.79	2.07	6:55	6.46	2.69	9.15	1.42
3:00	3.23	3.51	6.74	2.09	7:00	7.14	2.83	9.97	1.40
3:05	3.02	2.55	5.56	1.84	7:05	7.77	2.98	10.75	1.38
3:10	3.00	2.55	5.55	1.85	7:10	8.24	3.17	11.40	1.38
3:15	2.98	2.55	5.53	1.86	7:15	8.72	3.22	11.95	1.37
3:20	2.92	2.55	5.47	1.87	7:20	9.31	3.26	12.57	1.35
3:25	2.89	2.54	5.43	1.88	7:25	9.91	3.52	13.43	1.36
3:30	2.86	2.55	5.41	1.89	7:30	10.65	3.55	14.20	1.33
3:35	2.86	2.55	5.41	1.89	7:35	11.16	3.64	14.80	1.33
3:40	2.64	0.66	3.30	1.25	7:40	11.86	3.98	15.83	1.34
3:45	2.65	0.69	3.33	1.26	7:45	12.62	4.20	16.81	1.33
3:50	2.62	0.68	3.29	1.26	7:50	13.03	4.28	17.32	1.33
3:55	2.80	2.37	5.16	1.85	7:55	13.69	4.31	17.99	1.31

Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor
8:00	14.03	4.45	18.48	1.32
8:05	14.39	4.53	18.92	1.31
8:10	14.71	4.54	19.25	1.31
8:15	15.56	5.13	20.69	1.33
8:20	16.32	5.06	21.37	1.31
8:25	16.88	4.83	21.71	1.29
8:30	17.68	5.26	22.94	1.30
8:35	18.29	5.23	23.51	1.29
8:40	19.11	5.37	24.48	1.28
8:45	19.52	5.43	24.95	1.28
8:50	19.79	5.36	25.15	1.27
8:55	19.90	5.16	25.06	1.26
9:00	19.98	5.04	25.02	1.25
9:05	19.84	4.97	24.81	1.25
9:10	19.87	4.64	24.50	1.23
9:15	19.64	4.50	24.14	1.23
9:20	19.41	4.42	23.83	1.23
9:25	19.60	4.21	23.82	1.21
9:30	19.57	4.26	23.83	1.22
9:35	20.04	4.33	24.37	1.22
9:40	20.33	4.42	24.75	1.22
9:45	20.82	4.17	24.99	1.20
9:50	20.76	4.31	25.07	1.21
9:55	20.87	4.12	24.99	1.20
10:00	20.57	4.14	24.71	1.20
10:05	20.21	4.31	24.52	1.21
10:10	19.95	4.14	24.09	1.21
10:15	19.49	4.12	23.60	1.21
10:20	19.23	3.93	23.16	1.20
10:25	19.01	3.67	22.68	1.19
10:30	18.84	3.83	22.67	1.20
10:35	18.63	3.92	22.55	1.21
10:40	18.36	3.91	22.28	1.21
10:45	17.93	4.03	21.96	1.22
10:50	17.76	4.67	22.43	1.26
10:55	18.06	6.51	24.57	1.36
11:00	17.68	5.75	23.42	1.33
11:05	17.19	4.49	21.69	1.26
11:10	17.10	4.47	21.57	1.26
11:15	16.94	4.33	21.26	1.26
11:20	16.90	4.02	20.92	1.24
11:25	16.81	3.97	20.79	1.24
11:30	16.61	3.86	20.46	1.23
11:35	16.73	3.96	20.69	1.24
11:40	16.54	3.52	20.06	1.21
11:45	16.68	3.53	20.21	1.21
11:50	16.74	3.49	20.23	1.21
11:55	16.76	3.56	20.32	1.21

Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor
12:00	16.54	3.37	19.91	1.20
12:05	16.24	3.27	19.51	1.20
12:10	16.21	3.39	19.60	1.21
12:15	16.10	3.22	19.33	1.20
12:20	15.92	3.29	19.21	1.21
12:25	16.13	3.41	19.54	1.21
12:30	15.72	3.45	19.17	1.22
12:35	15.83	3.63	19.46	1.23
12:40	15.54	3.69	19.23	1.24
12:45	15.45	3.59	19.04	1.23
12:50	15.18	3.56	18.74	1.23
12:55	15.17	3.59	18.75	1.24
13:00	15.00	3.61	18.61	1.24
13:05	14.84	3.59	18.44	1.24
13:10	14.98	3.68	18.66	1.25
13:15	14.90	3.70	18.60	1.25
13:20	14.86	3.69	18.55	1.25
13:25	14.81	3.68	18.50	1.25
13:30	14.57	3.56	18.13	1.24
13:35	14.83	3.66	18.49	1.25
13:40	14.32	3.40	17.71	1.24
13:45	14.49	4.10	18.59	1.28
13:50	14.55	4.21	18.76	1.29
13:55	14.72	5.75	20.47	1.39
14:00	14.60	4.52	19.12	1.31
14:05	14.46	4.69	19.14	1.32
14:10	14.61	4.89	19.50	1.33
14:15	14.56	4.86	19.42	1.33
14:20	14.36	3.89	18.25	1.27
14:25	14.51	3.95	18.45	1.27
14:30	14.37	3.98	18.35	1.28
14:35	14.33	3.95	18.28	1.28
14:40	14.38	3.83	18.21	1.27
14:45	14.18	3.75	17.93	1.26
14:50	14.45	4.89	19.34	1.34
14:55	14.16	3.47	17.63	1.24
15:00	14.05	3.52	17.58	1.25
15:05	13.98	3.87	17.85	1.28
15:10	14.04	3.88	17.92	1.28
15:15	14.06	3.88	17.94	1.28
15:20	13.84	3.69	17.53	1.27
15:25	14.15	4.00	18.15	1.28
15:30	14.08	3.97	18.05	1.28
15:35	14.23	3.99	18.21	1.28
15:40	14.39	4.07	18.46	1.28
15:45	14.74	4.54	19.28	1.31
15:50	14.66	4.14	18.80	1.28
15:55	14.46	4.20	18.66	1.29

Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor	Time (Hr / Min)	Flow (L/s)	1 Std Dev (L/s)	Peak Flow (L/s)	Peaking Factor
16:00	14.08	3.58	17.65	1.25	20:00	16.37	3.22	19.58	1.20
16:05	14.26	3.66	17.93	1.26	20:05	16.41	3.11	19.52	1.19
16:10	14.16	3.50	17.66	1.25	20:10	16.40	2.99	19.39	1.18
16:15	14.02	3.51	17.53	1.25	20:15	16.17	3.07	19.24	1.19
16:20	14.23	3.51	17.74	1.25	20:20	15.79	2.98	18.77	1.19
16:25	14.15	3.35	17.50	1.24	20:25	15.94	3.21	19.15	1.20
16:30	14.52	3.51	18.03	1.24	20:30	15.81	3.22	19.03	1.20
16:35	14.99	3.86	18.85	1.26	20:35	15.57	3.23	18.80	1.21
16:40	14.91	3.74	18.65	1.25	20:40	15.28	3.24	18.52	1.21
16:45	14.86	3.71	18.57	1.25	20:45	15.38	3.47	18.85	1.23
16:50	15.24	3.75	18.99	1.25	20:50	15.22	3.37	18.59	1.22
16:55	15.30	3.65	18.96	1.24	20:55	15.17	3.45	18.63	1.23
17:00	15.54	3.78	19.33	1.24	21:00	15.00	3.32	18.32	1.22
17:05	15.66	4.02	19.68	1.26	21:05	14.83	3.29	18.12	1.22
17:10	15.96	4.13	20.09	1.26	21:10	14.86	3.41	18.26	1.23
17:15	16.13	4.03	20.16	1.25	21:15	14.57	3.30	17.87	1.23
17:20	16.18	4.15	20.33	1.26	21:20	14.26	3.13	17.39	1.22
17:25	16.26	4.18	20.44	1.26	21:25	13.99	3.44	17.43	1.25
17:30	16.33	4.12	20.45	1.25	21:30	13.68	3.39	17.08	1.25
17:35	16.20	3.75	19.94	1.23	21:35	13.18	2.98	16.16	1.23
17:40	16.16	3.73	19.88	1.23	21:40	12.96	2.85	15.81	1.22
17:45	16.30	3.68	19.98	1.23	21:45	12.76	2.75	15.51	1.22
17:50	16.29	3.54	19.83	1.22	21:50	12.87	2.92	15.79	1.23
17:55	16.37	3.53	19.90	1.22	21:55	12.70	2.75	15.45	1.22
18:00	16.56	3.49	20.04	1.21	22:00	12.38	2.79	15.17	1.23
18:05	16.46	3.33	19.79	1.20	22:05	12.18	2.74	14.92	1.22
18:10	16.50	3.43	19.93	1.21	22:10	12.07	2.74	14.80	1.23
18:15	16.44	3.16	19.60	1.19	22:15	11.93	3.47	15.40	1.29
18:20	16.55	3.36	19.90	1.20	22:20	11.79	3.60	15.39	1.30
18:25	16.89	3.71	20.59	1.22	22:25	11.63	3.67	15.30	1.32
18:30	17.07	3.90	20.96	1.23	22:30	11.32	3.62	14.94	1.32
18:35	17.20	4.09	21.29	1.24	22:35	11.03	3.07	14.10	1.28
18:40	17.37	4.02	21.39	1.23	22:40	10.80	2.91	13.71	1.27
18:45	17.62	4.12	21.73	1.23	22:45	10.39	2.48	12.87	1.24
18:50	17.58	4.11	21.69	1.23	22:50	10.12	2.56	12.68	1.25
18:55	17.50	4.04	21.55	1.23	22:55	9.87	2.28	12.14	1.23
19:00	17.69	3.99	21.68	1.23	23:00	9.59	2.12	11.71	1.22
19:05	17.74	3.69	21.43	1.21	23:05	9.46	2.16	11.62	1.23
19:10	17.53	3.41	20.95	1.19	23:10	9.07	2.27	11.34	1.25
19:15	17.88	3.37	21.25	1.19	23:15	8.72	2.18	10.90	1.25
19:20	17.91	3.41	21.32	1.19	23:20	8.55	1.94	10.49	1.23
19:25	17.73	3.29	21.01	1.19	23:25	8.43	1.96	10.39	1.23
19:30	17.31	3.18	20.48	1.18	23:30	8.29	1.94	10.23	1.23
19:35	17.23	3.12	20.35	1.18	23:35	8.09	1.99	10.08	1.25
19:40	16.87	3.35	20.22	1.20	23:40	7.96	2.43	10.39	1.31
19:45	16.60	3.34	19.93	1.20	23:45	7.88	2.49	10.38	1.32
19:50	16.49	3.22	19.71	1.20	23:50	7.69	2.59	10.28	1.34
19:55	16.42	3.08	19.50	1.19	23:55	7.44	2.72	10.16	1.37

Table L.2: Leakage Severity Sensitivity Calculations

Criteria	Sensitivity of Leakage Severity						
Equivalent Tenement of Catchment (ET)	1950	1950	1950	1950	1950	1950	1950
Equivalent Population (EP)	4622	4622	4622	4622	4622	4622	4622
Ratio EP / ET	2.37	2.37	2.37	2.37	2.37	2.37	2.37
Area of the Catchment (ha)	325.9	325.9	325.9	325.9	325.9	325.9	325.9
Density EP / Ha	14.18	14.18	14.18	14.18	14.18	14.18	14.18
Effective Area	100.2	100.2	100.2	100.2	100.2	100.2	100.2
Average Dry Weather Flow (L/s)	11.8	11.8	11.8	11.8	11.8	11.8	11.8
Peaking Factor "r"	2.1	2.1	2.1	2.1	2.1	2.1	2.1
Peak Dry Weather Flow (L/s)	24.5	24.5	24.5	24.5	24.5	24.5	24.5
Soil Aspect (S_{aspect}) Range 0.2 - 0.8	0.2	0.3	0.4	0.5	0.6	0.7	0.8
Network Defects (N_{aspect}) Range 0.2 - 0.8	0.2	0.3	0.4	0.5	0.6	0.7	0.8
Co-efficient $S_{aspect} + N_{aspect}$ (C)	0.4	0.6	0.8	1	1.2	1.4	1.6
Rainfall Duration	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour
Rainfall Frequency	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year
Intensity	26.8	26.8	26.8	26.8	26.8	26.8	26.8
Containment Standard	5 Year	5 Year	5 Year	5 Year	5 Year	5 Year	5 Year
Factor Size	0.78	0.78	0.78	0.78	0.78	0.78	0.78
Containment Factor	1.31	1.31	1.31	1.31	1.31	1.31	1.31
Rainfall Function (I)	27.4	27.4	27.4	27.4	27.4	27.4	27.4
Rainfall Dependant I/I (L/s)	30.7	46.1	61.5	76.8	92.2	107.6	122.9
Peak Wet Weather Flow (L/s)	55.3	70.6	86.0	101.4	116.7	132.1	147.5

Table L.3: Containment Standard Sensitivity Calculations

Criteria	Sensitivity of Containment Standard					
Equivalent Tenement of Catchment (ET)	1950	1950	1950	1950	1950	1950
Equivalent Population (EP)	4622	4622	4622	4622	4622	4622
Ratio EP / ET	2.37	2.37	2.37	2.37	2.37	2.37
Area of the Catchment (ha)	325.9	325.9	325.9	325.9	325.9	325.9
Density EP / Ha	14.18	14.18	14.18	14.18	14.18	14.18
Effective Area	100.2	100.2	100.2	100.2	100.2	100.2
Average Dry Weather Flow (L/s)	11.8	11.8	11.8	11.8	11.8	11.8
Peaking Factor "r"	2.08	2.08	2.08	2.08	2.08	2.08
Peak Dry Weather Flow (L/s)	24.5	24.5	24.5	24.5	24.5	24.5
Soil Aspect (S_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Network Defects (N_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Co-efficient $S_{aspect} + N_{aspect}$ (C)	1	1	1	1	1	1
Rainfall Duration	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour
Rainfall Frequency	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year
Intensity	26.8	26.8	26.8	26.8	26.8	26.8
Containment Standard	1 Year	2 Year	5 Year	10 Year	20 Year	50 year
Factor Size	0.78	0.78	0.78	0.78	0.78	0.78
Containment Factor	0.77	1.01	1.31	1.50	1.62	1.63
Rainfall Function (I)	16.0	21.0	27.4	31.3	33.7	34.0
Rainfall Dependant I/I (L/s)	45.0	58.9	76.8	87.8	94.6	95.3
Peak Wet Weather Flow (L/s)	69.6	83.4	101.4	112.3	119.1	119.8

Table L.4: Sensitivity of Storm Duration Calculations

Criteria	Sensitivity of Storm Duration					
Equivalent Tenement of Catchment (ET)	1950	1950	1950	1950	1950	1950
Equivalent Population (EP)	4622	4622	4622	4622	4622	4622
Ratio EP / ET	2.37	2.37	2.37	2.37	2.37	2.37
Area of the Catchment (ha)	325.9	325.9	325.9	325.9	325.9	325.9
Density EP / Ha	14.18	14.18	14.18	14.18	14.18	14.18
Effective Area	100.2	100.2	100.2	100.2	100.2	100.2
Average Dry Weather Flow (L/s)	12.0	12.0	12.0	12.0	12.0	12.0
Peaking Factor "r"	2.11	2.11	2.11	2.11	2.11	2.11
Peak Dry Weather Flow (L/s)	25.3	25.3	25.3	25.3	25.3	25.3
Soil Aspect (S_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Network Defects (N_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Co-efficient $S_{aspect} + N_{aspect}$ (C)	1	1	1	1	1	1
Rainfall Duration	1 hour	2 hour	3 hour	6 hour	12 hour	24 hour
Rainfall Frequency	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year
Intensity	26.8	36.2	44.0	63.0	91.2	127.2
Containment Standard	5 Year	5 Year	5 Year	5 Year	5 Year	5 Year
Factor Size	0.78	0.78	0.78	0.78	0.78	0.78
Containment Factor	1.31	1.31	1.31	1.31	1.31	1.31
Rainfall Function (I)	27.4	37.0	45.0	64.4	93.2	130.0
Rainfall Dependant I/I (L/s)	76.8	103.8	126.1	180.6	261.5	364.7
Peak Wet Weather Flow (L/s)	102.2	129.1	151.5	205.9	286.8	390.0

Table L.5: Sensitivity of Event Occurrence

Criteria	Sensitivity of Event Occurance					
Equivalent Tenement of Catchment (ET)	1950	1950	1950	1950	1950	1950
Equivalent Population (EP)	4622	4622	4622	4622	4622	4622
Ratio EP / ET	2.37	2.37	2.37	2.37	2.37	2.37
Area of the Catchment (ha)	325.9	325.9	325.9	325.9	325.9	325.9
Density EP / Ha	14.18	14.18	14.18	14.18	14.18	14.18
Effective Area	100.2	100.2	100.2	100.2	100.2	100.2
Average Dry Weather Flow (L/s)	12.0	12.0	12.0	12.0	12.0	12.0
Peaking Factor "r"	2.11	2.11	2.11	2.11	2.11	2.11
Peak Dry Weather Flow (L/s)	25.3	25.3	25.3	25.3	25.3	25.3
Soil Aspect (S_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Network Defects (N_{aspect}) Range 0.2 - 0.8	0.5	0.5	0.5	0.5	0.5	0.5
Co-efficient $S_{aspect} + N_{aspect}$ (C)	1	1	1	1	1	1
Rainfall Duration	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour
Rainfall Frequency	1 in 1 year	1 in 5 year	1 in 10 year	1 in 20 year	1 in 50 year	1 in 100 year
Intensity	23.7	37.2	44.9	52.9	64.3	73.6
Containment Standard	5 Year	5 Year	5 Year	5 Year	5 Year	5 Year
Factor Size	0.78	0.78	0.78	0.78	0.78	0.78
Containment Factor	1.31	1.31	1.31	1.31	1.31	1.31
Rainfall Function (I)	24.2	38.0	45.9	54.0	65.7	75.2
Rainfall Dependant I/I (L/s)	67.9	106.6	128.7	151.7	184.3	211.0
Peak Wet Weather Flow (L/s)	93.3	132.0	154.0	177.0	209.7	236.3

Table L.6: PWWF Catchment 3 Gravity, SPS 15, 21, 23, 26 and 29

Criteria	SPS 15	SPS 21	SPS 23	SPS 26	SPS 29	3 Gravity
Equivalent Tenement of Catchment (ET)	154	154	37	9	253	1263
Equivalent Population (EP)	365	365	88	21	600	2993
Ratio EP / ET	2.37	2.37	2.38	2.33	2.37	2.37
Area of the Catchment (ha)	68.6	29.6	22.1	2.9	3	199.7
Density EP / Ha	5.32	12.33	3.98	7.24	200.00	14.99
Effective Area	12.9	8.5	3.6	0.6	3.5	63.1
Average Dry Weather Flow (L/s)	0.41	1.28	0.49	0.15	2.62	7.16
Peaking Factor "r"	2.91	2.53	6.54	2.19	2.1	2.11
Peak Dry Weather Flow (L/s)	1.19	3.24	3.20	0.33	5.50	15.1
Soil Aspect (S_{aspect}) Range 0.2 - 0.8	0.8	0.8	0.8	0.5	0.5	0.8
Network Defects (N_{aspect}) Range 0.2 - 0.8	0.8	0.8	0.8	0.5	0.5	0.8
Co-efficient $S_{aspect} + N_{aspect}$ (C)	1.6	1.6	1.6	1	1	1.6
Rainfall Duration	1 hour	1 hour	1 hour	1 hour	1 hour	1 hour
Rainfall Frequency	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year	1 in 2 year
Intensity	26.8	26.8	26.8	26.8	26.8	26.8
Containment Standard	5 Year	5 Year	5 Year	5 Year	5 Year	5 Year
Factor Size	0.94	1.04	1.07	1.37	1.36	0.82
Contaiment Factor	1.31	1.31	1.31	1.31	1.31	1.31
Rainfall Function (I)	33.0	36.5	37.8	48.3	48.1	29.0
Rainfall Dependant I/I (L/s)	19.1	13.9	6.1	0.9	4.7	82.1
Peak Wet Weather Flow (L/s)	20.3	17.1	9.3	1.2	10.2	97.2

Appendix M: Nowra IFD Charts

Australian Government
Bureau of Meteorology

Location

Label: Nowra

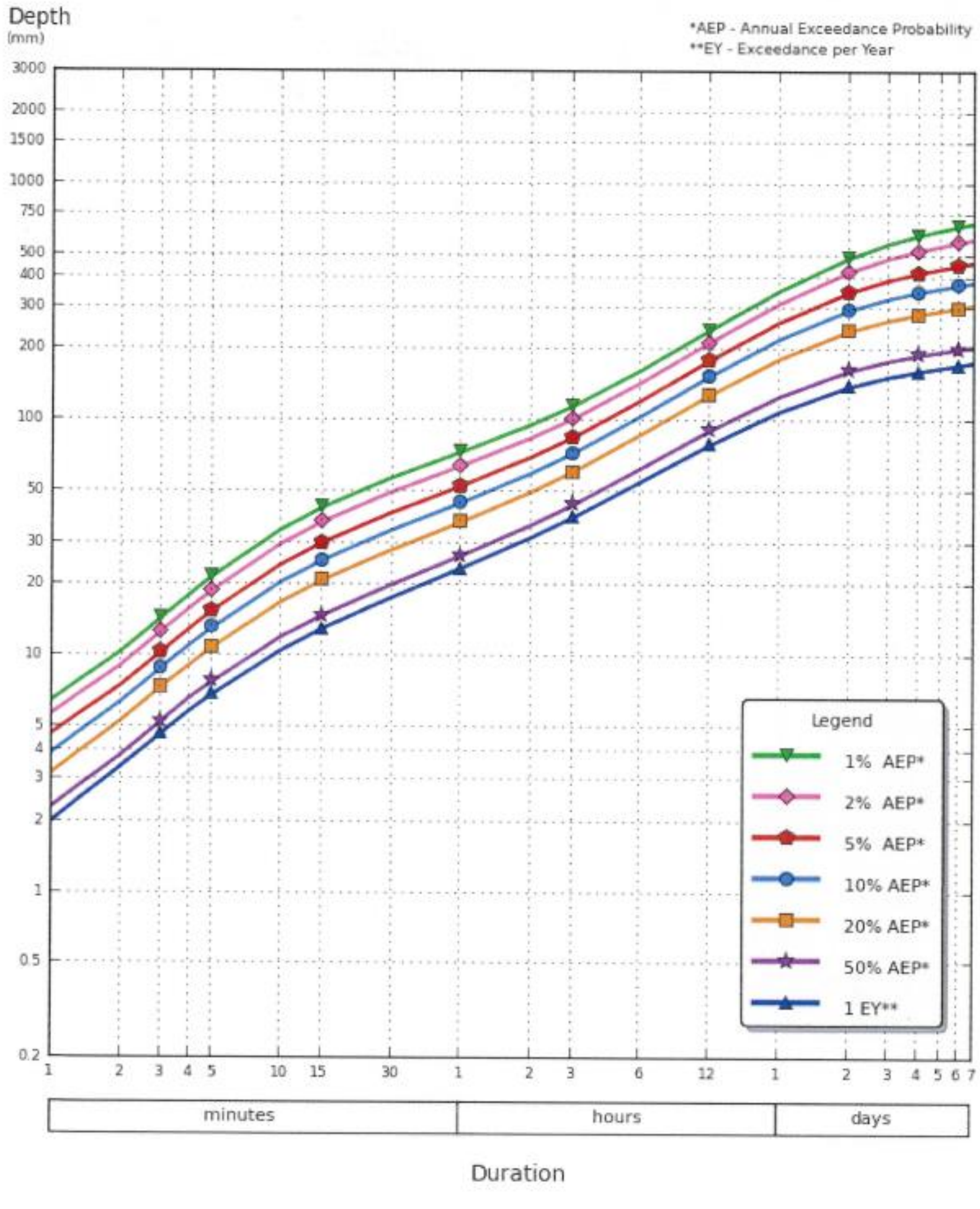
Latitude: 34.95 [Nearest grid cell: 34.9375 (S)]

Longitude:150.54 [Nearest grid cell: 150.5375 (E)]

IFD Design Rainfall Depth (mm)

Rainfall depth for Durations, Exceedance per Year (EY), and Annual Exceedance Probabilities (AEP).

	EY	Annual Exceedance Probability (AEP)					
Duration	**1EY**	**50%**	**20%**	**10%**	**5%**	**2%**	**1%**
1 min	2.0	2.3	3.2	3.9	4.7	5.7	6.5
2 min	3.4	3.8	5.3	6.4	7.5	9.1	10.4
3 min	4.7	5.3	7.4	8.9	10.4	12.7	14.5
4 min	5.9	6.7	9.2	11.2	13.1	16.0	18.3
5 min	6.9	7.8	10.9	13.2	15.6	19.0	21.8
10 min	10.6	12.1	17.0	20.7	24.5	30.0	34.5
15 min	13.1	14.9	21.0	25.5	30.3	37.1	42.7
30 min	17.8	20.2	28.3	34.4	40.7	49.8	57.2
1 hour	23.7	26.8	37.2	44.9	52.9	64.3	73.6
2 hour	32.1	36.2	50.0	60.0	70.3	84.7	96.3
3 hour	38.9	44.0	60.7	72.7	84.9	101.8	115.3
6 hour	55.3	63.0	87.7	104.9	122.2	145.6	164.1
12 hour	79.3	91.2	128.9	155.0	180.9	215.6	242.7
24 hour	109.6	127.2	183.6	222.8	262.0	314.9	356.3
48 hour	140.0	163.5	240.4	295.4	351.4	428.9	491.0
72 hour	154.5	180.7	267.8	331.4	397.3	490.2	565.9
96 hour	162.9	190.6	283.6	352.4	424.6	527.7	612.8
120 hour	168.7	197.3	294.2	366.7	443.2	553.5	645.3
144 hour	173.3	202.7	302.7	378.0	457.8	573.4	670.2
168 hour	177.5	207.6	310.5	388.1	470.6	590.5	691.1



Depth
(mm)
*AEP - Annual Exceedance Probability
**EY - Exceedance per Year
3000
2000
1500
1000
750
500
400
300
200
100
50
30
20
10
5
4
3
2
1
0.5
0.2
1
2
3
4
5
10
15
30
1
2
3
6
12
1
2
3
4
5
6
7
minutes
hours
days
Duration
Legend
1% AEP*
2% AEP*
5% AEP*
10% AEP*
20% AEP*
50% AEP*
1 EY**

Appendix N: Field Work Catchment Plans

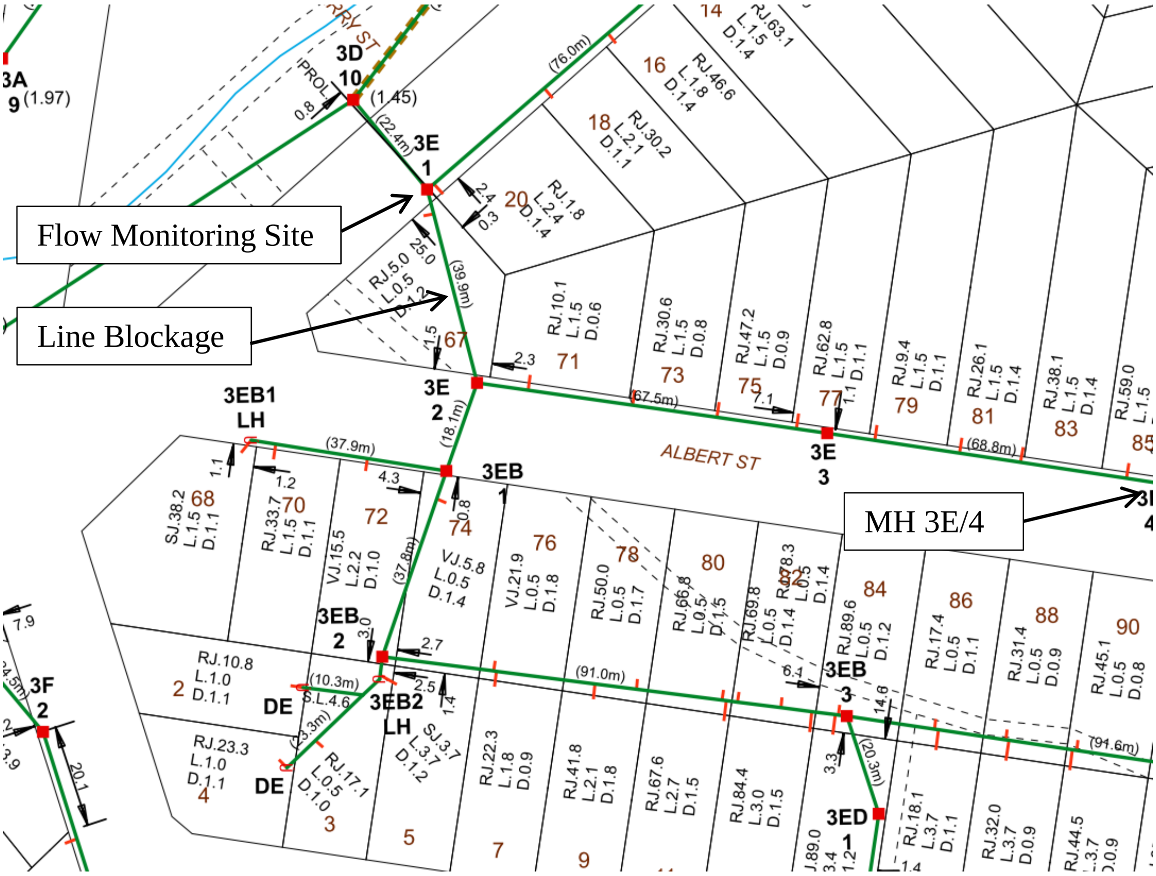


Figure N.1 - Field Work Catchment Plan 1 of 2

Figure N.2 - Field Work Catchment Plan 2 of 2