# Implementing Water Sensitive Urban Design approaches under the existing developments in urban areas

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# Abstract

Urbanisation and population growth lead to increases in impervious area in urban catchments, which also brings more stormwater volume and increases pollutant loads. Consequently, the negative impacts of stormwater on freshwater have increased. Water Sensitive Urban Design (WSUD) techniques are becoming popular as they help planners and authorities to manage urban water resources in an integrated way to reduce the impacts of stormwater runoff on receiving environment.

While research on Water Sensitive Urban Design (WSUD) has been ongoing for a considerable period, there are still gaps in our understanding of the scale, benefits, and effectiveness of implementing the systems, especially in the context of existing urban conditions. This study aims to address these knowledge gaps by exploring the optimal locations for implementing WSUD techniques implementation, at allotment and sub-catchment scales, with a focus on managing stormwater quantity and quality under the influences of urbanization and climate change in an urban catchment's outlet. In addition, distributed WSUD implementation techniques at different scale were the focus of this research. Distributed systems mean to locate these approaches within smaller lots and at the available spaces throughout the development area. In contrast to greenfield developments, the implementation of WSUD in existing developments (suburbs) has various constraints and impediments. This research has identified the constraints and impediments for the selection of appropriate WSUD techniques for implementation in an existing development. Another component of this research was to identify the most prominent scale for WSUD implementation and develop a framework to identify WSUD assets spatially to achieve desired flow management and stormwater quality at a catchment outlet. Two urban catchments were selected for this study located in Wyndham City municipality in Melbourne's west. Finding of this study advised that WSUD be taken into account as a potential solution in small sub-catchments to lower the study area's stormwater volume at the catchment outlet. In addition, the storm water system's load can be decreased by strategically putting WSUD techniques within the allotments or the available green open spaces. Though adding more WSUD techniques will not negatively impact a solution's performance, there are ways to promote acceptance among the community. As the effects of climate change continue to raise the risk, these solutions could also be examined in conjunction with other stormwater management techniques to see which one is the most economical in reducing the flow rate and volume of urban stormwater runoff. The findings indicate that the difference in total relative cost between optimal and nonoptimal solutions can reach well below $20 million dollars, and the effectiveness of load reduction for both stormwater quality and quantity at the study catchment could vary as much as 33% to 40% adopting a combination of raingardens, infiltration trenches and permeable pavements. Modelling and optimization in this study showed that, when applied in small sub-catchments, WSUD techniques performed better in terms of improving stormwater quality and reducing stormwater quantity. The improvement at a catchment scale was led to the same outcomes, as demonstrated by a comparison of two urban catchments and WSUD techniques implementation in extremely small and larger catchments size from 2 ha. The study outcomes will help local governments and relevant organisations manage their stormwater Master plans to make urban areas more sustainable and liveable.

# Declaration of Authenticity

“I, [Samira rashetnia], declare that the (PhD) thesis entitled [Implementing Water Sensitive Urban Design approaches under the existing developments in urban areas] is no more than 70,547 words in length including quotes and exclusive of tables, figures, appendices, bibliography, references, and footnotes. This thesis contains no material that has been submitted previously, in whole or in part, for the award of any other academic degree or diploma. Except where otherwise indicated, this thesis is my own work”.

“I have conducted my research in alignment with the Australian Code for the Responsible Conduct of Research and Victoria University’s Higher Degree by Research Policy and Procedures.

# Ethics Declaration

“All research procedures reported in the thesis were approved by the [Human Research Ethics HRE18-133].”

[redacted]

**(30/09/2024)**

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# Abbreviations

| | |
|---|---|
| ARR | Australian Rainfall and Runoff |
| ARI | Annual Recurrence Interval |
| ABS | Australian Bureau of Statistics |
| AEP | Annual Exceedance Probability |
| BOD | Biochemical Oxygen Demand |
| BoM | Bureau of meteorology |
| BMP | Best Management Practice |
| BPP | Best Planning Practice |
| BMPs | Stormwater Best Management Practices |
| C1 | Catchment 1 |
| C2 | Catchment 2 |
| EIA | Effective Impervious Area |
| EY | Exceedances per Year |
| GI | Green Infrastructure |
| GPT | Gross Pollutant Traps |
| IFD | Frequency Duration |
| LCCA | Life Cycle Cost Analysis method |
| LCC | Life Cycle Cost |
| LID | Low Impact Development |
| LIUDD | Low Impact Urban Design and Development |
| MW | Melbourne Water |
| NPV | Net Present Value |
| RWT | Rainwater tank |
| RG | Raingarden |
| SFEI | San Francisco Estuary Institute |

| | |
|---|---|
| SUDS | Sustainable Urban Drainage Systems |
| SuDS | Sustainable Drainage Systems |
| SWMM | Storm Water Management Model |
| SWM | Urban stormwater management |
| TIA | Total Impervious Area |
| TDS | Total Dissolved Solids |
| TSS | Total Suspended Solids |
| TN | Total Nitrogen |
| TP | Total Phosphorus |
| USEPA | The United State Environmental Protection Agency |
| WSUD | Water Sensitive Urban Design |
| WCC | Wyndham City Council |

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# 1. Chapter 1: Research Background and Importance

## 1.1. Introduction

Development and urbanisation transform the natural environment to the built environment. Consequently, vegetated areas are replaced with impervious surfaces such as roofs and roads. More impervious areas lead to more stormwater runoff volume and pollutants (Trudeau and Richardson 2015). In addition, there is a widespread recognition of urban stormwater runoff influence on stream ecosystems (Fletcher et al., 2014). Effective urban stormwater management (SWM) is crucial for mitigating the impacts of urbanization, and existing methods must adapt to meet the growing demands associated with urban growth and climate change. Hence, it is essential to apply an integrated approach such as WSUD to manage urban water systems and reduce the impacts of stormwater to make our cities more resilient and liveable (Molglia et al., 2012). Coombes (2018) explored the shift from conventional stormwater drainage systems to a holistic urban water cycle management approach, emphasizing the need to move beyond standalone green initiatives. Their key points include systems approach (advocates for integrating stormwater management with potable water, wastewater, and environmental flows), limitations of green pilots (While green infrastructure (e.g., rain gardens, green roofs) provides benefits, their isolated implementation lacks scalability and does not fully address broader urban water challenges, integrated planning (stresses the importance of cross-sector collaboration to develop sustainable and resilient urban water systems that align with land use and urban planning), policy and regulation: Calls for policy reforms to promote integrated water management strategies, shifting away from traditional drainage-centric approaches.

Brown et al., (2009a) explained that stormwater was seen as a “nuisance” as in the past and, was generally discharged into waterways untreated (Wong 2006). An approach was initiated in Australia in the early 1990s which is termed as “Water Sensitive Urban Design” and includes consideration of a long- term socio-environmental sustainability (Coombes et al., 1999; Argue 2004). Since then, the new approaches and systems have been studied and adopted in several cities to control stormwater quantity and improve stormwater quality.

In Australia, the primary focus has generally been on improving water quality. Although research on WSUD has been continuing for some time, there are still knowledge gaps in understanding implementation of the systems as well as their benefits and effectiveness. This study has explored optimal locations and type of WSUD approaches, for stormwater quantity and quality management, under land use change and climate change in existing urban development. Under this research distributed WSUD approaches were investigated. In contrast to greenfield developments, the implementation of WSUD in existing developments (suburbs) has additional constraints and impediments. Besides a literature review, a local survey has been conducted for this study obtaining local council's feedback on the constraints and impediments in urbanised catchments and the findings is published in a conference paper (Hydrology and Water Resource Symposium 2018). Therefore, this research has identified constraints and impediments of WSUD assets that affect the selection of most appropriate WSUD approaches for implementation in existing developments.

Our planet is critically under water stress, and it has been reported that currently, 1 billion people do not have access to clean drinking water (Mithen 2010). Climate change is likely to have further impact on water supplies as it has been forecast that precipitation patterns will change, and evaporation will increase (IPCC 2013; UNEP 2007). Recent reports specified that the two-thirds of the world population will most probably live under water stress by 2050 (UN Water 2007) so, it is a big challenge to accommodate growing populations with finite freshwater resources. Furthermore, population growth and urban development increase the area of impervious surfaces and cause various environmental harmful and pollutant substances due to the stormwater runoff.

Therefore, in order to manage urban water systems and minimize the effects of stormwater and increase the resilience and livability of our cities, it is essential to implement an integrated approach like WSUD. (Molglia et al., 2012). In the early 1990s in Australia, an approach was initiated which presently is termed as "Water Sensitive Urban Design" and consider a long- term socio-environmental sustainability (Coombes et al., 1999).

Since then, the new approaches and systems have been studied and adopted in several cities to control the stormwater quantity and improve the stormwater quality. Although the research in WSUD area has been continuing for some time, there are still wide knowledge gaps in understanding their implementation, benefits, and effectiveness (Bonneau et al., 2017; Li et al., 2017; Sargen 2015; Fletcher et al., 2014; Brown et al., 2013, McMillan et al., 2012, Ahiablame et al., 2012, Blecken et al., 2011 and Balakrishna et al., 2006). Some of the knowledge gaps identified through the literature and listed below:

- Understand the effectiveness of WSUD approaches to manage stormwater quality under different land use.
- Understand the WSUD performance under climate variability.
- Understand the WSUD role in enhancing the capacity of stormwater systems (by peak flow reduction).
- Understand the role of WSUD approaches in urban stream ecosystems protection by understanding the flow regime aspects to improve the knowledge on addressing geomorphic change through flow- regime management.
- Understand the urban stormwater runoff influence on stream ecosystems and the role of WSUD to reduce the negative impacts of runoff.
- Classify field and experimental data of WSUD approaches.
- Understand fate of infiltrated stormwater and groundwater pathways.
- Understand the WSUD treatment process.
- Understand the role of WSUD application to restore altered urban flow regime and how to translate the site scale outcomes to catchment scale ones.

Among the identified gaps, this study has explored the role of WSUD approaches under land use and climate change in existing urban catchment for stormwater quantity and quality management. In this research distributed WSUD implementation approaches were investigated. Distributed WSUD means to locate them within smaller lots and at the available spaces throughout the urban area. Moreover, unlike the greenfield developments, the implementation of WSUD in existing developments (suburbs) has various constraints and impediments. This research has identified these constraints and impediments for the selection of most appropriate WSUD approaches for implementation in an existing development based on various factors. The research aims and questions are explained in section 1.2.

## 1.2. Research Aims, Questions, and Significance

Since the early 1990s, the concept of Water Sensitive Urban Design (WSUD) has emerged as a sustainable response to urban stormwater management challenges (Coombes et al., 1999). Over the years, various WSUD approaches have been studied and implemented in cities worldwide to control stormwater quantity and improve water quality. However, despite continued research, significant knowledge gaps remain regarding the effectiveness, implementation challenges, and broader impacts of WSUD (Bonneau et al., 2017; Li et al., 2017; Fletcher et al., 2014).

Several studies have identified key areas requiring further investigation, including the effectiveness of WSUD under different land-use scenarios, its resilience to climate variability, and its role in protecting urban stream ecosystems (Sargen, 2015; Brown et al., 2013). Additionally, the application of WSUD in existing urban development's presents unique challenges, particularly concerning spatial constraints and system integration (Walsh et al., 2015). The need to better understand these aspects is crucial as urbanization accelerates, and climate change alters precipitation patterns, increasing the risk of flooding and water shortages (IPCC, 2013; UNEP, 2007).

Building upon these gaps in knowledge, this study aims to explore the role of WSUD approaches in mitigating stormwater impacts under land use and climate variability. Specifically, it has examined both distributed and end-of-pipe WSUD implementation strategies in existing urban developments, addressing key constraints and impediments to their adoption.

This research has focused on the following aims:

To develop a framework for the selection and application of WSUD techniques to mitigate the effects of urbanization under land use and climate change on stormwater quantity and quality in existing development.

Hence this research will explore solutions to the following research questions:

1- What are the constrains on WSUD implementation in an existing urban area?
2- What are the WSUD approaches which can be implemented optimally considering their locations, size and type to manage the stormwater quality and quantity?
3- How long the augmentation of the stormwater system can be deferred in the study area by WSUD approaches implementation considering climate variability and urbanisation?

Answering the above questions will fill significant knowledge gaps and makes the research significant as described in the following section:

Firstly, literature review indicates that most of the exiting research in WSUD systems has focused on their implementation in new infill developments or greenfield sites. Therefore, significant knowledge gaps exist in the implementation of WSUD approaches in existing developments without using holistic approaches. The proposed research investigated the optimal location of WSUD implementation in existing urban development considering land use change and climate change through an urban catchment.

Secondly, the information about the role of WSUD approaches in stormwater quality and quantity management at a catchment scale is lacking. Most of the existing conducted research are based on no or limited data on stormwater quality and quantity which are required in validating or calibrating the model. The outcomes of their studies are of limited use for water industry. Therefore, the proposed study undertook stormwater quantity and quality monitoring of an urban catchment in Melbourne's west to validate the modelling tools and address the effectiveness of chosen WSUD techniques at a catchment-scale. Thus, the outcome from the research will be more reliable.

The local governments and municipal authorities assign sustainable alternative methods to manage, capture and treat the stormwater in their areas by applying approaches like WSUD, however, it is challenging to determine an optimal mixture of different approaches at a range of scales, to manage the stormwater quality and quantity and minimize the overall cost. A few studies have explored optimal sizes of some of the WSUD approaches, however, this study combined a hydrological/hydraulic model and a suitable optimisation method to identify the most prominent scale for WSUD implementation and develop a framework to identify WSUD assets spatially to achieve desired flow management and stormwater quality at a catchment scale. The adopted simulation-optimization tool can be used in any urban area to identify the optimal locations of the WSUD approaches. Therefore, this study will make a significant contribution to existing knowledge benefiting researchers and planners in this area.

The outcome of the study will suggest a framework for WSUD systems implementation and has been listed the suitable implementation approaches in small and large existing developments. Moreover, constraints of implementing WSUD approaches were investigated considering the land use change in existing developments. These findings and suggestions are a new contribution to existing knowledge towards the application of WSUD approaches and will support the planners and engineers to better manage the stormwater quality and quantity in relevant organisations and local governments.

### 1.3. Research Study Area

Wyndham City Council (WCC) is located on the western edge of Melbourne. The area under the council's jurisdiction is experiencing rapid growth. Furthermore, the area has quite significantly different climate and rainfall patterns from the northern and eastern suburbs (Wyndham city stormwater management plan, 2015). To understand the optimal locations of WSUD systems and the best adoption method, to manage stormwater quality and quantity in urbanised catchments, two urban catchments were selected from WCC municipality of Melbourne for this study. These areas are well established and have been selected as this research is focussed on existing developments. Figure 1-3 shows the WCC municipality boundary and location in Melbourne. Figure 1-4 shows the selected urban catchments locations and areas.

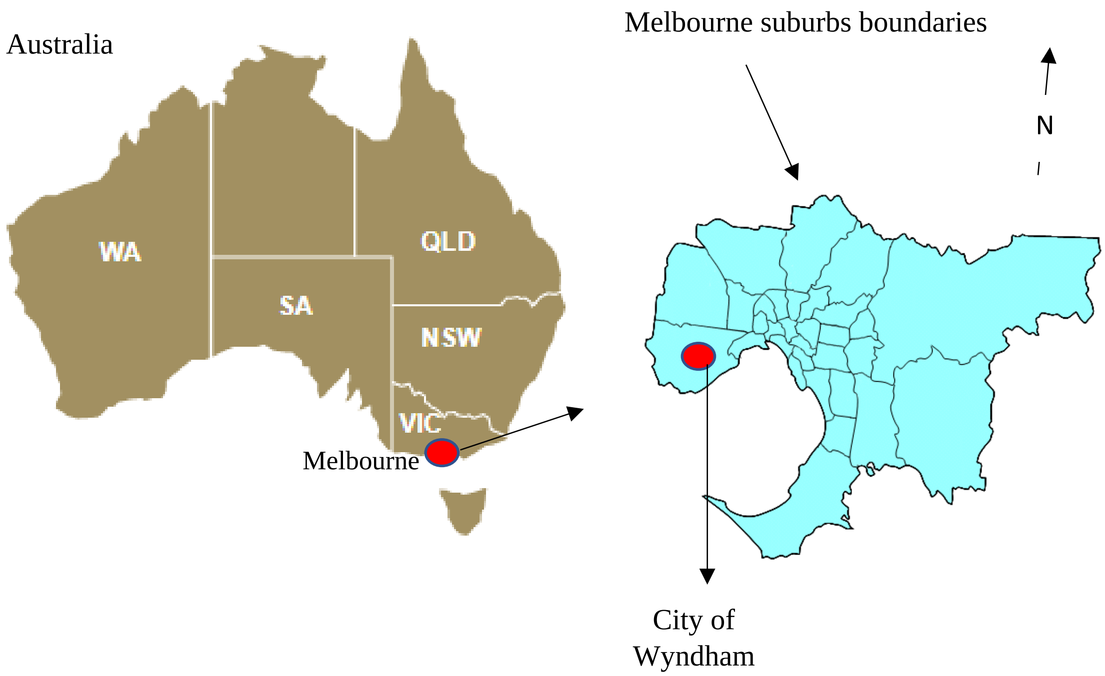



*Figure 1-1- Location of the WCC in Melbourne and Australia*

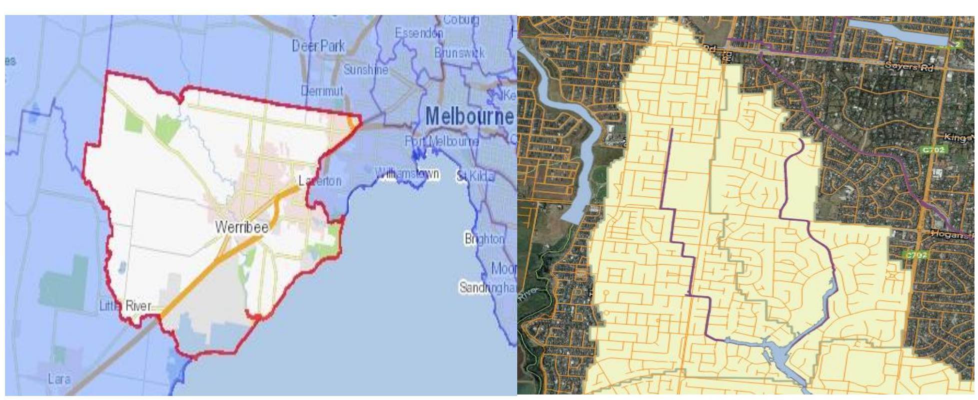



*Figure 1-2- The boundary of the urban catchments and their location in WCC municipality*

Wyndham City Council (WCC) which is located on the western edge of Melbourne and covers an area of 542 $km^2$ is experiencing rapid growth and development. In addition, the area has quite significant different climate and rainfall patterns within the northern and eastern suburbs. Two developed urban catchments were selected for the monitoring program. The catchments cover total area of 5.7 km2 and located on the left side of the Pacific Werribee shopping centre

## 1.4. Thesis Outlines

The first chapter of the research thesis provided an overview of the research background, significance, innovation, aims, questions followed by an outline of the thesis. Chapter two delved into the literature review, covering the history of WSUD approaches, terminology, benefits, implementation methods, local guidelines, fact sheets, and considerations for selecting suitable systems. Chapter Three outlines the research methodology, detailing each step involved in the study. It provides a concise description of each step, and the scenarios developed for stormwater modelling and optimization within the study area to answer the research questions. Chapter four presented considerations and essential criteria for WSUD system selection, including insights from a local survey conducted among municipal councils in Melbourne. Chapter five addressed the collection of stormwater quality and quantity data in the study area and outlines the challenges faced. In addition, covered the stormwater modelling tool (i.e., PCSWMM) calibration. Chapter six discussed the scenario modelling of stormwater quantity for chosen WSUD techniques at a typical allotment, a sub-catchment and through the whole study area. Chapter seven elaborated on the parameters and specifications of WSUD techniques adopted in the study and simulated in PCSWMM model. In Chapter eight, the modelling extends optimisation process for chosen WSUD techniques location selection at the sub-catchment scale. Finally, Chapter nine summarizes the findings and provided further recommendations.

# 2. Chapter 2: Literature Review

## 2.1. Introduction

This chapter provides an overview of WSUD system history, identified systems and implementation methods, benefits, available guidelines and WSUD selection considerations. The presented content in this chapter provides an overview of WSUD benefits and their role in mitigating against the impacts affecting urban stormwater systems.

## 2.2. Impact of Urbanisation on Stormwater

Rapid urbanisation causes various changes to the natural environment while puts more stress on urban stormwater management systems. Urbanisation also results in multiple changes to flow regimes in waterways including increases in the number and size of flood peaks and increases in the flow volume (Li et al., 2017; Chui et al., 2016). These changes are caused by importation of piped water, reductions in interception, infiltration and evapotranspiration, and reduced catchment storage (Liu et al., 2015). There can also be changes to the lowest flows in the catchment due to leakage from water supply and sewage systems (Ladson, 2019; Bonneau et al., 2017; Prosdocimi et al., 2015). Peak flows in urbanised catchments happen even more than 30% to 100% greater peak flows than in non-urbanized or less urbanized catchments (Rose and Peters 2001). The swift escalation of urbanization, coupled with environmental degradation and ecological harm in urban areas, has posed a significant impediment to achieving sustainable urban development. Indeed, the replacement of vegetated areas, naturally capture and store rainwater, often results in an increase in both the velocity and volume of stormwater runoff. (Chen et al., 2016).

## 2.3. Impact of Climate Change on Stormwater

Stormwater infrastructures were designed to minimise the rainfall impacts on human settlements however, climate change can make them ineffective (Cobbina, 2007). Climate change is likely to have further impact on water supplies as it has been forecast that precipitation patterns will change, and evaporation will increase (IPCC 2013; UNEP 2007). Resilience to climate change impacts is a critical challenge for urban communities particularly regarding water resources sustainable management and water environment protection (Wong and Brown 2009). Further, it is a challenge to accommodate growing populations with finite freshwater resources as urban areas expand rapidly. Currently, frequent, and large-scale flooding is increasing in urban areas due to climate change (Kang et al., 2016), and its influence on urban drainage systems is a major issue (Woods & Morgan 2010). Coombs and Barry 2014, developed a report for Ballarat city in Melbourne and analysed integrated water cycle management strategies, emphasizing stormwater management, climate impacts, and sustainable infrastructure planning. They highlighted the benefits of holistic approaches in mitigating flooding, protecting waterways, and enhancing urban resilience.

### 2.3.1. Adopted Climate and Land use Changes Scenarios in this research

To understand the WSUD performance under existing conditions in this research scenarios explained in Table 2-1 were adopted for climate change impact and land use (DELWP guideline 2020).

Table 2-1- Adopted climate change scenarios for optimization in this study

| Explanation | Existing study area condition | Future study area condition |
|---|---|---|
| Land use/urbanization | 56-57% | 77-80% |
| Climate change | One year rainfall data from the closest rainfall station to the study area | Increase in the rainfall intensity by 7.3% by 2050 |

- Land use: Urban areas are rapidly developing and becoming denser. The current situation of the study area was investigated to realize the impervious percentage. The value (57–64%) was then calibrated using actual data that had been gathered for the study area. The existing lots were assumed to be replaced with at least two dwellings during future construction, which would about increase the fraction impervious by 20% more.

- Climate change: Proper planning for new infrastructure and preventing potential harm to existing infrastructure require an understanding of the risks posed by climate change. Climate change is predicted to influence five components of design flood estimation (ARR_2019):

• Rainfall Intensity Frequency Duration (IFD) relationships.

• Rainfall temporal patterns.

• Continuous rainfall sequences.

• Antecedent conditions and baseflow regimes; and

• And compound extremes (e.g., riverine flooding combined with storm surge inundation).

The research study area's interim climate change factors were exported using the site's longitude and latitude utilising Australian Rainfall & Runoff Data Hub (https://data.arr-software.org/). The recommended increase in rainfall intensity for the area by 2050 was 7.3%, which used to calculate the future rainfall amounts.

$$Intensity\ future = \ Intensity\ \mathrm{current}\ *\ 1.05^{Tm} \qquad (2\text{-}1)$$

where $Intensity$ current is the design rainfall intensity (or depth) for the current climate, $T$is the temperature at the middle (or median) of the chosen class interval. Based on the roughly exponential relationship between temperature and humidity, 1.05 is the predicted temperature scaling, and the Tm is the temperature measured at the middle (or median) of the chosen class interval.

## 2.4. History of WSUD

Water sensitive urban design (WSUD) holds the promise of reducing the pollution of freshwater, providing additional water for human use, decreasing flooding potential, protecting ecosystems, and improving urban amenity. These are vital goals given increasing populations, rapid urbanisation, current water scarcity and climate change impacts on urban water resources and temperatures (Mekonnen and Hoekstra, 2016; IPCC 2019; Ritchie and Roser, 2019).

Stormwater is often considered a "nuisance" (Brown et al., 2009a). The primary focus of stormwater management was to develop efficient systems to convey stormwater quickly away from urban areas. The impact of such systems on water quality and flow patterns has also been recognised and thus integrated approaches to water quality and flow management are being promoted. Given the increased demand for water due to population growth, it is also essential to apply an integrated approach such as WSUD to consider stormwater as a potentially useful source and manage urban water systems to reduce the impacts of stormwater and make our cities more resilient and liveable (Molglia et al., 2012).

Integrated stormwater management is known by a variety of names, including Low Impact Development (LID), Low Impact Urban Design and Development (LIUDD), Stormwater Best

Management Practices (BMPs), Sustainable Urban Drainage Systems (SUDS), Sustainable Drainage Systems (SuDS), and Green Infrastructure (GI) (Benedict & McMahon, 2006; Fletcher et al., 2015). LID was used in Vermont, USA for the first time in a report on land use planning (Barlow et al., 1977). Later, in the early 1990s in Australia, creative approaches were developed which are presently termed "Water Sensitive Urban Design" and are focused on long-term socio-environmental sustainability (Coombes et al., 1999). These innovative systems have been studied and adopted in several cities across the globe to manage stormwater quantity, improve stormwater quality and to facilitate stormwater harvesting to augment fresh water supplies. The term WSUD was originated over two decades ago in Australia (Whelans et al., 1994). While LID has been often used in New Zealand and North America. The term BMP is used in North America to describe structured approaches to prevent the pollution in waterways (Fletcher et al., 2015). In the UK, the approaches to the stormwater management started in the late 1980s and the "Scope for Control of Urban Runoff" guidelines were published in 1992 to provide guidance on a range of technical stormwater control options (CIRIA UK 2007). The term GI which goes far beyond stormwater management emerged in the USA in the 1990s. This term has its origin in both landscape architecture and landscape ecology (Benedict & McMahon, 2006). Stormwater control measures (SCMs), compensatory techniques (CTs), alternative techniques (Ats) and source control are some other terms which have been used to describe structured WSUD responses in the US and France (Fletcher et al., 2015).

WSUD aims to manage stormwater runoff and reduce urbanisation impact on water cycle and waterways and is applicable to scales from a single property to a whole city (Lloyd, 2001). Lloyd et al., (2002, p. 2) described WSUD as a series of useful techniques for urban planning and design which minimise the hydrological impacts of urban development on the surrounding environment.

From engineering and urban planning points of view, WSUD approaches contribute to urban sustainability and improve liveability by integrating urban planning aspects and design based on the management, conservation and protection of the whole water cycle (Melbourne Water, 2009). In general, WSUD is broadly defined as an integrated approach which incorporates urban water cycle including water supply, wastewater, stormwater, groundwater as well as environmental protection and urban drainage (JSCWSC 2009).

Adopting WSUD has the potential to provide benefits that can be classified into three main groups as social, environmental and economic (Symons et al., 2015). These benefits include improved human well-being, improved microclimate, mitigation of climate change, air quality improvement, water cycle restoration, soil improvements, biodiversity enhancement, preservation of vegetation, increased amenity, improvements in commercial vitality, and increased ecosystem services. WSUD may also increase public awareness of design and construction of urban development and foster significant economic, social, and environmental benefits, which come from the sustainable and efficient use of water resources (Sargent 2015; Myers et al., 2013; Sharma et al., 2016).

WSUD systems categorised into filtration-based and retention-based technologies which can contribute to reducing a watershed "effective impervious area" (Fletcher et al., 2014). These systems can be applied at a residential property (house) scale, street scale in large development areas or integrated into the suburban scale (Department of Planning and Local Government, South Australia 2010). At the scale of a city there are attempts to create 'sponge cities and water sensitive cities (Radcliffe 2017). Sponge Cities can be classified as those where stormwater is naturally infiltrated, conserved and purified for potential reuse (Tu and Tian, 2015). A summary of WSUD approaches at a range of scales is provided in Table 2-2.

Table 2-2-List of WSUD approaches at a range of scales *(JSCWSC 2009).*

| Scales | **WSUD approaches** | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Rainwater Tank | Rain Garden | Green roof | Permeable pavements | Gross pollutant | Bio-retention | Swales and buffer strips | Sedimentatio n basins | Constructed wetlands | Infiltration Systems |
| Single residential property | X | X | X | | | | | | | X |
| Residential subdivision development | X | | | X | X | X | X | X | X | |
| Residential multi-unit development | X | X | X | X | X | X | | | | X |
| Streetscape development | | | | X | X | X | X | X | | X |
| Vehicle parking areas | | X | | X | X | X | X | X | X | X |
| Commercial and industrial development | X | | X | X | | X | X | X | X | X |

## 2.5. WSUD Key Benefits

Most WSUD technologies are intended to return a developed urban catchment to its pre-development hydrological condition (Damodaram et al., 2010; Shuster et al., 2008). Technologies can be implemented as a retrofit design in developed urban area to reduce the urban stormwater infrastructure stress or increase the climate change resilience. Multiple outcomes were listed for WSUD technologies which are: stormwater quality improvement; flow regime management; stormwater harvesting; flood control; mitigation of heat island impacts; protection of stream ecosystems; sustainability, social benefits, landscape amenity and economic value; mitigating climate change impacts; and management of sewer overflows (Duncan et al., 2014, Fletcher et al., 2014; Sarawat et al., 2016; Jamei and Rajagopalan 2017).

## 2.6. WSUD Local Guidelines

The first WSUD design guideline was released in 1994 in Western Australia (Whelans et al., 1994) and in 2004 Argue edited and published a book covering the concept of WSUD approaches and design approaches in the Australian context “Water Sensitive Urban Design: Basic Procedure for

Source Control". Since then, several local factsheets, guidelines and procedures have been developed in Australia and overseas for the implementation of WSUD approaches for flood control, pollution control and stormwater harvesting (Armitage et al., 2013; Melbourne Water 2013; Department of Planning and Local Government, South Australia 2010; U.S. EPA 2008; CIRIA UK 2007; CSIRO 2006).

## 2.7. WSUD Performance

Effectiveness of WSUD technologies can be identified via their hydrological and pollutant removal capabilities, however, it is more difficult to evaluate their capability to change hydrological condition of an area than removing pollutant concentration. Evaluating the hydrological impact of WSUD is inherently more challenging due to the necessity for long-term, large-scale monitoring, sophisticated modelling, and the influence of dynamic factors such as climate variability and land-use changes. Conversely, assessing pollutant reduction is comparatively straightforward, as it is based on quantifiable variations in stormwater quality over shorter timescales. Several studies have investigated the potential effects of rainwater tanks, bio-retentions (rain gardens), infiltration systems, permeable pavements, and vegetated swales to manage stormwater quantity to reduce the impact of urbanisation and development in urban areas (Goncalves et al., 2018; Ahamed 2017; Davis et al., 2012; Trowsdale and Simcock 2011; Yang et al., 2009).

### 2.7.1. Observed performance of WSUD assets for managing stormwater quantity

Limited studies have reported the performance of bio-retention and other WSUD systems. Bioretention: Bioretention or raingardens are devices composed of media (sand, gravel, loam) and trees or plants. They are suitable for storage, infiltration and evaporation of both direct rainfall and the runoff captured from surrounding areas and convey the excess stormwater to a pervious area. Raingardens are effective in stormwater quantity management by reducing runoff (Yang et al., 2009; Trowsdale and Simcock 2011). Table 2-3 reports reductions in mean annual volume flows and peak flows by adopting raingardens from some recent studies.

Table 2-3- Summary of flow and peak reduction reported in the literature for bio-retention systems.

| Reference | % Of reduction (Mean annual volume flow) | % Of reduction (Peak flow) | Storm/event |
|---|---|---|---|
| Yang et al., 2009 | 42% | 70% | 24-hr 180 mm |
| Hatt et al., (2009) | 33% | 80% | Real storms |
| Trowsdale and Simcock (2011) | 55% | - | - |
| Lucke & Nichols (2015) | 61% | 79-93% | 30-min 39% AEP event |
| Hoban, 2017 | 59% | - | Real storms |

A vegetated swale is a grass-lined channel with a flat bottom which receives stormwater runoff flow via its slope's sides (Davis et al., 2012). These shallow channels have shown to be effective in stormwater quantity management. In two separate studies, around 47% and 45% reduction of mean annual runoff volume was reported by adopting swales (Ackerman and Stein 2008; Barrett 2005). A medium density residential development with an area of 32ha was used as a case study area in Melbourne by Lloyd al. 2002 for stormwater runoff flow generation and timing observation. The roofs and roads runoff were collected by swales feeding into wetlands. Comparing the generated runoff volume in the development with a paired catchment with a traditional stormwater pipe system, it was understood that the residential development with swales and wetlands produced 50 to 100% less runoff volume. Besides, the peak discharge consistently was lower for the development with swales and wetlands. It was also observed that the generated stormwater was delayed by at least 10 minutes at the area with WSUD technologies.

Rainwater tanks have been mentioned as an ancient practice in water management, although they have retained their popularity in the modern cities as well (Gomes et al., 2012). These systems collect and store rainwater to reuse as primary or alternative water sources (Fewkes 2012). Compared with other developed countries, Australia has significant rainwater tanks uptake. In the capital cities of Australia, households experienced an increase in the proportion of rainwater tanks installation during 2007 from 15% to 28% and 34% in 2010 and 2013 respectively (Australian Bureau of Statics 2013). These systems reduced the domestic stormwater runoff volume up to 48.1% in Western Australia (Zhang et al., 2010). Rainwater harvestings are important components of stormwater control, according to a study by Coombes et al. (2016) on distributed water management technologies. Additionally, they emphasize the significance of thorough cost-benefit evaluations that take into account environmental enhancements, infrastructural savings, and stormwater reductions. By using PCSWMM model, Walsh et al., 2014 reported that maximum long-term watershed volumetric reductions between 10.1% and 12.4% were observed for the period of analysis (1948–2011) with a range of rainwater harvest storage sizes (227 L to 7571 L). Coombes et al., 2016 stated that sustainable housing discussions often focus on water conservation, but rainwater harvesting and vegetable gardens also significantly reduce stormwater runoff and pollution, benefiting urban waterways. As per their research, rainwater harvesting alone led to substantial reductions in stormwater runoff and pollutants (TSS, TP, TN) across major cities in Australia. Greater reductions occurred when rainwater overflows were directed to vegetable gardens, significantly enhancing water quality and stormwater management. Total reductions across all cities (with gardens included), 53,501 ML/year less runoff, 69 tonnes/year less phosphorus (TP) and 572 tonnes/year less nitrogen (TN).

Permeable pavements have been reported as an effective tool to manage flooding risk as they allow infiltration and evaporation (Freni et al., 2010; Ullate et al., 2011; Beecham et al., 2012). They consist of a permeable pavement top layer and an underdrain layer. The permeability of the top layer can vary from tens to thousands of millimetres per hour (Bean et al., 2007; Kuang et al., 2011). Stormwater runoff can be drained over the top layer of the permeable pavement even with a fast rainfall with a 100 mm/hr intensity (Hsieh and Chen 2012).

Infiltration trenches can capture stormwater runoff via a filter media such as stone or gravel and infiltrate into surrounding soils and underlaying groundwater a (Siriwardene et al., 2007). In a study conducted by Yazdi and Scholz (2008), 73% and 80% reduction of mean flow volume and peak flow was observed by adopting infiltration systems.

#### 2.7.1. Observed performance of WSUD assets for managing stormwater quality

One of the first WSUD objectives is quality improvement of stormwater runoff from areas subject to urban development and hence, to protect or improve water quality in urban streams and in receiving waters (Wong, 2006). Urban development generates pollutants depending on contaminant processes and pathways which in turn, are influenced by land development and use as presented in Table 2-4 (Sharley et al., 2017).

Table 2-4- Sources of stormwater pollutants [Adapted from literature review].

| Contaminant source | Sediments | Nutrients | Micro-organisms | Oxygen demanding material | Metals | Oils | Synthetic Organics |
|---|---|---|---|---|---|---|---|
| Soil erosion | ✓ | ✓ | ✓ | ✓ | ✓ | | |
| Erosion of stockpiles | ✓ | | | | | | |
| Fertilisers | | ✓ | | | | | |
| Sewer overflows | | ✓ | ✓ | ✓ | | | ✓ |
| Animal waste | | ✓ | ✓ | ✓ | | | |
| Fuels | | ✓ | | ✓ | | ✓ | |
| Fuel combustion | ✓ | | | | ✓ | ✓ | |
| Vehicle wear | ✓ | ✓ | | | | | |
| Industrial chemicals | | | | ✓ | | | ✓ |
| Household chemicals | | | | ✓ | | | ✓ |
| Industrial processes | | ✓ | | ✓ | ✓ | | ✓ |
| Pesticides | | | | | | | ✓ |

The WSUD systems appropriate for water quality improvement include, bio-retention systems, vegetated swales, infiltration systems, and permeable pavements which can treat sediments, heavy metals, and nutrients in stormwater (Ahammed, 2017). The ability of these systems to remove pollutants and nutrients for a different range of storms is summarised in Table 2-5 (Henderson et al., 2007; Deletic & Fletcher, 2006; Beecham et al., 2012; Yang et al., 2009; Hunt et al., 2008; Ismail et al., 2010; Ahearn and Tveten (2008); Ackerman & Stein, 2008; Rusciano & Obropta, 2007; Purvis et al., 2019; Myers et al., 2009; Blecken et al., 2009; Sun & Davis 2007; Lan et al., 2010).

Table 2-5-The ability of bio-retention systems, permeable pavements, and vegetated swales to remove stormwater pollutant under experimental conditions.

| **Pollutant** | **Bio-retention** | **Vegetated Swale** | **Permeable pavements** |
|---|---|---|---|
| | **Removal (%)** | | |
| Phosphorus | 94 | 20-46 | 30-60 |
| Nitrogen | 66-77 | 15-69 | 20-60 |
| Total suspended solids | 95 | 6-96 | 28-100 |
| Metals | 85-95 | 60 | 7-60 |
| Faecal coliform | 94-99 | 50 | - |

Constructed wetlands are a common intervention to remove a diverse range of pollutants such as dissolved and particulate contaminants from different land use such as agricultural, industrial, and municipal sources. However, there is a need to further determine the basic elimination and transformation processes in wetlands so that wetlands can be designed to remove specific pollutants such as nutrient and heavy metals (Malaviya and Singh 2012). Wetlands may also offer only temporary storage of contaminant inputs rather than permanent removal and there is the potential for bioconcentration of pollutants based on the type of pollutants (Helfield and Diamond, 1997).

## 2.8. Stormwater Management Quality and Quantity Objectives

To select the best WSUD approaches, it is important to define the stormwater management objectives based on the research aims and the questions which will be answered in this study. For this study, the objectives are stormwater quality improvement and stormwater quantity management. Table 2-6 presents the stormwater management indicators and objectives (CSIRO 2006).

Table 2-6- Stormwater management objectives and indicators (CSIRO, 2006; The Best Practice Environmental Management Guidelines: Stormwater (Victoria Stormwater Committee, 1999); EPA, Victoria, 1739.1, 2021)

| Objective | Indicator | Target |
|---|---|---|
| Water Quality | Total suspended solids (TSS) | 80% retention of the typical urban annual load |
| | Total phosphorus (TP) | 45% retention of the typical urban annual load |
| | Total nitrogen (TN) | 45% retention of the typical urban annual load |
| | Litter | 70% retention of the typical urban annual load |
| Objective | Indicator | Target |
| Water quantity | Runoff volume and runoff peak | Maintain discharge for the 1.5-year ARI at predevelopment levels |

## 2.9. WSUD Cost

Cost of WSUD construction and implementation is significant. However, there is evidence that WSUD adoption can be more cost-effective than conventional stormwater management systems (McPhee, 2019). The significant cost associated with conventional stormwater management systems is relevant to their effort to reduce urban flooding (Visitacion et al., 2009). WSUD implementation can reduce the traditional conveyance network load and save significant cost (Roy et al., 2008). Upgrading existing stormwater infrastructures can be costly and disruptive (Ashley et al., 2011). Therefore, it is necessary to find and implement new economic ways to manage urban stormwater runoff.

Economic analysis of WSUD implementation has provided important information associated with their cost-effectiveness. WSUD cost was analysed in Lafayette, Indiana, for four neighbourhoods, and it was reported that the associated implementation costs reduced as the adoption of WSUD techniques increased in an area, and they found that the cost per cubic meter of runoff reduction varied from $3 to $600.

(Wright et al., 2016). WSUD assets were ranked based on their implementation costs from least to most expensive by Stovin and Swan (2007) as infiltration basins, soakaways, ponds, infiltration trenches and permeable pavements. Their analysis did not include land acquisition.

Capital and life cycle costs of some WSUD assets were assessed for 50 years by Uda et al., (2013). It was understood that bio-retentions, infiltration trenches, and vegetated swales were the least expensive techniques for implementation, considering the cost of assets. Permeable pavements and green roofs were the more and most expensive assets for implementation. WSUD implementation showed an increase in property values in some areas (Van Roon, 2005). However, some developers and property owners believe, it causes losing the potential for other uses of lands which was assigned to WSUD implementation (Roy et al., 2008). Reducing lot sizes and including more open space and swales reduced the sale price and cost of construction (Williams and Wise 2009). They found that the developments with WSUD techniques had a much higher ratio of sale price to construction costs during some of the research period while during other periods, the ratio was worse. They also reported that clustered development constantly performed better than traditional developments.

Cost of WSUD mentioned as an essential factor in their reliability in flood management (Karamouz and Nazif 2013). In a study by Jia et al., (2012), it was determined that WSUD assets which were optimised for being cost-effective to reduce runoff volume had smaller dimensions than the recommended size.

While environmental benefits from WSUD techniques are achieved after few years, their implementation costs happen at an early stage. Therefore, a life-cycle cost analysis can be a suitable approach for the comparison of the techniques with conventional stormwater networks (Wise et al., 2010). Life cycle costing is a “process to determine the sum of all expenses associated with a product or project, including acquisition, installation, operation, maintenance, refurbishment, discarding and disposal costs” (Standards Australia, 1999, p. 4). The LCC often provides essential input into the best stormwater system selection where stormwater management systems needed to be evaluated (Taylor 2005). Importance of accurate unit cost data was reported by Sample et al., 2003 developing a costing approach based on land sizes, and Houdeshel et al., (2011) developed a tool for a life-cycle cost analysis for several WSUD assets.

### 2.10. Constrains of WSUD implementation

Various guidelines and fact sheets have been developed in different countries by local, state and federal governments to describe WSUD techniques design processes and associated considerations. WSUD implementation requires a clear definition of desired performance and the project objectives that will be achieved through adopting the systems (Joint Steering Committee for Water Sensitive Cities (JSCWSC, 2009). However, the associated limitations and constraints for the implementation of the system should be identified as they can reduce the long-term success of systems. Despite the reported advantages and benefits of WSUD to promote urban areas liveability, some concerns were also reported in the literature. For example, in the case of rainwater tanks, common issues included, pump switch failures, plumbing issues and public health issues due to mosquitoes (Moglia et al., 2014, 2016b). Permeable pavements also need significant maintenance attention to retain their effectiveness (Erickson et al., 2013). Successful application of WSUD requires consideration of local site conditions, property types, climate condition, acceptance by the community and the project budget. Moreover, it is vital to overcome the impediments to WSUD implementation by collaboration between policymakers, planners, academia, and other stakeholders (Sharma et. al. 2012). Training events, capacity building programs, and demonstration projects are effective interventions to address institutional WSUD impediments (Brown and Farrelly 2009). The majority of written and published guidelines and fact sheets support WSUD implementation in new development areas; however, retrofitting WSUD technologies must also occur in existing urban areas in order to achieve significant change in existing water quality (Weber et. al. 2009). Table 2-7 presents various constraints in WSUD implementation identified by several researchers.

Table 2-7- Potential constraints in WSUD implementation based on the research literature review (Rashetnia et al., 2022)

| Constraints | Reference(s) |
|---|---|
| Technical issues (e.g., rainwater harvesting pump and switch failures) | Moglia et al., 2016b |
| 1- Clash with electricity services<br>2- Poor construction<br>3- Failure to manage the construction site which led more sediment, erosion, and scour<br>4- Public safety risks due to flow conveyance or standing water<br>5- Lack of funds for regular maintenance | (JSCWSC) 2009 |
| Public health issue (e.g., mosquitoes in rainwater tanks) | Sharma et al., 2012; Moglia et al., 2016a |
| Limited funding | (JSCWSC) 2009; Brown and Farrelly 2009; Shepherd 2008; Ahiablame et al., 2012 |
| 1- Absence of Policy level commitment<br>2- Inadequate selection and acquisition of adequate soil media<br>3- Underground services and terrain | Myers et al., 2013 |
| 1- Lack of design experience<br>2- Lack of design standards | Matamala & Jayasuriya (2007), Roy et al., 2008; Beecham 2010; Sharma et al., 2012 and 2016 |
| Challenging regulative framework | Gardiner and Hardy 2005; Shepherd 2008; Brown and Farrelly 2009; Sharma et al., 2012 |
| Lack of governance structure | Gardiner and Hardy 2005; Roy et al., 2008; Shepherd 2008; Brown and Farrelly 2009; Lee et al., 2010 |
| Life cycle cost uncertainty | Gardiner and Hardy 2005; Roy et al., 2008; Brown and Farrelly 2009; Beecham 2010 |
| Uncertainty in community acceptance | Roy et al., 2008; Shepherd 2008; Beecham 2010; Sharma et al., 2012 |
| Costs, systems operation, and management uncertainty | Gardiner and Hardy 2005; Beecham 2010; Sharma et al., 2012; McAuley & Knights 2015. |
| 1- Local regulations<br>2- Available space | Ashley et al., (2013) |

| | |
|---|---|
| 3- Public acceptance | |
| Absence of appropriate modelling tools | Shepherd 2008; Beecham 2010 |
| Inadequate data management | Gardiner and Hardy 2005 |
| Absence of suitable urban water management targets | Shepherd 2008 |
| Absence of suitable marketing mechanism | Gardiner and Hardy 2005 |
| Lack of educating relevant staff for maintenance | (JSCWSC) 2009. |
| Lack of space in developed areas | Walsh et al., 2012; Rashetnia et al., 2018 |
| 1- Promotion absence from social communities and governments,<br>2- Insufficient governmental incentive programs | Ahiablame et al., 2012 |
| Slow realization by all agencies and stakeholders | Weber et al., 2009 |
| Lack of post-implementation monitoring and performance assessment | Sharma et al., 2016 |
| Water quality improvement and enhancement of aquatic habitat | Helfield and Diamond (1997), Martin (1988), |

## 2.11. Computer Modelling of LID and WSUD techniques

An early review by Elliott and Trowsdale (2007) delved into the potential of modelling to assess WSUD techniques. However, at that time, it was observed that existing models had limitations in representing contaminants related to water quality, and there were difficulties in integrating hydrologic models with external processes such as toxicity and habitat models. Since the publication of that review, advancements have been made in narrowing the gaps in model capabilities. A substantial body of literature now exists, presenting monitoring information that encompasses the beneficial uses of WSUD controls. While monitoring methods are constrained by specific conditions and periods due to associated costs, simulation modelling offers a valuable approach for determining spatial and temporal information across various scales (Ahiablame et al., 2012).

In general, water quantity is more frequently modelled than water quality, primarily because obtaining the necessary water quality data for model calibration is more challenging than acquiring hydrological data (Imteaz et al., 2013). A review of urban stormwater quality models conducted by Obropta and Kardos (2007) compared deterministic, stochastic, and hybrid modelling approaches, suggesting that hybrid approaches could potentially minimize prediction errors and uncertainties.

Bosley II (2008) conducted a comprehensive review of hydraulic/hydrology odels such as ANSWERS, CASC2D, DR3M, HEC-HMS, HSPF, KINEROS2, and SWMM. Additionally, Zhou (2014) examined modelling and concluded that open-source models can be challenging to use and often lack user support, while proprietary models provide greater support but can be prohibitively expensive for many potential users.

The integration of GIS (Geographic Information System) streamlines the data processing required for input into models, as highlighted by Viavattene et al., (2010) and Viavattene et al., (2008). Commercial software, such as PCSWMM, incorporates WSUD modelling and GIS integration. A GIS interface can assist users familiar with GIS in overcoming some of the technical complexities associated with many current models. Bacchin et al., (2014) introduced a tool that integrates ArcGIS and SWMM to analyse the spatial composition and configuration of urbanized areas. Some non-proprietary models like HEC-HMS (Scharffenberg, 2013) and L-THIA (Park et al., 2013) now offer GIS extensions.

There is limited literature that has quantified the impacts of WSUD at a watershed scale (Ahiablame et al., 2013). This is crucial for advancing the use of models, as results can be simulated from a lot-scale to a watershed scale and across various temporal scales, while applying field studies at larger scales can be impractical (Ahiablame et al., 2012).

The use of multiple models to assess the impact of stormwater management alternatives is common. Sharma et al., (2008) employed three models—Aquacycle for the urban water balance, PURRS for peak stormwater flows from properties, and Model for Urban Stormwater Improvement Conceptualization (MUSIC) for stormwater flows, contaminants, and treatment options—to analyse stormwater management options in Canberra, Australia. Damodaram et al., (2010) utilized HEC-HMS as a hydrological model and SWMM to compute hydraulic routing for a study on the campus of Texas A&M University. MIKE SHE has also been employed to evaluate hydrological impacts of urbanization and study potential benefits resulting from implementing WSUD methods (Trinh and Chui, 2013).

Researchers may opt to develop their own models as an alternative method to assess the performance of LIDs and WSUD. Chen et al., (2016) created a novel computer model called

Rainwater+, which utilizes the Natural Resource Conservation Service (NRCS) Curve Number method to compute runoff depth. Rainwater+ serves as an intuitive and interactive tool for early design processes, aiding in decision-making, design evaluation, rough cost estimation, and compliance checking. StormWISE, a model developed by McGarity (2011), functions as a screening method for optimizing improvements to water quality.

Despite the availability of various popular and widely used software dedicated to urban hydrology and stormwater drainage, such as HEC-HMS, SWMM, MOUSE, and MUSIC, rainfall-runoff modelling still requires further research (Fletcher et al., 2013). Multiple studies demonstrate various techniques to analyse the effectiveness of WSUD techniques in stormwater management (Ahiablame et al., 2012). Model sensitivity analysis, uncertainty analysis, and calibration are crucial aspects in determining the strength and accuracy of the results produced by the model (Fletcher et al., 2013). These issues collectively impact the modelling of WSUD techniques with hydrologic models. Section 7.2 in chapter seven reviewed some ways WSUD techniques have been simulated and the capabilities of hydrologic PCSWMM model to demonstrate the techniques.

### 2.11.1. Selection of Stormwater Modelling Tools for this Study

Hydrological, hydraulic and stormwater quality modelling tools were considered essential for this research. Literature review, research aims, case study characteristics and discussions with water industry professionals were the main drivers to select suitable modelling tools. Furthermore, Coombes & Bonacci Water (2012) supported the integration of multiple modelling approaches to enhance the reliability and practicality of water management solutions.

Lerer et al., (2015), reviewed 24 WSUD decision tools and stated that the tools differ in terms of the type of questions they answer and suggested the variability among tools depends on the local condition. Based on their suggestions, study aims, local condition and the questions which will be answered through this study, the MUSIC and the PCSWMM models were selected for the water quantity and quality modelling. MUSIC is a powerful hydrological and water quality assessment tool to model both simple and complex urban developments for stormwater management quality and quantity using a few WSUD techniques. MUSIC is widely adopted as the preferred stormwater quality model across much of Australia. In terms of decision-making on WSUD techniques and predict their performance to improve the stormwater quality and quantity, the MUSIC model was used to determine appropriate system in order to achieve target pollution reduction levels at a conceptual level in this study.

PCSWMM model or SWMM model was first developed by the USA Environmental Protection Agency (USEPA) in 1971 and has been accepted and used across the world as a tool for hydrological and hydraulic processes modelling of a catchment since then and it has been upgraded at various intervals. The model is an open-source model which often has used in design, analysis and planning of urban water systems (Rossman, 2015). Routing component of SWMM simulates a pipe network, storage, channels and is able to transport runoff underground and overland. Assigning a simulation period and under various time steps, the model can determine sub-catchments flow depth and rate as well as stormwater pipes water quality (James et al., 2010).

The PCSWMM model, which is a combination of SWMM 5, and Geographical Information System (GIS) which can provide a complete package for 1D and 2D rainfall- runoff processes analysis. Besides, PCSWMM can model stormwater source control technologies to manage stormwater quality and quantity (James et al., 2010).

The PCSWMM model was used for both allotment and sub-catchment scale modelling runs and optimization, while the MUSIC model was only utilized for WSUD effectiveness on stormwater quantity at an allotment and sub-catchment scale due to its limited use of the Muskingum-Cunge routing approach and only consideration of travel time for the pipe's network.

### 2.12. Optimization

There are several variables to consider when adopting WSUD techniques in an urban area. There are numerous potential combinations of techniques, as well as the number of them, size, and where each technique should be placed in a sub – catchment to provide optimal benefit. The hydrologic parameters of an urban catchment can vary dramatically, which causes WSUD measures to perform differently depending on where they are implemented. With stormwater management expenditures constantly being a challenge, obtaining the maximum efficiency from WSUD techniques at the lowest cost is crucial. Another important task is to find the most cost-effective solutions. As a result, the purpose of optimization in this study was to find the most advantageous combinations of WSUD techniques at various cost levels so that decision makers can choose the solution that best suits their requirements and budget.

### 2.12.1. Multi-Objective Optimisation Approach

The single-objective optimization aims to determine the maximum and minimum values for a specific objective. Linear programming, stochastic hill climbing, and gradient searches are examples of single-objective optimization approaches (Zhang, 2010). Caramia and Dell'Olmo (2008) define a fundamental single objective problem as follows:

$$Min\, f \quad x \epsilon\, S, \tag{2-1}$$

where $f$ is a scalar function and S is a set of constraints defined as

$$= \{x\ \epsilon\ R^m : h(x) = 0, g(x)\ \gg 0\} \tag{2- 2}$$

Multi-objective optimisation seeks to optimise several objectives at the same time. Multi-objective optimisation can be stated mathematically as:

$$\min\ [f_1(x), f_2(x), \dots, f_n(x)] \qquad\qquad x\epsilon\ S, \qquad\qquad (2-3)$$

S is a set of constraints and n >1. The objective space is the space that the objective vector belongs, and the image of the feasible set under F which is known as the attained set (Caramia and Dell'Olmo, 2008).

$$C = \{y\ \epsilon\ R^n\ ;\ y = f(x), x\epsilon S\} \tag{2-4}$$

In multi-objective optimisation problems, there is usually not one best solution; instead, there is a set of alternatives known as the Pareto optimum set. A vector $x^* \in S$ is said to be the Pareto optimal If all other vectors $x \in S$ have a greater value for at least one objective function, $fi$, where $i$=1…, $n$, or have the same value for all objective functions in the multi-objective problem then (Caramia and Dell'Olmo, 2008). Basically, if a vector u is said to "pareto dominate" another vector v, then all values of u are less than or equal to their corresponding values of v and at least one component of u is strictly less than the corresponding component of v. Because the objectives are being decreased, multi-objective solutions with lower values are considered dominant (Eckart, 2015). The Pareto front or Pareto surface is the image of all efficient solutions (Caramia and Dell'Olmo, 2008). The shape of the trade-offs between objective functions is determined by the Pareto front, which is defined by the Pareto optimal set (Caramia and Dell'Olmo, 2008; Hadka and Reed, 2013). For a given multi-objective issue, the Pareto optimum set P can be defined as

$$\mathcal{P}^* = \{x \in \wedge | \neg\exists x' \in \Lambda, F(x') \prec F(x)\} \quad (2\text{-}5)$$

Meaning that options "x" are part of the Pareto optimum set, and that no solution from the set of possible solutions would dominate "x." (Hadka and Reed, 2013). The Pareto front is the representation of the Pareto optimum set. Caramia and Dell'Olmo (2008) used the shape of the Pareto front to illustrate the trade-offs of objective functions. For a Pareto optimum collection, Hadka and Reed (2013) define the Pareto front.

$\mathcal{P}^*$as

$$\mathcal{P}\ \mathcal{F}^* = \{F(x) |\ x \in \mathcal{P}^*\} \quad (2\text{-}6)$$

Tabu searches, simulated annealing, scatter searches, and genetic or evolutionary algorithms are all methods for tackling multi-objective optimisation problems. A genetic algorithm is a sort of evolutionary algorithm that has advantages over other techniques and is often used in multi-objective water resource problems (Sivanandam and Deepa, 2007; U.S EPA, 2006). Genetic algorithms (GA) employ a population-based method in which multiple solutions participate in each iteration, resulting in the evolution of a new population of solutions.
Genetic algorithms have been found to be more effective than standard techniques like gradient searches and linear programming in solving multi-objective optimisation issues (Fonseca and Fleming, 1995). They are popular in addressing multi-objective optimisation problems because they do not require derivative knowledge, are very straightforward, and can be applied to a wide range of situations (Deb, 2011). Because all objectives are relevant in a multi-objective optimisation issue, the principle of identifying the best solution cannot be applied to just one.

Different solutions frequently result in trade-offs between different goals. A solution that works well for one goal almost always implies compromise on other goals (Deb, 2011). This limits the capacity to select a solution that is optimal in only one objective, resulting in the multi-objective optimisation goals of determining a solution set that is on the Pareto front and a solution set that is diverse enough to reflect the entire Pareto front (Deb, 2011).

Genetic algorithms (GA) are stochastic search techniques that mimic the process of natural selection, which offers new random solutions to retain and increase desirable qualities of a solution set (U.S. EPA, 2006). Because they process a set of solutions in parallel and take advantage of similarities through crossover, genetic algorithms (GA) are very useful for solving large optimization problems (Zhang, 2009; Zitzler and Thiele, 1999). Solutions are produced in each new generation by choosing people according to their fitness, and then a second generation of solutions is produced by using the crossover and mutation operators. After then, the crossover and mutation operators are used to generate a new generation of solutions. Similar to natural adaptation, this process is done again and over, evolving the' population of solutions and corresponding level of fitness to the challenge (Eshelman, 1991).

Additionally, genetic algorithms are a popular multi-objective optimization technique that is used to investigate low-impact advancements. A number of research, including SWMM, Baek et al. (2015), Duan et al. (2016), Eckart et al. (2018), Jung et al. (2016), Karamouz and Nazif (2013), have used Gas in conjunction with simulation models to run simulation-optimization models that are able to accomplish multiple objectives.

### 2.12.2. Optimisation in WSUD Design

In recent years, several studies have been conducted to optimize WSUD techniques for stormwater quality and quantity management. WSUD component selection, sizing, and placement are all components to consider. Table 2-8 contains a list of some of the studies.

Table 2-8- List of Recent Studies Adopted Several Approach for WSUD Techniques Optimization

| **Authors** | **Single or Multi-objective** | **Objectives** | **Purposes** | **Selection, Sizing, or Location of BMPs** | **Devices Considered** | **Optimization Technique Used** | **Simulation Model Used** |
|---|---|---|---|---|---|---|---|
| Jayasooriya et al., (2016) | Single | Cost | Water quality | Sizing | Sediment basins and Bioretention | Trial and error | MUSIC |
| Loaiciga et al., (2015) | Single | Cost | Retention, water quality | Sizing | Detention storages | Linear programming | Incorporated in LP |
| Loaiciga et al., (2015) | Single | Cost | Retention, water quality | Selection | Catchment basins | Integer linear programming | Incorporated in ILP |
| Lee et al., (2012) | )Single | Cost | Retention, water quality | Selection and location | Pervious buffers, Bio-retention and porous pavers | Scatter search, NSGA II | SWMM and purpose written model |
| Reichold et al., (2010) | Single | Flow regime change | Flow regime | Location | Impervious area | Genetic algorithm plus local search | SWMM, Ecological impact model |
| Zhen et al., (2004) | Single | Cost | Water quality | Sizing and location | Stormwater ponds | Scatter Search | AnnAGNPS |
| Di Matteo et al., (2017b) | Multi, multiple problem formulations | Cost, water quality (TN), harvesting capacity, amenity | Water quality, stormwater har-vesting, amenity | Project selection | Biofilters, wetlands, s w a l e s | Pareto Ant Colony Optimization Algorithm (PACOA) | MUSIC |
| Di Matteo et al., (2017a) | Multi | Cost, *TSS*, and volumetric red liability | Water quality, stormwater harvesting | Selection, sizing, and location | Biofilters, wetlands, sediment basins, ponds | NSGA II | MUSIC |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Marchi et al., (2016) | Multi | *NPV* (Average and Range) | Stormwater harvesting | Sizing | Sedimentation basins and wetlands | NSGA II | WaterCress |
| Chichakly et al., (2013) | Multi | Cost, water quality (*TSC* | Water quality | Sizing and location (within subbasins) | Bioretention, detention pond | Differential Evolution | Hydrological Simulation Program— Fortran (HSPF), Sensitivity Analysis |
| Zare et al., (2012) | Multi | Cost, water quality (*TSS* and BOD) and total SW runoff | Flooding, retention, water quality | Sizing and location (within subbasins) | Bioretention, porous pavers, barrels (rain tanks), land use | NSGA II | SWMM |
| Lee et al., (2012) | Multi | Cost, flow volume, and pollutant load | Retention, water quality | Selection and location | Pervious buffers, Bio-retention, and porous pavers | Scatter search, NSGA II | SWMM and purpose written model |
| Lee et al., (2005) | Multi | Cost, volume retained, and pollutant retained | Retention, water quality | Selection and sizing | Detention ponds and on-site, wet weather controls | Premium Solver in Excel (Evolutionary solver, NLP) | Spreadsheet based on STORM |

The listed studied in the table have been classified according to the following studies and criteria:

1. Optimisation approach (e.g., single objective or multi-objective)
2. Optimisation objective(s) (e.g., cost, water quality, reliability)
3. Stormwater management target including reduction in flooding, flow retention, water quality improvement, and stormwater harvesting.
4. Optimisation aimed at selection, sizing, or determining the location of WSUD techniques
5. Considered WSUD techniques.

The single-objective studies often focused on cost reductions, subject to reaching limits on the minimum percentage of pollutants removed and/or the minimum amount of stormwater maintained on-site. Stormwater harvesting benefits are considered in the objective function of Marchi et al.,

(2016) as a reduction in the cost of the alternative mains water. Furthermore, given net present value (NPV) is determined over a number of climate change scenarios, the problem is solved using a multi-objective algorithm with two formal objectives corresponding to the average and range of NPV over these scenarios.

The multi-objective studies produce a Pareto optimal front, which consists of a set of solutions that represent the best trade-offs between cost and other objectives, such as water quality improvement, on-site stormwater retention, and stormwater harvesting.

Water quality enhancement is frequently one of the goals of stormwater systems, with flow retention being a major consideration in most studies. Stormwater harvesting is only mentioned in a few research (Marchi et al., 2016; Di Matteo et al., 2017a, b; Di Matteo et al., 2018). Only one study (Zare et al., 2012) directly considers the reduction in flood peak. The predicted reduction in flood damages is incorporated in the objective function in this study.

In most of the research, the sizes of WSUD techniques are used as decision variables, with selection and location also being evaluated in a few. Buffer strips, detention storages, sedimentation basins, and biofilters are among the stormwater techniques that have been investigated. Reichold et al., (2010) takes an alternative approach, considering the percentages of impervious area in all sub catchments as choice variables. These numbers were chosen to minimize the change in the natural flow regime, which is defined by 33 hydrological parameters.

Various optimization strategies are applied in the various research. The most use EA (NSGA II, genetic algorithm, differential evolution, PACOA), which is an optimization technique that interacts with a simulation model. Only one study (Loaiciga et al., 2015) applied linear and integer linear programming techniques (in which some of the decision variables can only take integer values). In this example, the model is expressly written to contain equations representing stormwater device simulation, and no separate simulation model is utilized.

No formal optimization is utilized in the study by Jayasooriya et al., (2016) since the search space is small enough to allow trial-and-error. SWMM (Rossman, 2015), MUSIC (eWater, 2011a), WaterCress (Clark et al., 2002), HSPF (Bicknell et al., 2001), and AnnAGNPS were among the simulation models utilized (AGNPS, 2001). This study explored several planning and optimisation tools like UrbanBEATS and Optimatics to assess their capabilities for optimization considering this research objectives. However, this study used GreenPlan-IT tool for optimisation considering the tool capabilities (i.e., being linked to PCSWMM and the research objectives) (refer to section 8.2).

## 2.13. Conclusion

Rapid population growth, urbanization, and climate change are key drivers necessitating the development of innovative stormwater management strategies. Over the past two decades, urban stormwater management has evolved with the adoption of WSUD techniques. While WSUD techniques offer numerous benefits, their effectiveness is highly dependent on site-specific conditions and spatial placement. Reviewing existing studies indicated that WSUD approaches are particularly effective in mitigating hydrological impacts during storm events with shorter return periods. When integrated with conventional stormwater infrastructure, they also enhance performance in managing larger storm events.

The majority of existing studies have primarily focused on the benefits of WSUD implementation in greenfield developments. However, a review of the literature conducted in this study has identified a significant knowledge gap in understanding the most effective implementation strategies for existing urban development urbanization. The extent to which WSUD can enhance stormwater quality and quantity management in established urban areas remains uncertain and requires further investigation.

Listed from the identified gaps in section 1.1, this study explored the role of WSUD approaches in managing stormwater under land-use and climate change conditions in existing urban catchment. Specifically, it investigates the effectiveness of distributed WSUD implementation, which involves strategically placing WSUD techniques within smaller lots and available urban spaces. Unlike greenfield developments, WSUD implementation in existing urban areas is subject to various constraints and challenges. This research identified these constraints and evaluated the factors influencing the selection of the most appropriate WSUD approaches for implementation in established developments. Next chapter has explained the reserch steps and details of each step.

# 3. Chapter 3: Research Methodology

In Australia, WSUD techniques have focused not only on improving water quality but also on managing water quantity by integrating storage solutions such as infiltration trenches, rainwater tanks, sedimentation basins, and wetlands. These approaches aim to restore hydrological processes and maintain flow regimes closer to natural conditions. Although research on WSUD has been ongoing for some time, there are still information gaps in terms of system implementation, benefits, and efficacy. This study investigated the most suitable locations and types of WSUD techniques for managing stormwater quantity and quality in existing urban development, taking into account both land use change and climate change. In this study, distributed WSUD techniques were explored. Compared to greenfield developments, implementing WSUD in existing developments (suburbs) has extra limits and barriers.

Aside from a literature review, a local survey was undertaken for this project to get feedback from local councils on limitations and barriers in the implementation of WSUD approaches in urbanised catchments, and the results are published in a conference paper (Hydrology and Water Resource Symposium 2018). As a result, this study highlighted limits and obstacles to WSUD assets that influence the selection of the most suited WSUD techniques for implementation in existing developments. This chapter mapped the research study steps and methodology in details.

The research methodology flowchart is presented in Figure 3-1. Steps taken for the research explained in detail in following sections.

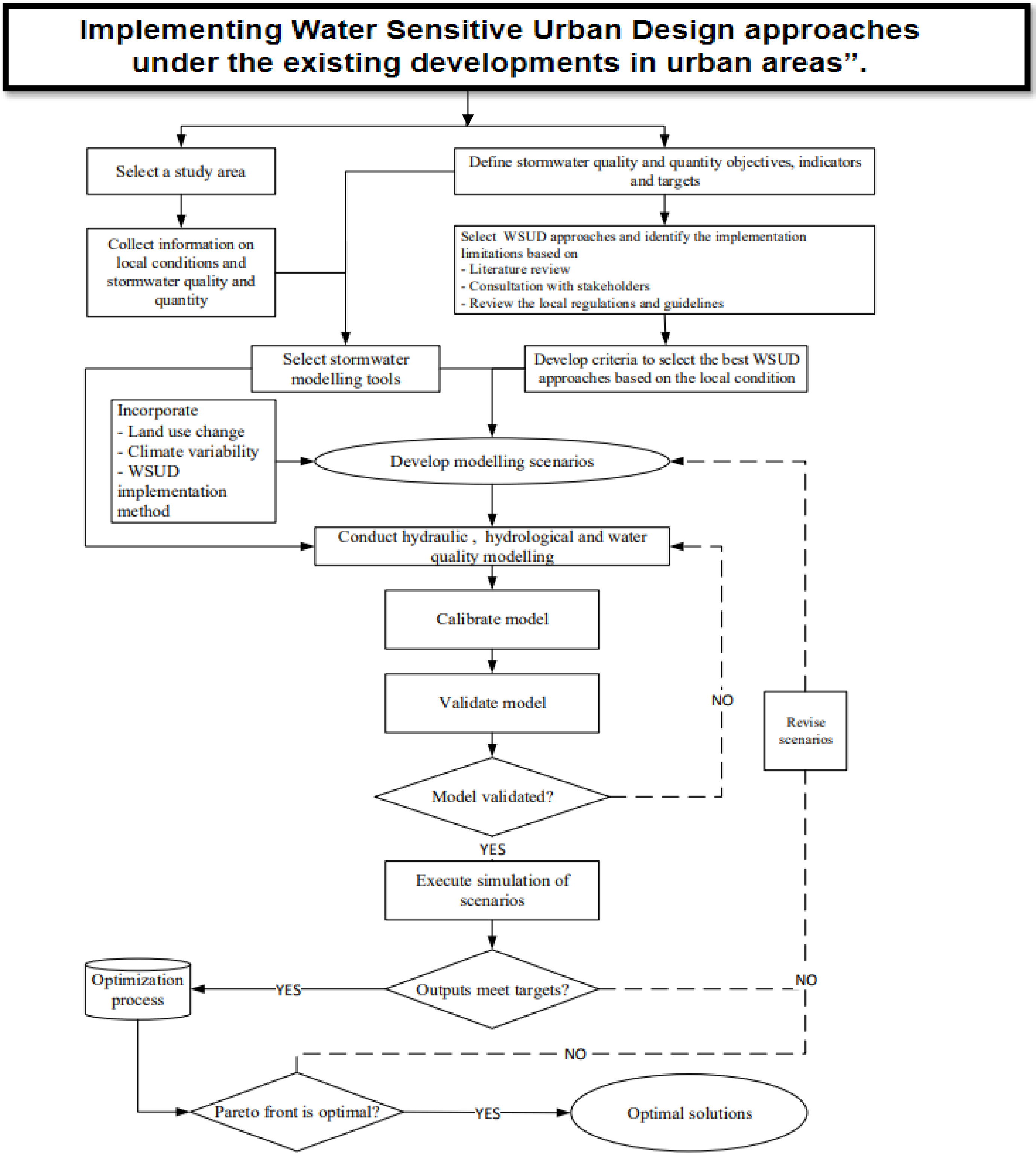



*Figure 3-1-Illustration of the research methodology*

### 3.1. Step One – Understand Research's Gaps in WSUD Area

The first and most important step in any research is a thorough and in-depth review of the literature. A comprehensive review of existing studies was essential to establish the current state of knowledge, identify key gaps, and justify the need for further investigation in this study. This task was carried out to systematically assess research on WSUD techniques, specifically to highlight gaps in their application and it was found that there is a knowledge gap in understanding the technique's effectiveness at different scales on stormwater quality and quantity management under urban catchments existing condition (refer to Sections 1.1 and 1.2).

Despite ongoing advancements in WSUD research, significant knowledge gaps remain regarding the technique's implementation at different spatial scales, their effectiveness under climate change, and their overall advantages and efficacy in highly urbanized catchments. Additionally, it was understood, while numerous studies have examined WSUD techniques in isolated scenarios, there is limited research on their large-scale integration within densely developed urban environments using actual data. Therefore, this research focused on evaluating the most common techniques effectiveness (refer to step 3.4) at a developed urban catchment outlet while considering key factors such as existing land use, ongoing urbanization, and projected climate change impacts by 2050. Given the rapid growth of urban areas and the increasing challenges associated with stormwater management, understanding how WSUD interventions perform under both current and future conditions was crucial.

To provide a robust assessment, stormwater quality and quantity modelling was conducted for the existing conditions in the study area. The results were then compared with projections that incorporate anticipated changes in land use and climate. This comparative analysis enables a deeper understanding of how WSUD strategies can be optimized to enhance urban resilience against future hydrological and environmental challenges. Ultimately, these insights will contribute to more effective policy development, infrastructure planning, and sustainable urban water management practices.

### 3.2. Step Two - Study Area Selection

Selecting an appropriate research area was next, after a literature review and identification of a suitable gap to be addressed. To select a well-established urban catchment with readily available stormwater data to be simulated for the current development and future condition, an extensive search was conducted in Melbourne. After contacting several local authorities to seek their support, an urban area in Melbourne's west was selected (refer to section 1.4).

### 3.3. Step Three - Stormwater Quality and Quantity Objective Definitions

Uncontrolled urban stormwater run-off poses a risk to the values of waterways and bays. Several organizations have developed guidelines that acknowledge the state of the science and the potential harm that urban stormwater flows might cause, with the goal of improving Victoria's management of this resource. It encourages using best practices in the environment to reduce the risk of harm to people's health and the environment, and it offers data to help with the planning and implementation of new urban stormwater management systems. To understand how the defined objectives can be used as a target for this research, several publications were evaluated, including CSIRO, 2006; The Best Practice Environmental Management Guidelines: Stormwater (Victoria Stormwater Committee, 1999); and EPA, Victoria, 1739.1, 2021. The selected objectives for the quality and quantity aspects are shown in a table in Section 2.8 of this thesis.

This was done to find out if, the best solutions and developed scenarios could meet the specified and chosen stormwater quality and quantity objectives, given the existing study area's climate, land use, and future development situations,

### 3.4. Step Four - WSUD Choosing for this Research

Understanding the constraints and factors related to the implementation of WSUD techniques in the existing urban catchments was one of the critical aspects of this research. As a result, a list of WSUD techniques were developed using an existing local factsheets and guidelines as well as a review of the literature. Melbourne's local government stormwater engineers were contacted and asked to list the limitations of each technique according to their knowledge and rank the most preferred techniques implemented in their municipality. The advantages and disadvantages of the systems were discussed with experts and based on undertaken analysis on the collected data and the results of this task, the most widely used WSUD techniques were chosen for storm water modelling and scenario development for this study at different scales. The study initially considered a range of WSUD techniques at the allotment scale, including rainwater tanks, raingardens, permeable pavements, infiltration trenches, and vegetated swales. Following a detailed analysis of their effectiveness in managing stormwater quantity at a typical allotment within the study area, only raingardens, infiltration trenches, and permeable pavements were selected for optimization at the whole catchment scale for quality and quantity aspects. Rainwater tanks and swales were excluded due to their limited effectiveness in controlling stormwater quantity at both the allotment and catchment levels. However, given the widespread adoption of rainwater tanks in Melbourne and to compare with available research, their performance was further assessed at both the sub-catchment and catchment scales to explore their overall contribution to stormwater quantity management (chapter 6). Chapter four of the thesis has covered the WSUD survey developed and the findings of this task for this study. Figure 3-2 summarised the selected WSUD techniques and their assessment at different scale and the steps they were examined in this study.

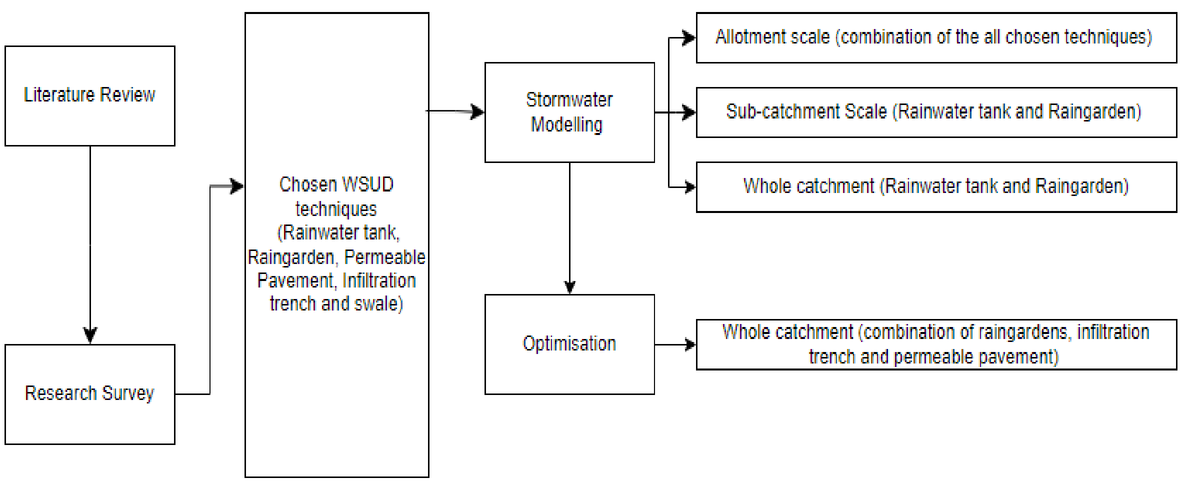



*Figure 3-2- Illustration of the chosen WSUD techniques for stormwater modelling and optimization*

### 3.5. Step Five - Stormwater Quality and Quantity Modelling Tools Selection

Two hydrological and stormwater quality software packages PCSWMM and MUSIC were selected for this research. The selection of suitable modelling tools was justified through a comprehensive process that included a thorough literature review, alignment with research aims, assessment of model capabilities to accurately simulate the study area, and the need for integration with an optimization tool. Additionally, discussions with water industry professionals provided valuable insights, ensuring the chosen models were appropriate for addressing the study's objectives. MUSIC is a powerful tool to model both simple and highly complex urban stormwater systems using WSUD approaches. MUSIC is widely adopted as the preferred stormwater quality modelling tool across much of Australia. In terms of decision-making on WSUD approaches and predict their performance to improve the stormwater quality and quantity, the MUSIC model was used to determine appropriate system to achieve target pollution reduction levels at a conceptual level in this study. Storm Water Management Model (SWMM) which is developed by The United State Environmental Protection Agency (USEPA) is a program that computes and simulates dynamic rainfall-runoff for a single event and long-term (continuous) runoff quantity and tracks the depth of stormwater as well as the flow rate and peaks. The software also simulates different WSUD systems (Chaosakul et al. 2013). Model's further explanations and capabilities have been covered in chapter five. The PCSWMM model was used for both allotment and sub-catchment scale modelling runs and optimization, while the MUSIC model was only utilized for WSUD effectiveness on stormwater quantity at an allotment and sub-catchment scale due to its limited use of the Muskingum-Cunge routing approach and only consideration of travel time for the pipe's network. In addition, the MUSIC was used for the study area soil's adjustment (refer to Table 6-15).

There is insufficient stormwater quality and quantity data available to study the effects of WSUD adoption in developed areas. For this reason, it is necessary to produce such data for additional study. A Melbourne Water-funded monitoring program carried out in the west Melbourne research study area to calibrate the stormwater modelling tool (i.e., PCSWMM model) (a commercial version of SWMM - https://www.pcswmm.com/) selected for this study, the aim was to collect stormwater quality and quantity data over a year. Stormwater data collection, challenges, analysis, as well as PCSWMM model calibration is covered in chapter five.

### 3.6. Step Six – Stormwater Modelling Scenario Developments

Selected WSUD techniques and establishing research objectives followed the execution of stormwater modelling scenarios. Table 3-1 summarised the scenarios developed for this study:

Table 3-1-Examined scenarios for stormwater modelling in this research

| No. | Scale | Scenario | Explanation |
|---|---|---|---|
| 1 | Typical allotment of the study area | To explore the impact of chosen WSUD techniques (Rainwater tanks, raingardens, permeable pavements, infiltration trenches, and swales) on a typical allotment stormwater quantity (peak flow size and mean annual flow reduction)- | MUSIC and PCSWMM Models didn't need to be calibrated for this task as the pervious and impervious fraction ratio were obtained from a typical dwelling in the study area. This task findings and results are presented in chapter six. |
| 2 | One sub-catchment | To explore the impact of most adopted techniques in an urban catchment (i.e., rainwater tank and raingardens) on the stormwater quantity (Runoff volume and peak flow size). | MUSIC and PCSWMM Models were used and calibrated using literature review methodology covered in chapter six and then validated later based on the collected data for the study area |
| 3 | Whole catchment (the study area) | To explore the impact of most adopted techniques in urban catchment (i.e., rainwater tank and raingardens) on the stormwater quantity (Runoff volume and peak flow size) | MUSIC and PCSWMM Models were used and calibrated using literature review methodology covered in chapter six and then validated later based on the collected data for the study area. |

Stormwater modelling was conducted using two chosen stormwater models before the optimization work to understand the impacts and system performance at various scales. Additionally, the scenarios for the current and future development conditions only included changes in land use change. Climate change, however, was disregarded at this point because the techniques had shown that they could achieve the maximum functionality and reduction on stormwater quantity at a small scale under land use change only.

### 3.7. Step Seven – Optimisation

The optimization process was initiated after it was understood how the chosen techniques functioned at various scales under land use change to manage stormwater quantity. Permeable pavement, infiltration trench, and bioretention techniques were included in the optimization process, considering the findings of the analysis of the systems in allotment and sub-catchment scales on peak flow size and stormwater runoff volume reduction, literature review, and common WSUD techniques in Australia's urban catchments. This task undertook after the PCSWMM model calibration and validation (refer to chapter five).

This study explored several planning and optimisation tools like UrbanBaets and Optimatics to assess their capabilities for optimization and meeting this research requirements. At last, the Green-Plan IT tool, which is developed by the San Francisco Estuary Institute, was utilized for the optimization task. This program aids planners and engineers in the cost-effective selection and placement of WSUD techniques in urban catchments.

The program was also run in the calibrated PCSWMM model for the whole study area. This assignment was completed with focus on both quantity and quality. In addition, both land use change and climate change scenario for future condition were adopted in optimisation process. The findings indicate that the difference in total relative cost between optimal and nonoptimal solutions can reach well below $20 million dollars, and the effectiveness of load reduction for both quality and quantity at the study catchment could vary as much as 33% to 40%. Table 3-2 summarised the scenarios ran for the optimisation task in this study:

Table 3-2 -Examined scenarios for optimization.

| Task | Scale | Scenario | Explanation |
|---|---|---|---|
| Stormwater Quantity | Whole catchment | Adopting allotment scale techniques (Raingardens, Infiltration trench and permeable pavements) | Optimisation run for a single and continuous one-year events under existing and future land use and climate change |
| | Whole catchment | Adopting Sub-catchment scale techniques (Raingardens, Infiltration trench and permeable pavements) | Optimisation run for a single and continuous one-year events under existing and future land use and climate change |
| Stormwater Quality | Whole catchment | Adopting allotment and sub-catchment scale techniques (Raingardens, Infiltration trench and permeable pavements) | Optimisation run for a single and continuous one-year events under existing and future land use and climate change |

# 4. Chapter 4: Associated Constraints to Implement WSUD Techniques Under Existing Condition.

## 4.1. Introduction

The primary benefits of Water Sensitive Urban Design (WSUD) include the enhancement of stormwater quality, the harvesting and reuse of stormwater for non-potable applications, the reduction of runoff volume and peak flow, and the protection and restoration of natural ecosystems such as streams, creeks, rivers, and wetlands within urban developments. (Argue 2004; Morgan 2013; Coutts et al., 2013; Duncan et al., 2014; Fletcher et al., 2014; Norton et al., 2015; Walsh et al., 2015; Liu et al., 2016; Li et al., 2016; Jamei and Rajagopalan 2017). Two fundamental philosophies are identified to guide WSUD (Lloyd 2001); Best Management Practice (BMP) and Best Planning Practice (BPP). BMP considers non-structural and structural elements of WSUD assets and BPP considers the site planning and assessment components for the asset's adoption. Integration of BMP and BPP is vital for sustainable management of WSUD (Ahammed 2017).

There are number of factors and considerations, which are helpful in selecting BMP approaches, to achieve design objectives for a given site, development, or a catchment. To plan a stormwater management scheme undertaking a site analysis and land capability assessment are the very first steps. In addition, it is important to understand the stormwater management objectives, targets, and indicators. Several guidelines and factsheets were developed over the last 20 years to address the most important criteria needed to be considered for WSUD best management practices (Argue 2004; Department of Environment, Western Australia 2004; CSIRO 2006; Healthy Waterways 2006; Australian Runoff Quality 2007; Department of Planning and Local Government, South Australia 2010; Melbourne Water 2013)

Table 4-1 presents the most important considerations for selecting WSUD assets based on the available guidelines and factsheets. Most recent studies have addressed and discussed WSUD adoption in new developments covering greenfield and infill areas (Balakrishna et al., 2006; Blecken et al., 2011; McMillan et al., 2012, Ahiablame et al., 2012; Brown et al., 2013; Fletcher et al., 2014; Walsh et al., 2015; Sargen 2015; Li et al., 2016; Bonneau et al., 2017). In general, there is a lack of information on WSUD implementation in existing urban developments, a knowledge gap that is addressed in this research and chapter.

Table 4-1- Important considerations for WSUD approaches selection

| | |
|---|---|
| **Site considerations** | Geology and soils |
| | Landforms |
| | Site Safety |
| | Current drainage patterns |
| | Climate condition |
| | Land use |
| | Significant natural features |
| | Existing urban infrastructure |
| | Historical/cultural features |
| **Stormwater management considerations** | Quality improvement |
| | Quantity management (flood and flow peak control; flow regime management) |
| | Sediment control |
| | Systems functionalities (i.e. if the systems are suitable for quality or quantity management) |

## 4.2. Understand WSUD techniques implementation limitations

One crucial component of this research was to understand the WSUD techniques associated limitations and considerations for their implementation under the existing urban developments. A survey was designed to compile a comprehensive list of the most commonly adopted WSUD techniques in Melbourne's urban areas, developed based on an extensive literature review and existing local guidelines and fact sheets (Table 4-2). This list was then linked to the corresponding implementation constraints and key considerations (Table 4-3). The identification of these constraints and considerations was initially informed by a thorough literature review and further refined through the survey process. To facilitate data collection, stormwater engineers from local government authorities in Melbourne were invited to participate. The study's methodology and objectives were explained to them, and they were asked to rank the identified limitations for each WSUD technique based on their professional expertise. Data was collected through in-person interviews, phone discussions, and self-administered survey responses.

This table was developed adopting the summary of Table 2-7 of this thesis and completed throughout a conducted survey in Melbourne among the local councils.

Table 4-2- List of the most suggested WSUD assets at different scales (*DPLG 2010)*

| No | WSUD asset – street and catchment scale | No | WSUD asset – lot scale |
|---|---|---|---|
| 1 | Sedimentation basins (SB) | 1 | Rainwater Tanks (RWT) |
| 2 | Buffer Strips (BS) | 2 | Trash racks, baskets and Booms (TB) |
| 3 | Bio-retention systems (aka raingardens) Bior | 3 | Rain Gardens (RG) |
| 4 | Swales (SW) | 4 | Roof systems (Green/ Blue roofs) (Roofsys) |
| 5 | Permeable Pavements (PPs) | 5 | Siphonic Roof Water Systems (Siph) |
| 6 | Infiltration systems (i.e. trenches, basins and wells) (IS) | 6 | Simple Downspout Disconnection (SDC) |
| 7 | Sand filters (SF) | 7 | Dry wells |
| 8 | Constructed Wetlands (CWs) | 8 | Geocellular/modular systems (G(Msys)) |
| 9 | Ponds and Lakes (P&L) | 9 | Soak pits/ Soakaways/ Wells (SP) |
| 10 | Gross Pollutant Traps (GPTs) | 10 | Leaky wells and infiltration trenches (LW) |

Table 4-3-List of considerations and limitations for WSUD implementation based on the literature (refer to Table 2-2 in section 2.5. of the thesis)

| No | Considerations/limitations | Description |
|---|---|---|
| 1 | Available Space | Is there enough space for the asset's adoption considering the type of WSUD as the urban areas developing? |
| 2 | Ease of access for construction and maintenance | Is that easy to access the assets considering traffics to deliver and maintain? |
| 3 | System Functionality | Is functionality of the assets being a limitation considering stormwater management aims (i.e. quality, quantity, harvesting…) |
| 4 | Life cycle cost and available funding | Is the total required cost and available funds a challenge for the assets for adoption and for ongoing maintenance? |
| 5 | System Operation | Is the assets operation and delivery a limitation for their applications? |
| 6 | Soil condition | Is the site soil condition a limitation considering the type of WSUD system? |
| 7 | Development layout | Is the assets suitable considering development layout or suitability to surrounding land use (e.g. thoroughfares, parking, shopping, industry, etc)? |
| 8 | Operation OH&S | Is the OH&S a challenge during construction or after completion? |
| 9 | Steep terrain | Is the site terrain/ slope a challenge for adoption? |
| 10 | Climate condition | Are the assets suitable for climate condition? |
| 11 | Land use characteristics | Are the assets suitable for residential, industrial or different type of land use? |
| 12 | Environmental performance | Will the assets affect the public health during and after operation and completion? |

## 4.3. Melbourne's Local councils' feedback and Results

Information received from sixteen local council in Melbourne. Their experts provided information and ranked the limitations for the WSUD assets they had experience with. The assets included: sedimentation basins, bio-retention systems (raingardens), infiltration systems, sand filters, constructed wetlands, ponds and lakes, gross pollutant traps and rainwater tanks. Roof systems

(Green/ Blue roofs), Siphonic Roof Water Systems, Simple Downspout Disconnection, Dry wells, Geocellular/modular systems, Soak pits, Soakaways, wells, leaky wells and infiltration trenches are highlighted as less adopted or less suitable assets for adoption in many areas of Melbourne considering the type of soil and climate condition, although they are more widely used in areas with sandy soils such as Western Australia. Some of the experts were of the view that a combination of considerations would impact on the selection of suitable asset(s).

### 4.3.1. Considerations in WSUD Implementation

WSUD implementation considerations/ limitations are discussed below as listed in Table 4-3 above and ranked as per the information provided:

1- Available space: more build-up development makes it harder to find space for some systems. Therefore, this limitation was ranked as the most important one for systems such as sedimentation basins, raingardens, constructed wetlands, ponds and lakes. However, it can be managed for swales and small size bio-retentions. Besides, it has been highlighted that "available space for some specific systems (i.e. bio-retentions) depends on the applied quality and quantity objectives. Small bio-retention can fit in the landscape but may have low system functionality". Available space also can be a challenge for rainwater tanks and dry wells in lot scale.

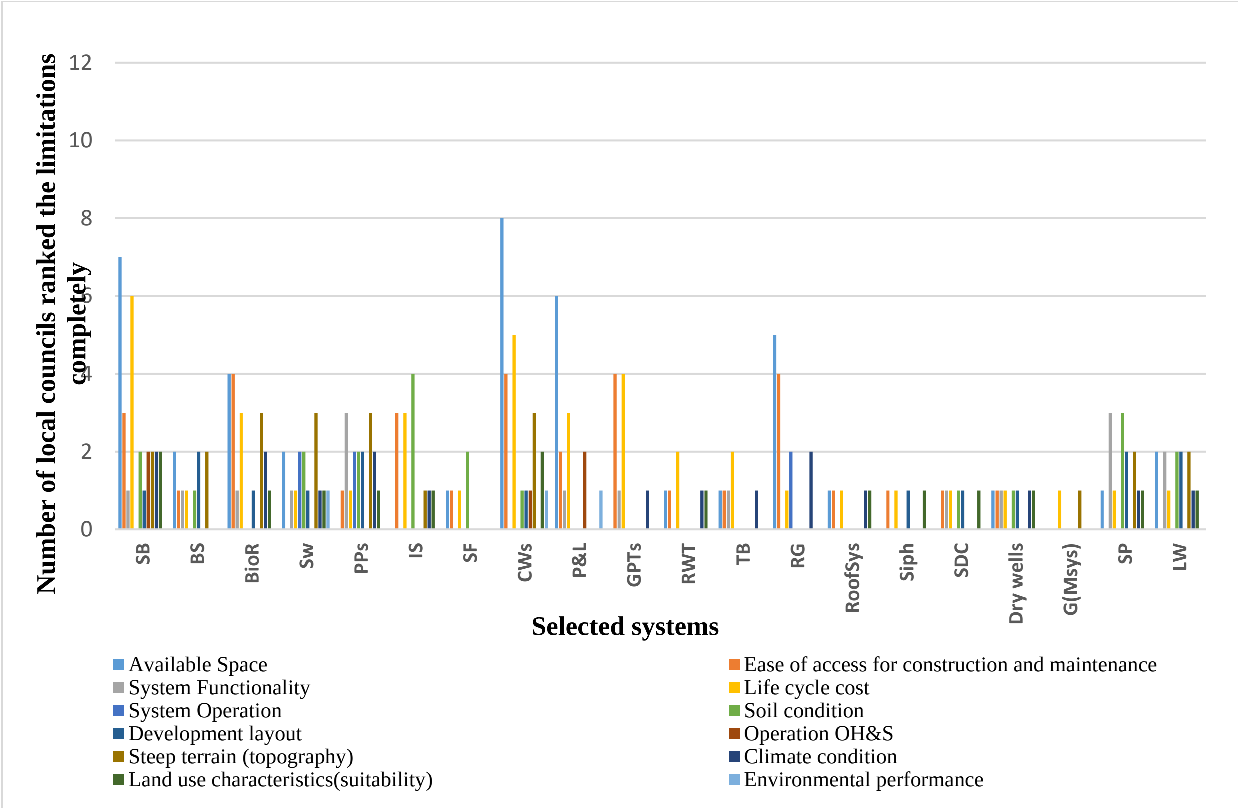
2- Ease of access for construction and maintenance: this limitation was also ranked as another important one due to the traffic and access to the systems in the existing condition. Sedimentation basins, bio-retention systems (raingardens), infiltration systems (i.e. trenches, basins and wells), sand filters, constructed Wetlands, ponds, lakes, and Gross Pollutant Traps (GPT) are required to be maintained by local government. Therefore, allotment scale systems are more preferred to be adopted. However, some issues brought into attention by some experts for these types of systems which can be problematic to either council or the adjacent owners. Problems such as outflow either into a different private property or council's road or lack of knowledge for the systems maintenance to keep their efficiency at a good level. Based on the results, swales and permeable pavements seems easier for maintenance.

3- System functionality: this limitation was mentioned as a matter of concern for permeable pavements, soakaways and leaky ways than the other systems with respect to stormwater quality management and limitations of soil type.
4- Life cycle cost and available funds were highlighted for most of the systems as a limitation, and all the associated costs for any type of adopted system was a matter of concern. However, this limitation has a higher weight for bio-retention and infiltration systems, sedimentation basins, roof systems and constructed wetlands than the other assets.
5- System operations: this limitation was highlighted for permeable pavements and raingardens in street scale adoption due to the traffic and maintenance. In addition, adopting raingardens was an issue at the lot scale because of ability and willingness to operate and maintain the systems.
6- Soil condition: this factor was mentioned as a limitation for infiltration systems application and as a consideration for adopting sedimentation basins, buffer stripes, swales and constructed wetlands. Besides, based on the results, it was identified that dry wells, soak pits, soakaways, leaky wells are not very practical in the selected areas for this study due to the type of soil and climate condition.
7- Development layout or suitability to surrounding land use: this one was mentioned as a limitation for swales, permeable pavements, constructed wetlands and buffer stripes.
8- Operation OH&S: this criterion was a challenge during the systems constructions and delivery; however, based on the safety rules and regulations adopted by councils, it is not a big concern.
9- Steep terrain: the site terrain/ slope (either steep or low) was mentioned as a big challenge for adopting swales, buffer strips, permeable pavements and constructed wetlands in street and catchment scale. In an allotment scale, this criterion can be a limitation for adopting leaky wells, infiltration trenches and soakaways.
10- Climate condition: this criterion was not a big concern except for rainwater tanks and bio-retention systems. Also, some systems are not much suitable for being adopted in Melbourne due to the climate condition (e.g., green roofs).
11- Land use characteristics: this criterion was considered as a challenge, especially for the allotment scale systems such as rainwater tanks, siphonic roof water systems and

dry wells. Resident's lack of knowledge for systems maintenance and costs are mentioned as limitations for applying these systems.

12- Environmental performance or public health: this criterion was highlighted as a big concern for ponds, lakes, constructed wetlands and raingardens due to the health problems may cause for the public realm (e.g., mosquitos).

Except the listed limitations in this study, some experts added other limitations including presence of other urban services (e.g., electricity, water, sewerage, communication), community objection and claims about systems issues and maintenance process, vegetation impact, gap of household's knowledge, consideration of the owners and their willingness to adopt the systems and downstream benefits. Biophysical and urban form, distance to metropolitan centre and age of development are some other important drivers for WSUD placements and abundance in Melbourne (Kuller et al., 2018). "Physical constraints, (e.g., restrictions the availability of open space and physical conditions (suitable geology), technical capacity, expertise of proponents and policy support at the state or local level, and ongoing funding for implementation and maintenance of the systems in South Australia" are listed as some other constraints for WSUD adoption (Mayer et al., 2013).




Legend as per Table 4-2 above

*Figure 4-1- Ranking of the selected WSUD assets considering their implementation limitations in studied areas*

### 4.3.2. Assets Preferences

Based on provided ranking for the selected WSUD techniques in a range of 1 to 3 and by calculating average weight for the techniques, preferred systems which achieve higher weight are stated as swales, bio-retention (raingardens) and GPT at street and catchment scale for the studied areas (Figure 4-2). However, in some areas average weight for constructed wetlands, ponds and lakes are close to GPTs or bio-retention systems. Based on the collected data for the studied areas bio-retention systems, swales and constructed wetlands are the common ranked systems. In ranking range, 1 is assigned to the limitations with high level of importance and 3 is assigned to the limitations with low level of importance for implementation. Rank 2 shows the medium range of importance for implementation limitations.

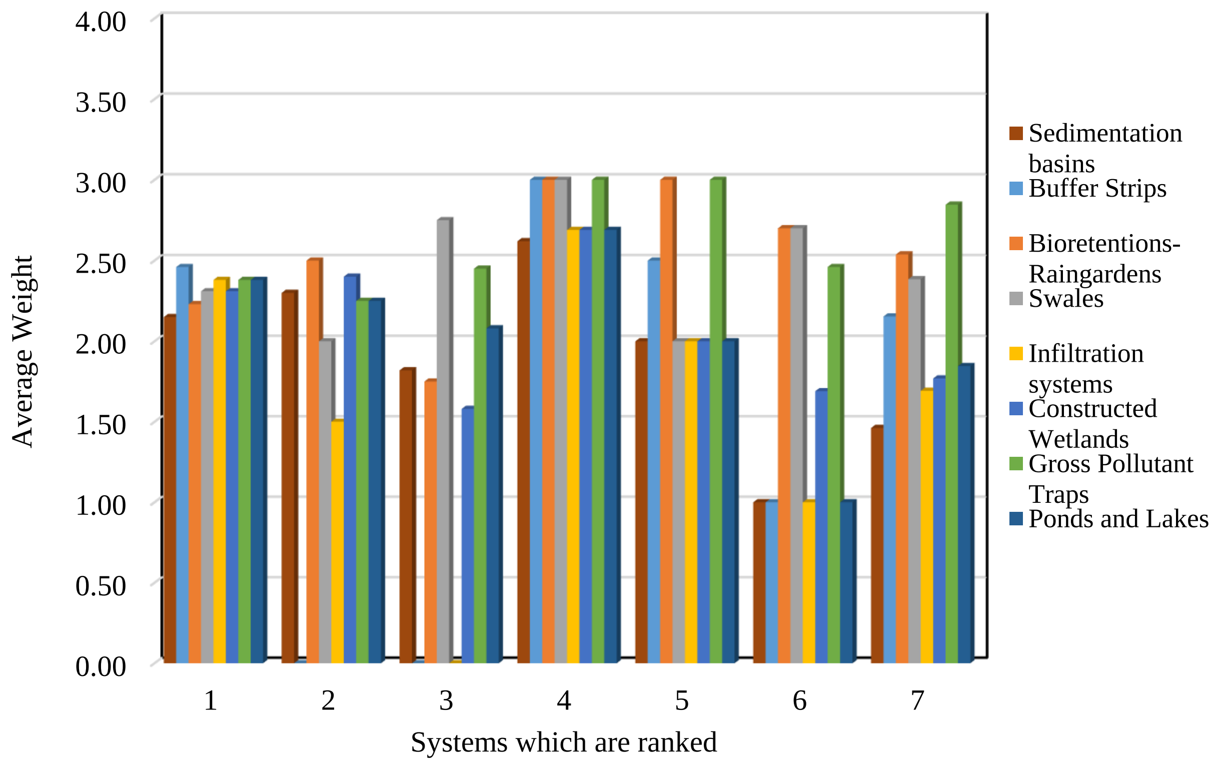



*Figure 4-2- Preferred system for WSUD assets adoption for the studied areas*

In addition, discussion with experts confirmed that at street scale if there is enough space; they are interested in adopting GPTs, raingardens, and bio-retention systems as these are suitable to manage runoff stormwater quantity and quality. Swales were also expressed as one of the most preferred systems as their maintenance is easy. In addition, buffer stripes can be suitable as they are easy to maintain and do not need large space for adoption. Wetlands are explained as good for implementation at the end of catchments systems if enough space is available. The general criteria, which was highlighted by experts for adopting preferred systems in their areas, is treatment capability and keeping cost effectiveness. Wetlands, raingardens, ponds, and lakes are stated as the most prominent systems in Melbourne (Kuller et al., 2018).

## 4.4. Conclusion

Several studies have discussed WSUD techniques adoption in greenfield and new development areas. This chapter discussed the implementation associated and limitations for the most commonly adopted WSUD techniques in existing developments in local government municipalities in Melbourne.

A survey was designed to compile a comprehensive list of the most commonly adopted WSUD techniques in Melbourne's urban areas, developed based on an extensive literature review and existing local guidelines and fact sheets. This list was then linked to the corresponding implementation constraints and key considerations. The identification of these constraints and considerations was initially informed by a thorough literature review and further refined through the survey process. To facilitate data collection, stormwater engineers from local government authorities in Melbourne were invited to participate. The study's methodology and objectives were explained to them, and they were asked to rank the identified limitations for each WSUD technique based on their professional expertise. Data was collected through in-person interviews, phone discussions, and self-administered survey responses. It can be highlighted that there are varieties of limitations and considerations for implementation of WSUD techniques in existing urban developments, so an average weighting approach was used for suggesting the best system in the studied areas.

Based on the available space, bio-retentions (raingardens) swales and GPTs are the most preferred systems for the studied areas in street and catchment scales. Swales and buffer stripes are also easy to maintain so can be suitable for existing conditions. However, preferred systems can vary based on the local condition and a combination of limitations and considerations.

Local governments encourage lot scale systems for adoption, however, residents' willingness, cost of maintenance and gap of knowledge are the most important limitations for systems application at lot scale. Based on collected information, selecting best WSUD techniques, keeping the system functionality at an efficient level and minimising the cost are big challenges for local governments while they also need to pay attention to public satisfaction and their feedback to select the best management practice for their areas. Such an information would help in decision making on the best practice management selection in existing urban areas with respect to the local problems and condition. Stormwater engineers ranked not all the listed techniques and limitations. The reason for that was, some techniques were not common or were less experienced (i.e., Geocellular/modular systems).

Based on the results of this survey, the most common techniques were chosen for hydraulic modelling and scenario running for this study. Chosen techniques included bio-retention (raingardens), vegetated swales, infiltration trenches, permeable pavements, and rainwater tanks. Bioretention and raingardens are the same therefore, raingardens have been used in this thesis.

# 5. Chapter 5: Stormwater Quality and Quantity Data Collection and Model Calibration

## 5.1. Introduction

Limited stormwater quality and quantity data is available to conduct research to understand the impact of WSUD implementation in existing developed areas. Thus, there is a need to generate such data for further research. A monitoring program conducted in the research study area located in west Melbourne funded by Melbourne Water. The aim was to collect stormwater quality and quantity data for a year to calibrate the stormwater modelling tool chosen for this research.

In consultation with Wyndham City Council, two sub-catchments were selected for the monitoring program. The sub-catchments are located in Werribee and discharge into a Melbourne Water Drain (D1), which is managed by Melbourne Water (MW). The use of suitable hydrological models to simulate rainfall-runoff events was an important component of this research. The Personal Computer Storm Water Management Model (PCSWMM) being the most widely used models in the world and across the Australia were chosen for stormwater modelling and rainfall-runoff simulation. Adopting the collected stormwater runoff data for the research study area, models' parameters were adjusted to represent the actual condition of the area. This chapter has discussed collected data, challenges in data collection, representation of WSUD techniques in hydraulic models and the calibration process undertaken for this research.

## 5.2. Stormwater Pollutant Drivers in Urban Areas

Urbanization has a considerable impact on stormwater quality in addition to its effects on the hydrology of the catchment. The physical, chemical, and microbiological quality of stormwater is predominantly declining, which has a negative impact on the aquatic environment of receiving water bodies (Liu 2011). Urbanization's effects on water quality can include salinity, temperature increases, sedimentation, dissolved oxygen depletion, introduction of harmful compounds, and biological effects (Zoppou 2001). Rainfall and the subsequent surface runoff pollute receiving waters and cause them to diverge from their ideal situation (House et al., 1993). Stormwater runoff can produce more pollutants than residential sewage discharge that has undergone subsequent treatment (Droste and Hartt 1975; Helsel et al., 1979).

The most significant pollution sources in urban areas are the several anthropogenic activities that are frequent, including industrial, commercial, and residential areas (Qin et al., 2010). Industrial processes, building development, and commercial activities all add a lot of pollutants to the urban environment. Both permeable and impervious surfaces can harbour these contaminants. Pollutants will be detached and transported to receiving waters as a result of the energy associated with rainfall and runoff (Jartun et al., 2008). Urbanization has led to the development of effective drainage systems, which makes it simple for contaminants to reach receiving waters. Sonzogni et al., (1980) found that urban areas had suspended solids and nutrient loads that were 10 to 100 times higher than those in non-urbanized areas during their study of pollution inputs from surface runoff. Similar trends were reported by Line et al., (2002), including a tenfold increase in solids loads and a doubling of nutrient loads from urbanised areas in comparison to rural areas. The quality of urban stormwater is greatly influenced by catchment and rainfall characteristics (Liebens 2001; Ahearn et al., 2005; Shigaki et al., 2007). While rainfall factors are crucial in the transport of pollutants, catchment characteristics primarily affect pollution sources and loads. Commercial land use leads in a comparatively high total solids load, according to research by Miguntanna (2009) and Herngren (2005) into various urban land uses. They connected this to the substantial volume of traffic and the range of anthropologic activities. In the Cosumnes Watershed in California, Ahearn et al., (2005) analysed 28 sub-basins and discovered that the proportion of urban to rural areas significantly influenced the loading of nitrate-N and total suspended particles. According to Huang et al., (2007), there was a higher level of nitrogen and phosphorus loading in stormwater runoff from commercial, residential, and park catchments. They explained this by pointing to these land use types' of rather thick vegetation. The association between rainfall quantity and phosphorus compounds was researched by Shigaki et al., in 2007. They discovered that runoff produced by rainfall with an intensity of 75 mm/h compared to 25 mm/h had considerably higher amounts of dissolved reactive phosphorus (DRP), particle phosphorus (PP), and total phosphorus (TP). High-kinetic-energy raindrops from heavy downpours can more easily separate contaminants that are stuck to catchment surfaces. Thusly discharged contaminants from catchment surfaces are easily carried to receiving waters by runoff.

Through physical and chemical processes, contaminants are transferred to the receiving waterways as stormwater runoff passes through catchment surfaces (Mikkelsen et al., 1994). Numerous reasons of stormwater quality degradation in metropolitan settings have been identified by researchers (for example Gnecco et al., 2005; Brown and Peake 2006; Kim et al., 2007). The following are the main sources of pollution:

- Vehicular traffic
- Industrial processes
- Vegetation
- Construction and demolition activities
- Corrosion
- Spills
- Erosion

Pollutant introduction causes stormwater quality to deteriorate in urban areas. The kind and load of the pollutant determine the degree of degradation.

Since other contaminants can adhere to the surfaces of suspended solids, they are regarded as the most significant pollutant (Ball et al., 2000). In addition, nutrients and organic carbon were named as the two most significant water contaminants in catchments of Southeast Queensland, according to research by Healthy Waterways (2011). As a result, the main pollutants that affect the quality of urban stormwater, such as suspended solids and was found significantly in the stormwater collected data or this research is mentioned below.

## 5.3. Primary Stormwater Pollutants

Urban areas experience a decline in stormwater quality due to the introduction of pollutants. The extent of this decline varies depending on the type and quantity of pollutants present. Suspended solids are particularly significant as they can attract other pollutants to their surfaces (Ball et al., 2000). Moreover, a Healthy Waterways report (2011) highlights nutrients and organic carbon as the primary water pollutants in Southeast Queensland catchments. As a result, this discussion focuses on the major pollutants—suspended solids, nutrients, and organic carbon—and their impact on urban stormwater quality. Additionally, toxicants and litter are addressed due to their common occurrence in urban stormwater.

### 5.3.1. Suspended solids

Because other pollutants including heavy metals, minerals, viruses, and hydrocarbons can bind to the solid’s particles, suspended solids play a crucial role in stormwater quality (Ball et al., 2000;

Vaze et al., 2000; Jartun et al., 2008; Kayhanian et al., 2008; Clark and Pitt 2009). According to research, suspended solids are the greatest non-point source contaminant in urban receiving waters on a volumetric basis (Munoz and Panero 2008). Physical and chemical effects are the two main ways that suspended solids affect the quality of water. Reduced water transparency, which prevents photosynthesis, is one of the physical effects. The suffocation of bottom-dwelling animals and plants as well as alterations to the substrata can also result from excessive loads of solids.

The chemical effects of suspended solids on water quality should receive more consideration than their physical effects. Adsorption of other contaminants and their transfer into receiving waters are the main chemical effects of solids. Pollutant movement is significantly influenced by particle size. Typically, even in a fast flow, large particles are difficult to suspend, whereas small particles are simple to suspend for extended periods of time transported to receiving waters (Goonetilleke and Thomas 2003). Stormwater runoff typically transports suspended materials from road surfaces, rooftops, building sites, and pervious places. According to Nelson and Booth (2002), human activity contributes more to the concentration of fine particles than do natural sources. In the urban catchment they studied, Gobel et al., (2007) found that roadways had higher concentrations of total suspended solids (TSS) than other places. Land use patterns have an impact on the kind and number of suspended particles (Stein et al., 2008). The soil and its protective cover are very likely to be disturbed by anthropogenic activities like construction and plant removal. As a result, rainwater runoff can easily loosen and move the exposed soil.

### 5.3.2. Nutrients

The living tissue depends on nutrients, which comprise elements like calcium, potassium, iron, manganese, phosphorus, carbon, and nitrogen. Nitrogen and phosphorus molecules are the most significant nutrients and the most prevalent in urban stormwater runoff.

An oversupply of nutrients in water bodies can cause plants to grow too quickly, which can lower the dissolved oxygen content of the water because of microbial degradation that occurs when the plants die and decay. Water bodies' biological environments can be deteriorated by an increase in anaerobic bacterial overgrowth and the extinction of aquatic life as a result of a drop in dissolved oxygen. In urban settings, eutrophication—a condition caused by nutrient enrichment—has grown to be a significant environmental concern (Gray et al. 2008; Lewitus et al. 2008). Furthermore, illnesses like stomach tumours can be brought on by high nitrates in drinking water. Furthermore, water with more than 10 mg/L of nitrate might taste unpleasant and produce physiological discomfort (Straub 1989).

Stormwater discharge receives nutrients from a range of sources. Plant debris, animal litter, lawn fertilizer, wastewater from homes and businesses, and automobile emissions are a few of them (Wong et al. 2000; Graves et al. 2004). Stormwater discharge carries nutrients in both dissolved and particle forms. Researchers have found that while nitrogen is largely delivered in dissolved form by stormwater runoff, phosphorus is primarily transferred in particulate form (Uusitalo et al. 2000; Wong et al. 2000; Quinton et al. 2001; Jian et al. 2007). Carpenter et al. (1998) have identified septic tanks and industrial wastewater as significant sources of nitrogen pollution. According to Emilsson et al. (2007), fertilizer is yet another significant source of nitrogen. Researchers have looked into how fertilizer and manure affect runoff's phosphorus and nitrogen contents. For instance, a study conducted in 2007 by Huang et al. revealed that compared to other urban land uses, residential and park land uses have greater total nutrient concentrations.

Carpenter et al. (1998) have identified septic tanks and industrial wastewater as significant sources of nitrogen pollution. According to Emilsson et al. (2007), fertilizer is yet another significant source of nitrogen. Researchers have looked into how fertilizer and manure affect runoff's phosphorus and nitrogen contents. For instance, a study conducted in 2007 by Huang et al. revealed that compared to other urban land uses, residential and park land uses have greater total nutrient concentrations.

This is explained by the numerous variables influencing the interdependent processes. These variables include the type of soil, the characteristics of rainfall and runoff, land use, and human activities within the watershed (Goonetilleke and Thomas 2003). In cities, vehicle emissions are another source of nutrients. According to Sawer et al. (2000), the two types of vehicles that contribute the most to nitrogen oxides (Nox) are light- and heavy-duty vehicles. 51% of nitrogen oxides in California in 2006 came from vehicles, according to data from the California Air Resources Board (CARB 2006).

### 5.3.3. Organic Carbon

Compounds referred to as “organic carbon” or “oxygen-demanding materials” can be converted by bacteria into carbon dioxide (CO2) and water. The main effect of organic matter is that it causes microbial oxidation, which lowers the amount of dissolved oxygen in water bodies. Moreover, it can contaminate water supplies, produce offensive odours, and lessen the recreational value of waterways (Ellis 1989; Warren et al. 2003). Furthermore, Herngren et al. (2010) discovered that organic carbon has a significant impact on the dispersion of hydrocarbons by affecting how well they adsorb onto solids and how stormwater runoff carries them. Compared to coarse particles, organic matter is more attracted to fine particles.

Street sweeping is ineffective at removing tiny particles, which can include anywhere from 34 to 99.5% of oxygen demand loading, according to Sator and Boyd (1972). Therefore, surface runoff collects most of the organic stuff that is present on street surfaces.

In their study of stormwater runoff from a road surface in Louisiana, USA, Sansalone and Tittlebaum (2001) found that organic matter accounted for an average of 29% of the total suspended solids (TSS) concentration. Moreover, suspended solids’ ability to sorb hydrophobic organic compounds and certain heavy metals, like Pb and Zn, is increased when organic carbon is adsorbed to them (Parks and Baker 1997).

The ease with which organic matter decomposes microbially—a trait that is often regarded as advantageous—returns contaminants to the dissolved phase. Urban receiving waterways receive a significant amount of organic carbon from street surfaces, including plant debris and street litter (Gilbert and Clausen 2006; Gobel et al. 2007; Pappas et al. 2008). According to Sator and Boyd (1972), street sweeping frequency and catchment characteristics have a significant impact on the loading and concentration of organic carbon. Furthermore, compared to inorganic materials, organic carbon accumulates on street surfaces far more quickly (Sartor and Boyd 1972).

One likely explanation is that leaves, and litter predominate over deposited sand and dust particles in urban environments. Moreover, the amount of organic carbon changes according to how land is used. For all particle sizes, residential land use has the largest organic carbon loading, according to Miguntanna (2009). This was explained by the fact that there was more vegetation in the vicinity of the residential road surface under investigation. According to Egodawatta’s 2007 research, a sizable portion of the organic chemicals detected in surface pollution from roads were soluble for all land uses.

### 5.3.4. Toxicants

Chemical substances that are detrimental or damaging to living things are known as toxicants. Usually, human activity releases toxicants into the environment (Lippmann 2009). In stormwater quality study, heavy metals and hydrocarbons are two of the most significant toxicants and are of special concern (Herngren 2005; Brown and Peake 2006; Liu et al. 2010). Because of their potential toxicity, heavy metals found in urban stormwater runoff have received a lot of attention. Furthermore, unlike the majority of other contaminants, they do not break down in the environment. Car traffic is one of the main sources of heavy metals in stormwater runoff (Dong et al. 1984; Sansalone and Buchberger 1997). Furthermore, air deposition and building sidings are thought to be sources of heavy metals in stormwater runoff. Like heavy metals, hydrocarbon molecules are mostly carried by solids because of their great affinity, especially for tiny particles. Because of their toxicity to aquatic animals, multiple origins, and potential for high concentration, polycyclic aromatic hydrocarbons, or PAHs, are a particular issue when it comes to hydrocarbon contaminants in stormwater runoff (Beasley and Kneale 2002). Both natural and human-caused sources can produce PAHs (Larkin and Hall 1998; Van Metre et al. 2000). Anaerobic organic material deterioration and forest fires are the principal examples of natural sources, whereas storage facilities and automobiles are the principal examples of anthropogenic sources. PAHs are found in the aquatic environment from both natural and anthropogenic sources, but the contributions from anthropogenic sources are significantly higher (Stenstrom et al. 1984).

### 5.3.5. Litter

Any land usage can produce litter. Litter is not a major contributor to water contamination, despite being extensively dispersed. Litter's most noticeable effect is on aesthetics because of its huge particle size, which causes it to float on the water's surface. Litter buildup can also impede stormwater conveyance by clogging urban drainage systems. Litter may affect the quality of the water in an urban environment, but because of its size, it is difficult for stormwater runoff to collect. Additionally, routine street sweeping can be an excellent way to get rid of litter. Litter is therefore typically not considered to be a significant urban stormwater pollutant (Liu 2011).

## 5.4. Land use and Pollutant Definition

The characteristics of the catchment and the amount of rainfall have a significant impact on the quality of urban stormwater. Pollutant generation is primarily influenced by catchment factors, whereas pollutant transportation is primarily influenced by rainfall variables.

While rainfall characteristics reflect the changes in rainfall events primarily in regard to duration and intensity, catchment characteristics incorporate a variety of elements such as land use, urban structure, and urban area location (Liu 2011)

Build-up and wash-off processes are closely related to pollutant formation and transportation. Therefore, the effects of catchment and rainfall parameters on these two pollutant processes describe how they affect urban stormwater quality.

### 5.4.1. Pollutant Buildup

Pollutant build-up is referred to as both dry and wet deposition-related pollutant accumulation.

It is a complicated process that is affected by a variety of elements, including traffic, land use, impermeable surfaces, and previous dry periods (Vaze and Chiew 2002; Deletic and Orr 2005).

Pollutants are brought into an urban catchment by human activity and natural sources, and are then eliminated by rainfall, wind, and street sweeping (Pitt 1979). Wet and dry deposition are the key factors that affect how pollution buildup is impacted by natural events (Savinov et al., 2000; Deletic and Orr 2005).

Pollutant buildup is attributed to anthropogenic activities such as industrial operations, automobile emissions, and tyre wear (Brodie and Porter 2006; Sabina et al., 2006). In order to conduct a quantitative examination of the build-up process, many researchers have attempted to apply mathematical formulae (see, for instance, Egodawatta 2007; Liu et al., 2010).

### 5.4.2. Pollutant Wash-off

When pollutants that accumulated on the surface during the dry season are added to stormwater flow, the process is referred to as washing off. It can be viewed as a mix of the conveyance and the separation from the catchment surface. First, as raindrops hit the ground, they wet the surface, dislodging particle pollution and dissolving water-soluble contaminants. Second, stormwater runoff is used to convey the separated contaminants.

The qualities of a downpour, such as its strength, greatly affect the number of contaminants that wash off (Vaze and Chiew 2002; Brodie and Rosewell 2007). TSS concentration tended to be high with an increase in rainfall intensity, according to research by Gnecco et al., (2005) examining the association between contaminants wash-off and stormwater runoff. This shows that rainfall variables have a direct impact on the pollutant properties of runoff. Higher rainfall intensity results in a higher pollution burden. This is explained by the fact that greater rainfall intensity has the ability to carry and separate contaminants from the surface more effectively. According to Renard et al., (1997), the threshold intensity for pollution wash-off from rural catchments might range from 67 mm/h to 50 mm/h (Hudson 1993). This suggests that significant rainfall intensity and relatively high rainfall kinetic energy would be necessary for contaminants to wash off from rural catchments.

## 5.5. Stormwater Monitoring Sites Selection and Requirements

Wyndham City Council (WCC) which is located on the western edge of Melbourne and covers an area of 542 $km^2$ is experiencing rapid growth and development. In addition, the area has quite significant different climate and rainfall patterns within the northern and eastern suburbs. Two developed urban catchments were selected for the monitoring program. The catchments cover total area of 5.7 km2 and located on the left side of the Pacific Werribee shopping centre. It has been surrounded by Sayers Road in the north and the Heaths Road in the south. The Wootten road is the main street located to the west side of the site (Figure 5-1).

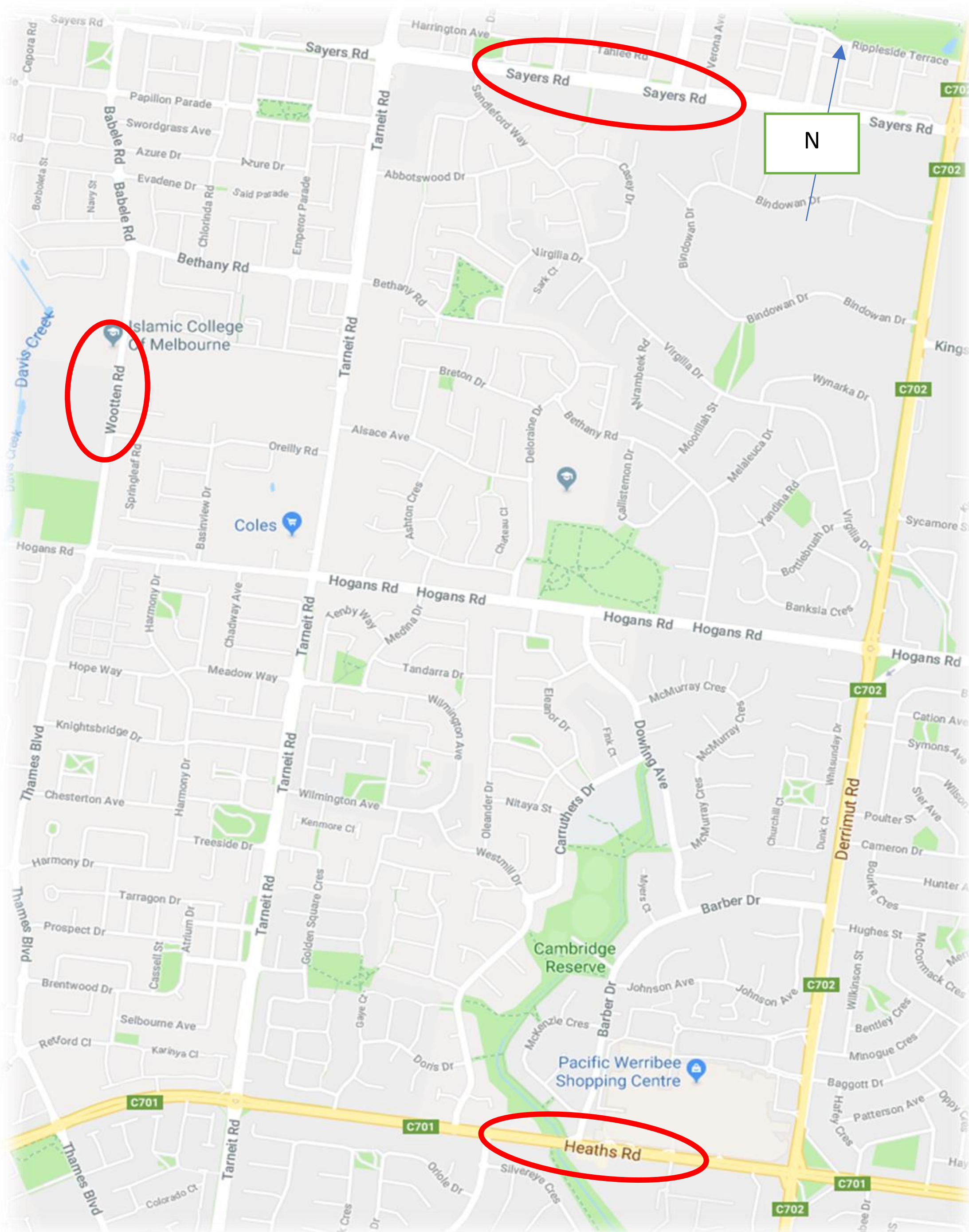



*Figure 5-1- Location of the sub-catchments were chosen for stormwater monitoring program.*

Two catchments were selected within the existing area to monitor the urban stormwater run- off quality and quantity. Both areas discharge into D1 drain therefore, two spots were identified along with the stormwater pipe networks in each catchment as overland flow locations (Figure 5-2).

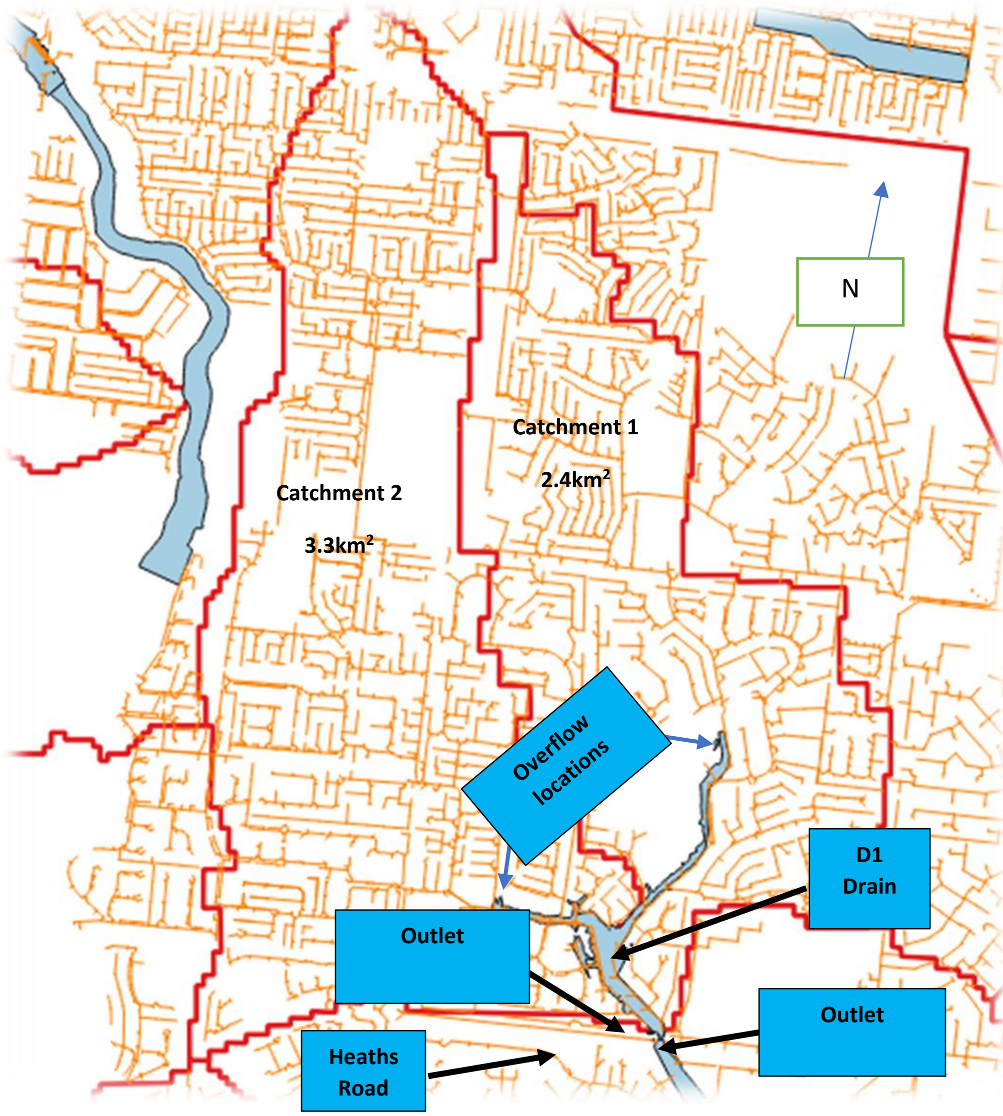



*Figure 5-2- Selected sub-catchments for stormwater quality and quantity monitoring*

Local information for the stormwater network pipes, D1 drain, and pits were collected from WCC and MW. After finalising the catchments boundaries, understanding D1 drain location, and collecting stormwater pipes and pits information as GIS layers, site inspections were conducted with WCC stormwater engineer to select the stormwater pits (manholes) to install the stormwater quality and quantity monitoring equipment. The locations of these stormwater pits are shown in Figure 5-3.


*Figure 5-3- Existing manhole's locations for the pipes 1 and 2 in the site.*

Figures 5-4 and 5-5 show the pipes and pits in the area for both stormwater network pipes. The first pit close to the stormwater pipe outlets were chosen to install the monitoring instruments. Stormwater quality auto- sampler was installed in the catchment 2 to collect stormwater samples for rainfall events more than 2 millimetres. Work was commenced by ALS global company in May 2019 and finished in August 2020. Stormwater flow data was collected every 6-minutes for both catchments via a flow meter and recorded in data loggers.


*Figure 5-4- Pipe 1 outlet and manhole*


*Figure 5-5- Pipe 2 outlet and manhole*

The dimensions of the pipes, pits and sites' requirements are listed in Table 5-1 and Table 5-2.

Table 5-1- Manhole’s information

| Pipes | Pit number | Pit ID | Dimension |
|---|---|---|---|
| Pipe number 1 –<br>Catchments 1(1350mm) | Pit 1 | 8071/MH003 | 580mm x 880mm<br>Depth =1900mm |
| Pipe Number 2 –<br>Catchment 2 (1500mm) | Pit 2 | 8090/MH001 | 2100mm x 4920mm<br>Depth = 2260mm |

Table 5-2- Sites monitoring requirements.

| Sites | Requirements |
|---|---|
| Catchment 1 (Site 1) | - Flow monitoring in 1800 mm stormwater pipe<br>- Over land flow monitoring in 1650 mm stormwater pipe |
| Catchment 2 (Site 2) | - Flow monitoring in 1350 mm stormwater pipe<br>- Flow monitoring and quality autosampler cabinet |

## 5.6. Stormwater Data Collection

Installed equipment were included a data logger, a battery, a flow meter sensor in the stormwater pipes and a pressure sensor and a ruler for overland flow monitoring locations (Figure 5-6, 5-7 and 5-8).

*Figure 5-6- Data logger and battery*

*Figure 5-7- Flow meter sensor in the stormwater pipes*

*Figure 5-8- Pressure sensor and ruler*

Stormwater flow data was collected and recorded for 12 months period every 6- minutes interval. Collected data was used for stormwater modelling tools catchment's parameters calibration in the study area.

Stormwater quality samples were collected by an automatic autosampler (Figure 5-9). The sampler was included fourteen bottles. It was run for selected rainfall events with more than 2-mm rainfalls throughout the year. There are three approaches for stormwater sample collection including flow-weighted, time- weighted and user-defined. The auto-sampler was set at 100mm level in the stormwater pipe to start – sample's collection and continued to fill the bottles every 15 minutes. In each round of data collection, several bottles from the fourteen bottles were chosen for stormwater quality analysis. A list of urban stormwater pollutants was prepared based on the literature review and was considered for stormwater samples analysis in ALS Global laboratory for the collected samples for each rainfall event individually (Table 5-3).

*Figure 5-9- Automatic autosampler cabinet and required attachments.*

Able 5-3-List of selected stormwater pollutant for quality analysis

| **Pollutant parameters** | |
|---|---|
| *Nutrients (Nitrogen, Nitrite $NO_2$, Ammonia $NH_3$, Nitrate $NO_3$, Total Phosphorous (TS), Soluble Phosphorous (SP)) | |
| Metals | Cd |
| | Cr |
| | Cu |
| | Ni |
| | Pb |
| | Zn |
| | Mn |
| | Fe |
| Cyanide | |
| Sodium, Calcium, Chloride | |
| Sulphate | |
| Hydrocarbons | |
| Faecal bacteria | |
| Total Suspended Solids (TSS), Total Dissolved Solids (TDS) | |
| Oxygen-demanding Substances and Dissolved Oxygen (Biochemical Oxygen Demand (BOD), Chemical Oxygen Demand (COD), and Total Organic Carbon (TOC) | |

## 5.7. Stormwater Data Collection Challenges

In total, thirty samples were collected for quality analysis during the monitoring period. Two of them were grabbed manually and the other samples were collected by the auto-sampler after specific rainfall events. The autosampler had fourteen bottles and for different events and in each month, different bottles were collected for quality analysis from bottle 1 to 14. Stormwater runoff was monitored for the whole period for both selected catchments however, there were challenges associated within the stormwater pipe's monitoring. These challenges caused flow data loss for some short periods. It was vital to install the equipment in a cabinet or inside the stormwater pipes to reduce the vandalism. Therefore, concentration of silt inside the data logger, remote report connection loss and flat batteries where the most important challenges came up during the monitoring period.

## 5.8. Collected Stormwater Quality and Quantity Data

### 5.8.1. Collected Stormwater Quantity Data

Stormwater runoff for both catchments is plotted against the time for the monitoring period. Maximum stormwater runoff peaks were observed as 2.2 m3/s and 2.3 m3/s for catchment 1 and 2 respectively (Figure 5-10 and Figure 5-11).

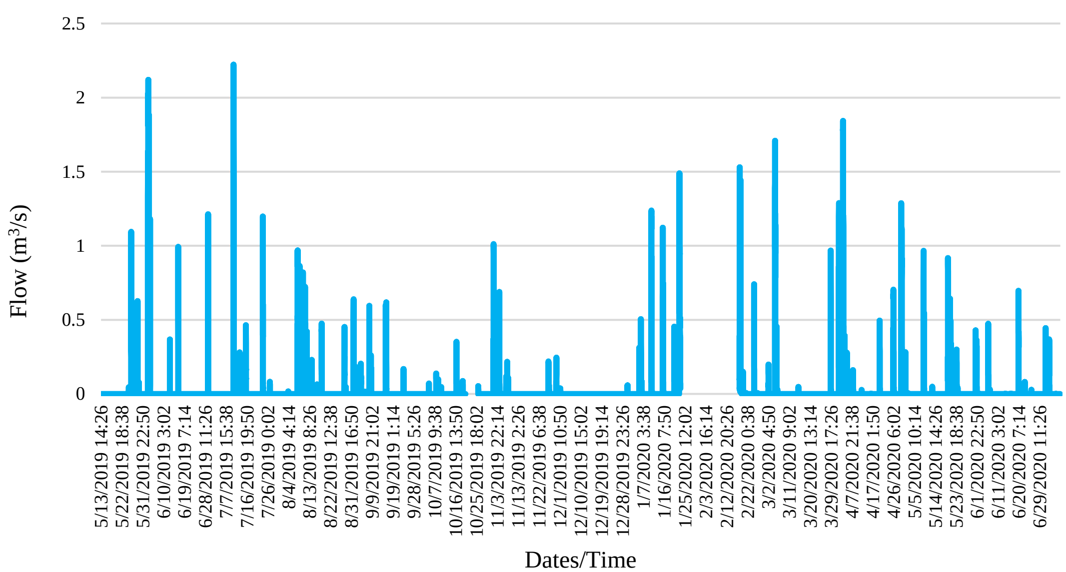



*Figure 5-10- Stormwater runoff reported and collected for the catchment 1*

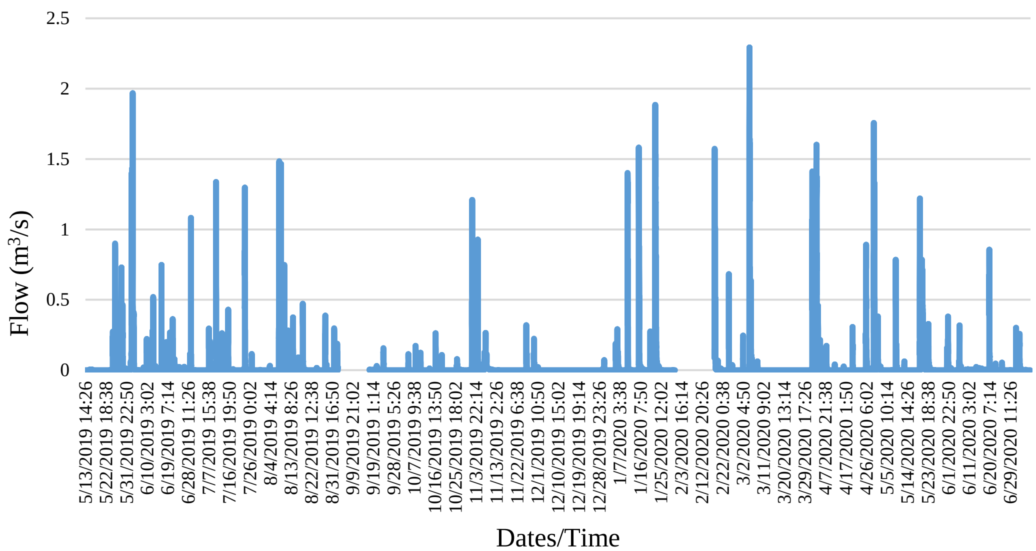



*Figure 5-11- Stormwater runoff reported and collected for the catchment 2*

For some periods which data was lost, gaps are shown on the graphs. The collected data was used to adjust the sub-catchments parameters in PCSWMM model which is covered this chapter.

### 5.8.2. Collected Stormwater Quality Data

Results of twenty-eight samples collected by the auto-sampler and analysed in the ALS Global laboratory are presented in this section. List of selected pollutants for analysis has been shown in Table 4-3. Heavy metals didn’t show significant changes for the collected samples during the whole monitoring period (Figure 5-12 and Figure 5-13). The analysis of heavy metals was ignored for two months. Heavy metals varied from 0.001mg/l to 0.11 mg/l and Total Iron varied from 0.48 mg/l to 6 mg/l in the collected samples. Range of changes for Total Dissolved Solids (TDS), Total Suspended Solids (TSS), Total Nitrogen (TN), Total Phosphorus (TP) and Biochemical Oxygen Demand (BOD) are presented in Figure 5-14 to Figure 5-18. TDS varied from 51 to 394 mg/l, TSS varied from 5 to 1030 mg/l, TN and TP varied from 0.16 to 3 mg/l and 0.06 to 0.38 mg/l respectively and BOD varied from 3 to 43 mg/l.

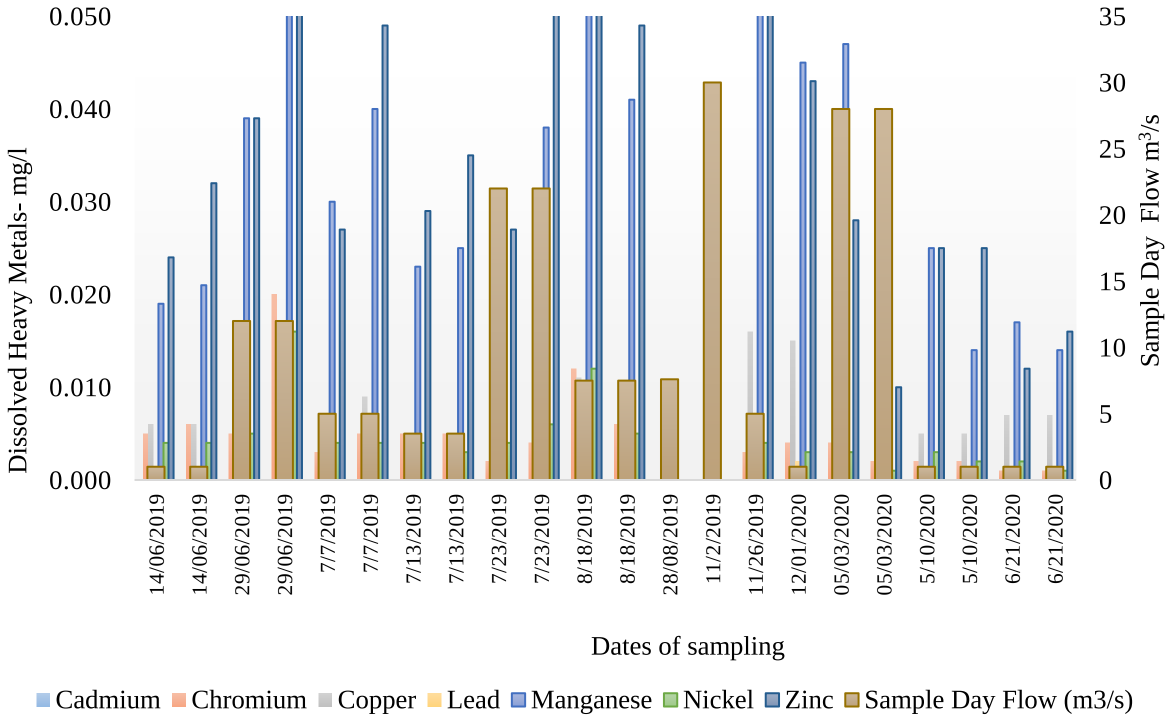



*Figure 5-12- Total Heavy Metals changes in collected samples in the sub-catchment outlet*

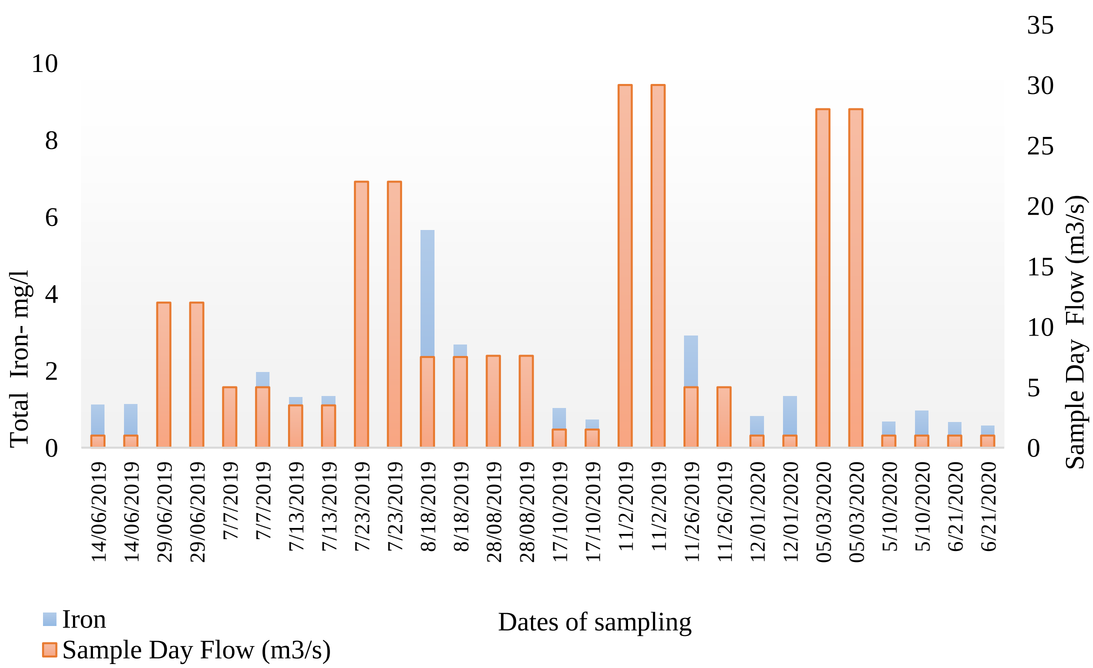



*Figure 5-13- Iron changes in collected samples in the sub-catchment outlet*

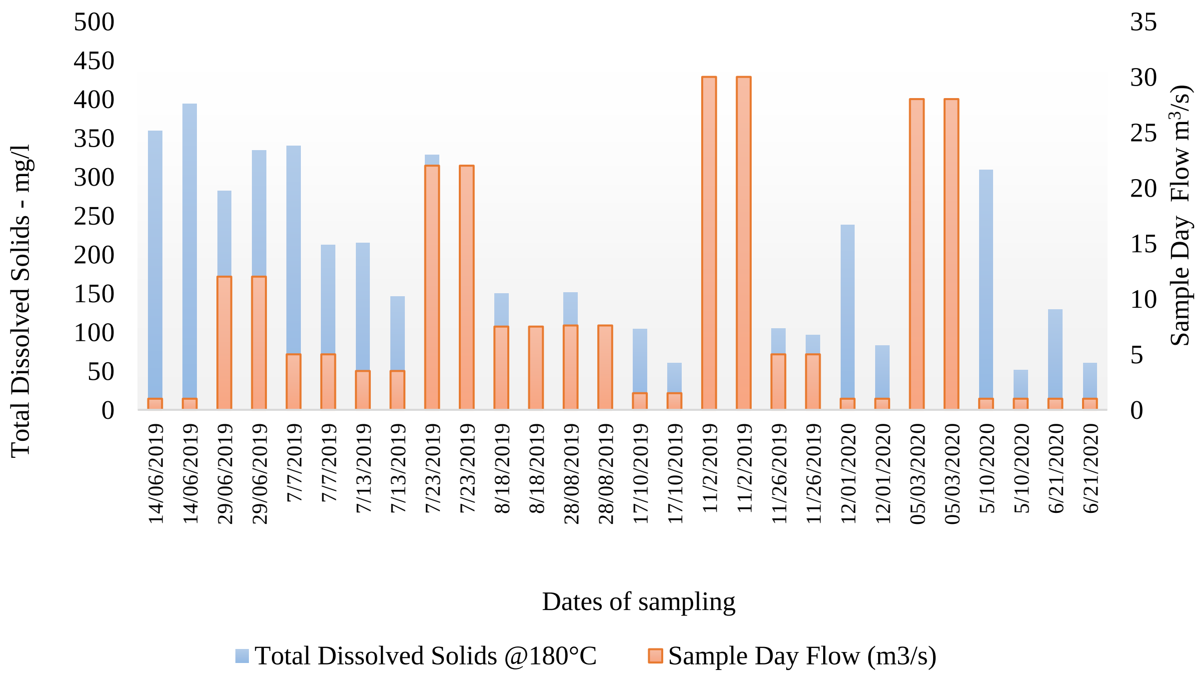



*Figure 5-14- TDS changes in collected samples in the sub-catchment outlet*

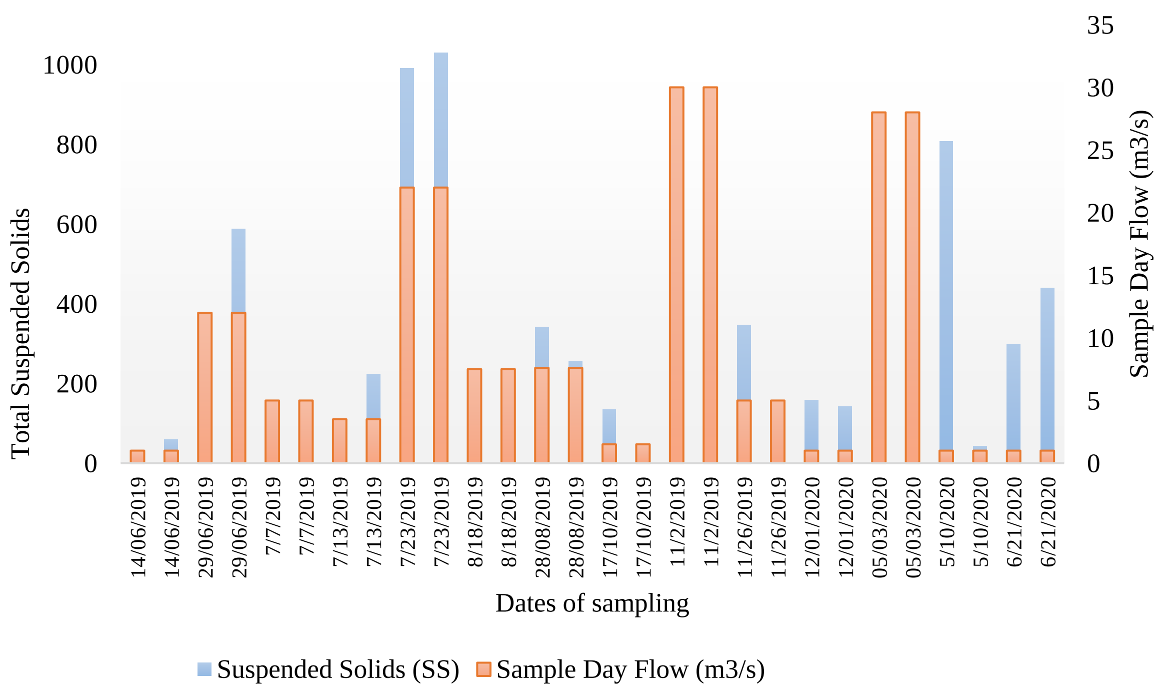
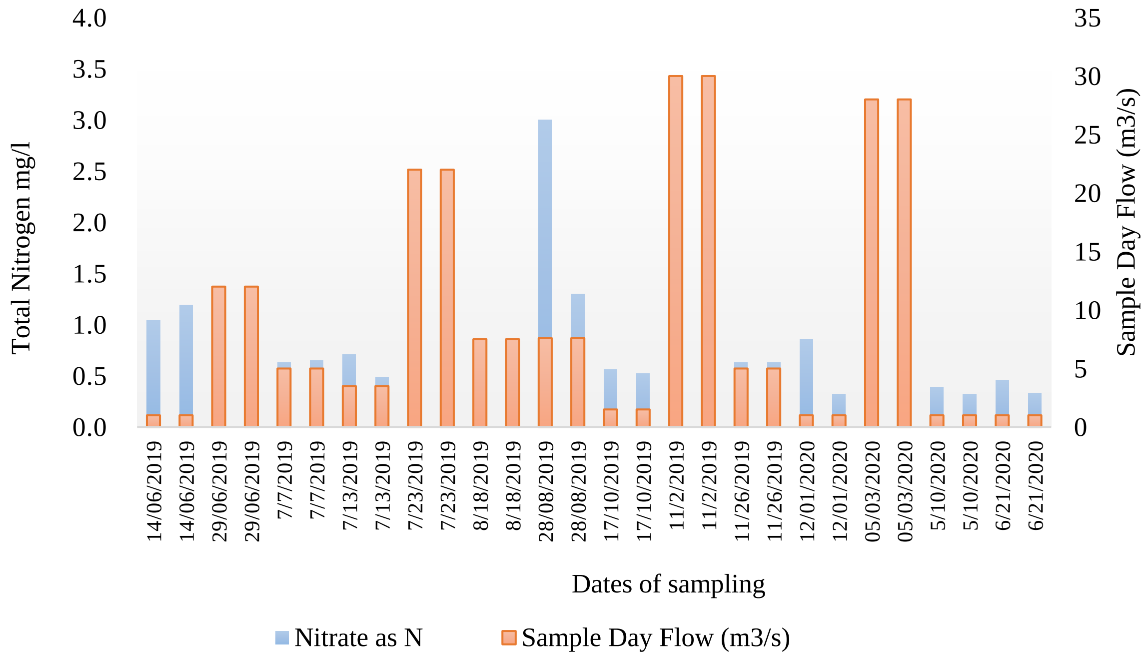
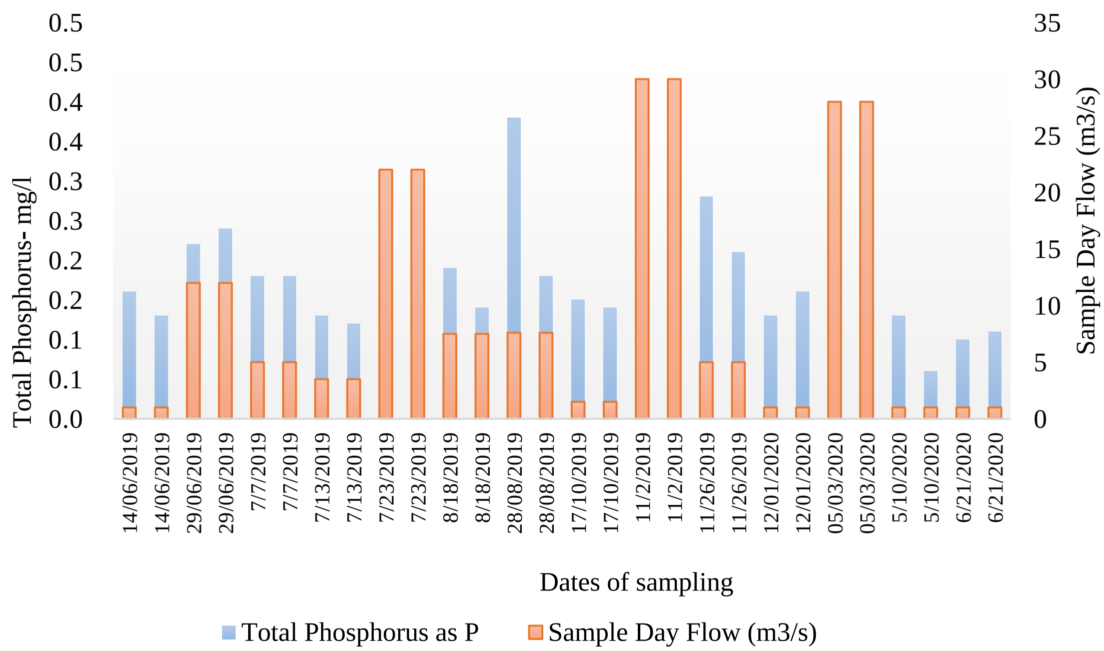


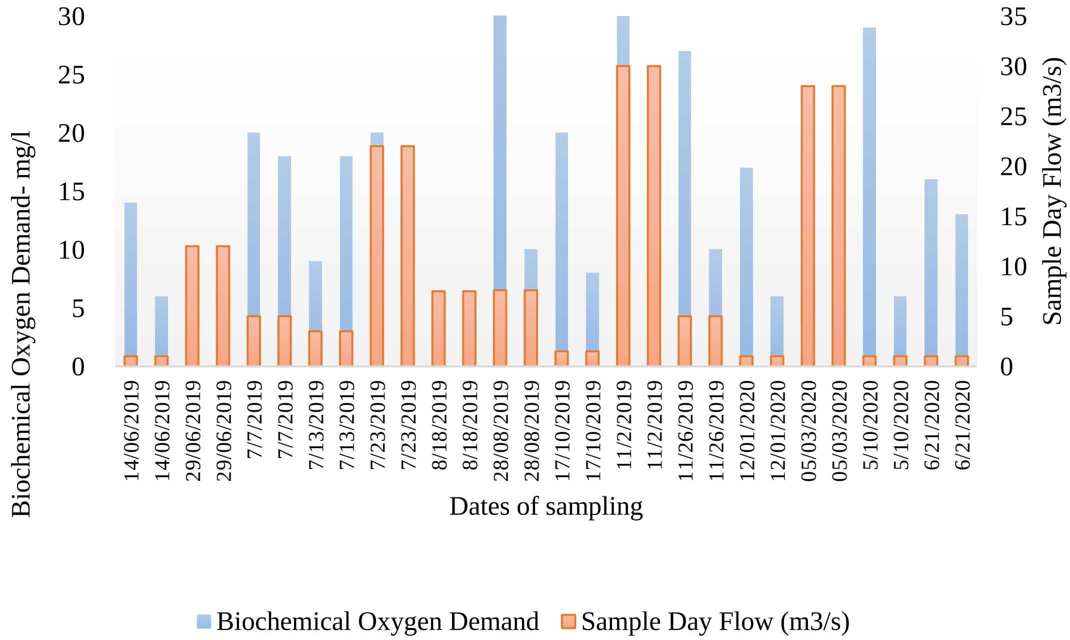

*Figure 5-15- TSS changes in collected samples in the sub-catchment outlet*




1554 *Figure 5-16- TN changes in collected samples in the sub-catchment outlet*




1555 *Figure 5-17- TP changes in collected samples in the sub-catchment outlet*




1556

*Figure 5-18- BOD changes in collected samples in the sub-catchment outlet*

Figure 5-19 further shows the standard deviation and mean value of pollutants, which are compared with the values of urban catchments worldwide and in Australia in Tables 5-5 and 5-6. TSS values are noticeably high while TP, TN, and Cd values are quite low in the research area.

1557


*Figure 5-19- Standard Deviation and Mean for pollutants and collected samples in the sub-catchment outlet.*

Several monitoring works have recorded urban stormwater pollutant in some urban catchments in Melbourne that are summarized in Table 5-4. All catchments are in the eastern and south-eastern Melbourne and experiences climatic conditions.

Table 5-4- List of sites have been monitored for stormwater quality In Melbourne (Francey, 2010).

| Site | Primary land use | Area (ha) | Total Fraction Impervious | Number of events |
|---|---|---|---|---|
| Gilby Rd, Mt. Waverley | Commercial | 28.2 | 0.78 | 49 |
| Kilgerron Crt, Narre Warren | Rural residential | 10.5 | 0.2 | 40 |
| Sheperds Bush, Glen Waverley | Medium density residential | 38 | 0.4 | 38 |
| Monash roof, Clayton | Coated aluminium roof | 0.046 | 1 | 30 |
| Eley Rd, Burwood East | Mixed | 186 | 0.46 | 31 |
| Madden Grove, Richmond | High density residential | 89.2 | 0.74 | 40 |
| Ruffeys Lake, Doncaster | Medium density residential | 105.7 | 0.51 | 54 |

A comparison of SD and Mean calculated for the collected samples for this research and available data for Melbourne's catchments was conducted and the results are provided in Table 5-5.

Table 5-5- Summary of pollutants for urban catchments in Melbourne compared with results analysis collected for this research

| Site | Parameter | No of Events | SD | Mean (mg/l) |
|---|---|---|---|---|
| Gilby Rd, Mt. ***Waverley*** | TSS | 49 | 59.2 | 7.65 |
| | TP | 47 | 0.13 | 0.22 |
| | TN | 47 | 0.82 | 1.13 |
| Kilgerron Crt, ***Narre Warren*** | TSS | 41 | 65.8 | 10 |
| | TP | 17 | 0.20 | 9.01 |
| | TN | 17 | 0.72 | 32.6 |
| Sheperds Bush, ***Glen Waverley*** | TSS | 40 | 79.5 | 20.6 |
| | TP | 36 | 0.18 | 0.23 |
| | TN | 36 | 0.97 | 2.34 |
| Monash roof, ***Clayton*** | TSS | 30 | 35 | - |
| | TP | 30 | 0.04 | - |
| | TN | 30 | 0.69 | - |
| Eley Rd, ***Burwood East*** | TSS | 32 | 93.2 | 7.27 |
| | TP | 16 | 0.06 | 0.63 |
| | TN | 16 | 0.54 | 3.41 |
| Madden Grove, ***Richmond*** | TSS | 40 | 92.1 | 12.6 |
| | TP | 39 | 0.31 | 0.42 |
| | TN | 39 | 1.27 | 11.6 |
| Ruffeys Lake, ***Doncaster*** | TSS | 52 | 56.3 | 16 |

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Typical concentrations of worldwide stormwater pollutants have been summarized in Table 5-6 (Fuchs et. al. 2004).

Table 5-6- Typical concentrations of worldwide stormwater pollutants

| Worldwide | | | This research | |
|---|---|---|---|---|
| **Parameter** | **No of records** | **Median (mg/l)** | **No of records** | **Median (mg/l)** |
| TSS | 178 | 141 | 30 | 241 |
| BOD | 88 | 13 | | 14.5 |
| COD | 136 | 81 | | 36.35 |
| TP | 149 | 0.42 | | 0.16 |
| TN | 17 | 2.36 | | 0.07 |
| Cd | 54 | 2.3 | | 0.004 |

## 5.9. Stormwater Models Calibration

Two urban catchments were selected in WCC municipality with an area of 570ha (Figure 5-20). Physical characteristics of the area is presented in Table 5-7. There are 17,272,503 meters of pipe in total length and 5806 pits in the area. Additionally, pipe sizes range from 100 to 2250 mm.

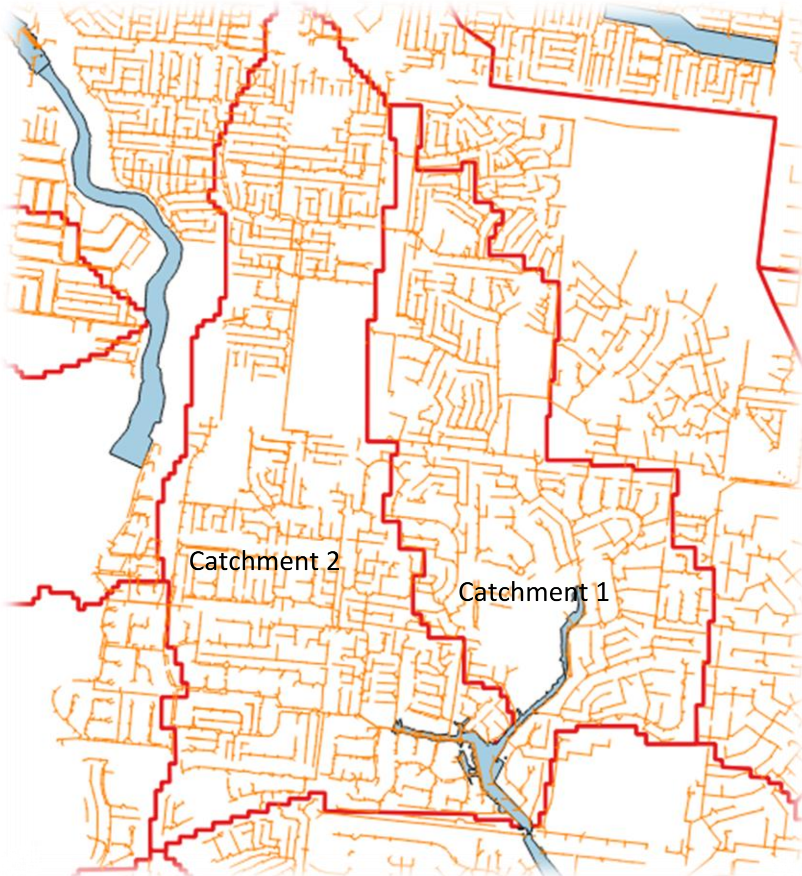


*Figure 5-20- Two urban catchments selected for the research study area.*

Table 5-7- Physical Characteristics of the catchments

| **Parameters** | **Catchment 1** | **Catchment 2** |
|---|---|---|
| Area (ha) | 240 | 330 |
| Average slope % (m/m) | 0.5 | 0.5 |
| Effective Impervious area (%) | 60%- (Calibrated 56%) | %60- (Calibrated 57%) |
| Length of pipe - 100mm to 350mm | 10,659,139 m | |
| Length of pipe - 350mm to 750mm | 5,443,470 m | |
| Length of pipe - 750mm to 1200mm | 758,465 m | |
| Length of pipe - 1200mm to 1750mm | 307,327 m | |
| Length of pipe - 1750mm to 2250mm | 104,102 m | |

The most important parameter for estimating urban stormwater runoff is percentage of the pervious and impervious areas. There are different types of surfaces in urban areas including Directly Connected Areas (e.g., roofs or paved areas), Indirectly Connected Areas (e.g., disconnected roofs and gardens) and pervious areas such as parklands (Ball et al., 2016). The first step in urban rainfall run-off modelling is estimating accurate imperviousness of urban catchments (Alley and Veenhuis, 1983). Before understanding the Effective Impervious Area (EIA), the Total Impervious Area (TIA), was estimated based on assumptions and neglecting depression losses (refer to Cherkaver (1975); Beard and Shin (1979) however, using TIA without considering direct connection to the stormwater network can lead to over-estimation of urban run- off volume. According to the literature review three methods can be applied to estimate the EIA including:

- Streamflow and rainfall record regression analysis if sufficient data exists.

- Typical EIA/TIA ratios from the literature.

- GIS maps to estimate TIA.

In this study and at early stages, as there was not sufficient data available for the rainfall and streamflow regression method, the EIA/TIA ratio was derived from the literature review and EIA was estimated for sub-catchments based on the GIS maps and existing land use in the study area.

- ***EIA/TIA ratio***

Several EIA values suggested in Melbourne Water's MUSIC modelling guidelines (2018) considering the catchments land use. In addition, Table 5-8 presents range of EIA/TIA for some Australian catchments based on mentioned methods. The method of derivation is also noted. Ball and Powell (1998) estimated EIA for Powells Creek in NSW. Their method undertook analysis of rainfall and runoff data as well as comparing the antecedent moisture (AMC) according to the rainfall in the days leading up to the storm event. Their results showed EIA for the total catchment area in a range of 35% to 44% depending on the AMC. A ratio of EIA/TIA was calculated to be around 74% for 26 catchments which were located in Australia, USA, Canada, UK, Japan, and Europe by Boyd et al., (1993). They applied a regression method.

Table 5-8- Ratio of EIA/TIA in some Australian catchments

| EIA to TIA | Reference (s) | Recommended for | Method of derivation |
|---|---|---|---|
| 55-65 % | Phillips et al., (2014) | Most Australian catchments | Regression |
| 74-80% | Phillips et al., (2014) | One catchment in ACT due to the higher degree of connected surfaces | Regression |
| 70-80% | Phillips et al., (2014) | used detailed mapping of different land-uses and aerial photography to estimate the TIA | GIS methods |

Dotto et al., (2009) calibrated EIA values for five catchments located in Melbourne and reported a mean range from 0.11 to 0.45. In the absence of any local streamflow and rainfall data in urban catchments, a suitable ratio for EIA/ TIA can be derived between 50% to 70% for most urban catchments (Phillips et al., 2014). For this study the ratio has been considered 56% as suggested by Phillips et al., (2014) for most Australian catchments. However, the effectiveness impervious area will be finalised after stormwater data collection and model calibration and validation.

Using aerial photos, drainage or land-use maps, cadastral information and terrain can be considered as suitable sources to estimate the TIA and EIA ratios in GIS method. Detailed land use GIS layer was used within the selected catchments to estimate the EIA. The following steps explain the estimation process:

1- First, the key land use areas are identified within one small sub-catchment separately from Catchment 1 and 2 as a sample. It includes vacant lands, green fields, residential properties boundaries, paved areas, and roads (Figure 5-21).

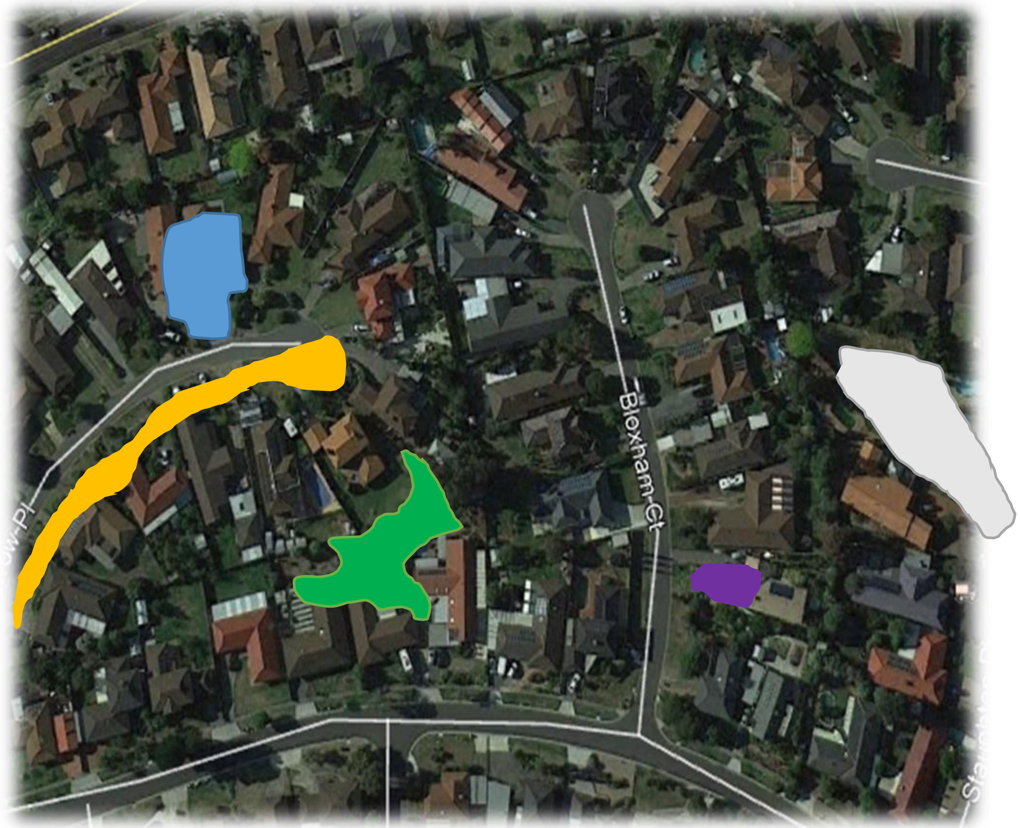


*Figure 5-21- Key land use components selected in the study area*

Legend: Paved areas Vacant lands Parks and open spaces Roads Property boundaries

2- Based on identified key land uses, the overall TIA has been calculated for the sample sizes selected from C1 and C2.

3- After calculating TIA and adopting the percentage derived for the ratio of EIA/TIA the EIA amounts were estimated for all sub-catchments.

Results of the EIA calculated according to the explained method in this section and using 56% for the EIA/TIA ratio was compared with available EIA percentage for the study area and later was adjusted by the calibration process adopting the collected data for the area.

## 5.10. Research Study Area Rainfall and Evaporation

Rainfall and evaporation data were derived from Skeleton Creek at Sayers Road Hoppers Crossing and Laverton Station (Figure 5-22 and 5-23) respectively. The rainfall and evaporation data obtained were used 6-minute and daily data respectively.

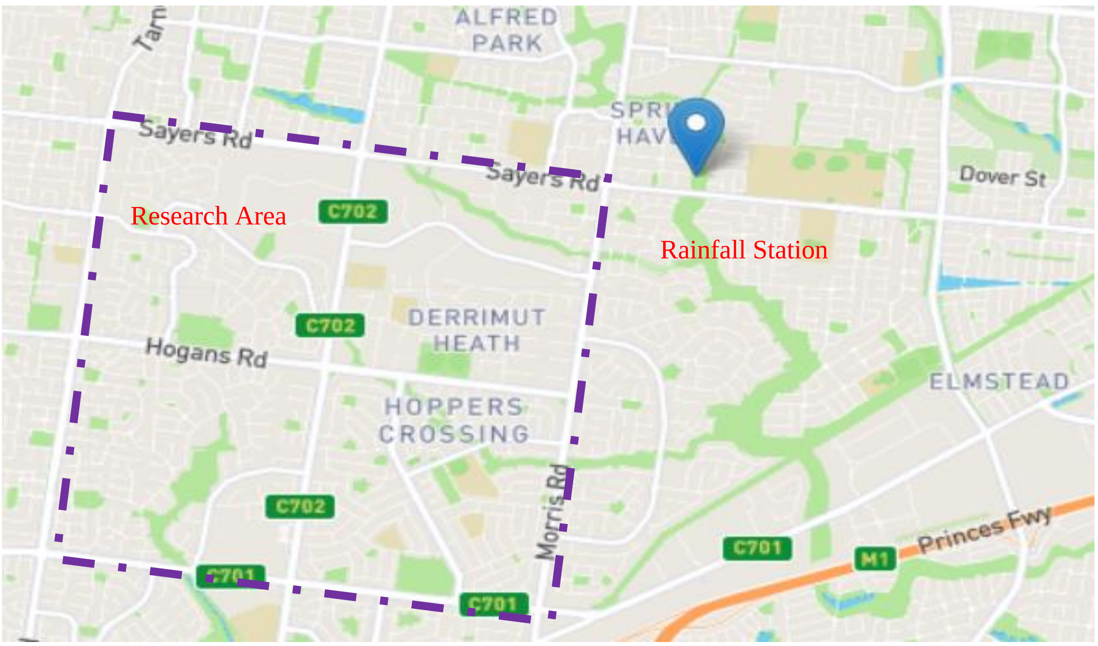


*Figure 5-22- Rainfall Station Location*

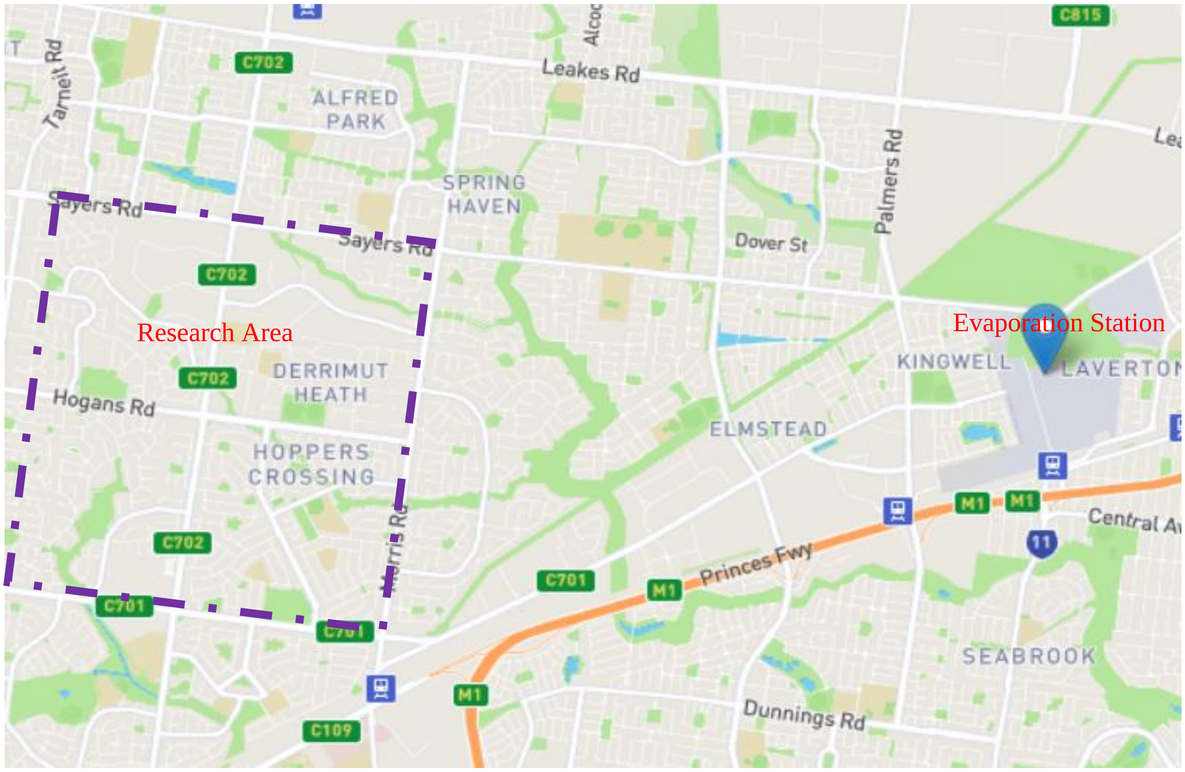


*Figure 5-23- Evaporation Station Location*

Total rainfall and evaporation for the study area and for the monitoring period (May 2019 to August 2020) was reported 594.6mm and 1515.6mm, respectively. Maximum rainfall happened in April 2020 (91.2 mm) while the maximum evaporation happened in December 2019 (228.9mm) (Figure 5-24).

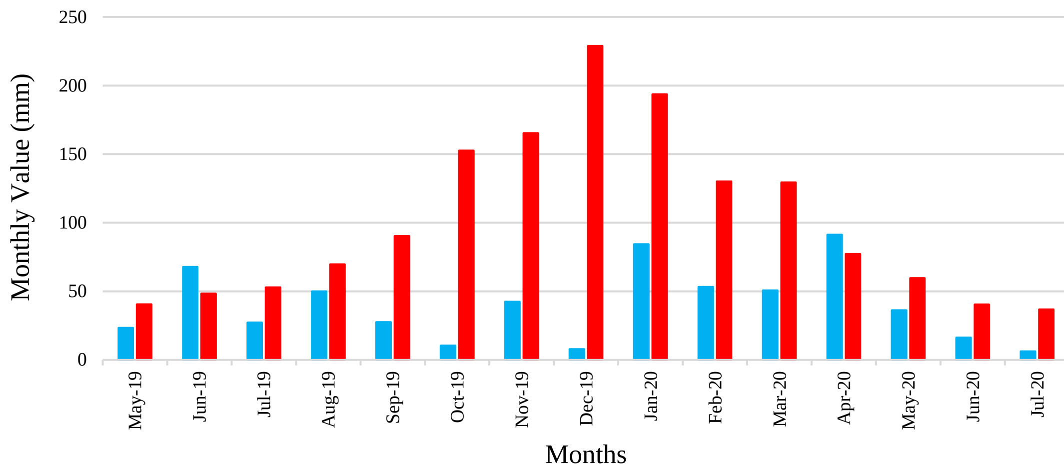


*Figure 5-24- Monthly Rainfall and Evaporation in the study area*

## 5.11. PCSWMM Models Findings in Stormwater Quality and Quantity Management

Utilizing the SWMM5 model (USEPA, 2016c), it was projected that the implementation of WSUD techniques would result in a runoff reduction of 55% to 66% for peak discharge and 25% to 121% for flow volume when compared to a scenario without the features. The effect of the techniques on reducing runoff during peak flows for return periods of 50, 80, and 100 years was assessed to be in the range of 6% to 16% for peak runoff and 33% to 37% for runoff volume.

China was confronted with the challenge of incorporating the "Sponge City" philosophy into areas developed during the construction boom from the 1990s onward. This was including regions where traditional hutongs (alleys formed by lines of single-storey residences) were replaced by high-rise apartments. Even in recently constructed facilities, there is room for improvement. Jia et al., (2012) reevaluated the design of the 36-hectare Beijing Olympic Village, consisting of 42 high-rise residential buildings. While the village initially integrated some WSUD features, such as porous pavements, roof gardens, infiltration trenches, green spaces, and rainwater tanks, the authors used the Storm Water Management Model (SWMM) (USEPA, 2016c) to demonstrate that additional WSUD techniques enhancements, including redirecting roof runoff through green spaces, increasing detention times in storage facilities, and implementing well-designed bioretention cells, could result in a 27% reduction in total runoff volume and a 21% reduction in peak flow rate compared to the original construction.

Liao et al., (2013) investigated the effects of implementing five distinct (WSUD) scenarios in a 374-hectare catchment in Shanghai, China. They utilized EPA SWMM (U.S. EPA, Cincinnati, Ohio, USA) to simulate three base scenarios, which included the predevelopment site, the post development site, and the post development site incorporating WSUD. The WSUD features comprised porous pavement, bioretention systems, infiltration trenches, rain barrels, and swales. However, the study provided limited details regarding assumed sizes and other simulation parameters for these devices. Simulations were conducted for 1-, 2-, and 5-year Average Recurrence Interval (ARI) design storms.

Jato-Espino et al., (2016) presented findings on the reduction of runoff volume and combined sewer surcharge using the EPA SWMM (U.S. EPA, Cincinnati, Ohio, USA) model for a 31-hectare catchment in Donostia, Spain. The study compared the impact of a design storm with a 10-year Average Recurrence Interval (ARI) on the catchment in its current state and for scenarios involving green roofs or permeable pavement. The simulations demonstrated that green roofs and permeable paving led to a reduction in runoff and sewer surcharge volumes by 38% and 68%, respectively. Permeable paving was particularly effective in this study for two reasons, as noted by the authors: it had a larger surface area, and it also had a greater total storage volume for collecting runoff based on the EPA SWMM data input. The study did not clarify whether there was a contributing impervious area to the permeable paving, which could also enhance its performance. As is customary, the study did not provide data on the available volume of each WSUD storage before the simulated storm.

## 5.12. PCSWMM Model Development

All required data to develop the research study area in PCSWMM model, was collected from Wyndham City Council, Melbourne Water, Southern Rural Water (SRW) and Bureau of Meteorology (BoM) as listed in Table 5-9.

Table 5-9- List of required data for the study area simulation in PCSWMM model

| No. | Required data for hydraulic and hydrological modelling |
|---|---|
| 1 | Contours and networks (in GIS layers), and1x1m DEM (derived from 2008 LiDAR) |
| 2 | Area (ha) and catchment's boundaries |
| 3 | Existing stormwater system (size of pipes and diameters, Stormwater pit's depths and invert levels) |
| 4 | Land use and soil parameters |
| 5 | Natural waterway's locations and information (e.g., D1 Drain) |
| 6 | All the existing WSUD locations and information (there is only open Gross pollutant Trap in the study area) |
| 7 | Stormwater quality and quantity data |
| 8 | Rainfall data (6 minutes steps interval) and evapotranspiration data from BoM |

In PCSWMM model, sub-catchments are assumed as nonlinear reservoirs. While the input is running, rainfall can get evaporated, become runoff or infiltrated (Rossman, 2015). The whole area was divided into 2000 sub-catchments regardless the land use and just considering the stormwater network outlets. The sub-catchments sizes varied from 0.5ha to maximum 2 ha. Interception, surface wetting and depression storage from ponding determine the sub-catchments storage capacity. Sub-catchments include impervious and pervious sub-areas. The sub-areas runoff can be routed from one to the other sub-area, and or both sub-areas can drain to an outlet (Rossman, 2015). Another key factor for stormwater runoff determination is infiltration. Runoff generates when the soil depression storage and infiltration is exceeded by water depth (Zhang, 2010). There are three available methods in PCSWMM model for infiltration calculation including Horton's equation, SCS and Green-Ampt method. For this study, Green-Ampt method was adopted as is the most recommended method. Evaporation also can be determined in the model using the area evaporation as time series or climate files. If sub-catchments route to each other, they can receive run-on from each other and therefore, sub-catchments runoff can be calculated using Manning equation to represent their flows (Rossman, 2015).

Routing is one of the benefits for using PCSWMM model and the model is able to simulate complicated routing stormwater networks. After importing stormwater pipes and nodes in the model, routing can be carried out. PCSWMM does the flow routing through the stormwater pipe network by the conservation or momentum equations. Kinematic wave routing, dynamic wave and steady flow routing approaches are included in the model to solve the routing equations. For this study, the dynamic wave routing approach was considered as is the most complex method.

## 5.13. Stormwater Model Quantity Calibration

After model development and finalizing stormwater quantity data collection task, to make reliable runoff prediction for the study area, the most suitable parameters in PCSWMM model were selected through the model calibration process. The model uses a Sensitivity-based Radio Tuning Calibration tool (SRTC). To start the calibration process, parameter's uncertainties and initial values should be specified. Model control parameters categorized into two types including measured and inferred parameters (Choi and Ball 2002). From the measured parameters, pipe diameter and roughness were considered and among the inferred parameters, catchment slope and width, impervious ratio, Manning's roughness coefficient were included. These were chosen upon the model performance and the input data files uncertainties. A description of the catchment's parameters is presented in table 5-10.

Table 5-10- Catchment's parameters descriptions and references in PCSWMM model

| Parameter | Description | Amount and Reference (s) |
|---|---|---|
| N Imperv | Manning's n for overland flow over impervious area | 0.01–0.015 (Huber et a1.1988; Sun et al., 2012) |
| N perv | Manning's n for overland flow over pervious area | 0.02–0.8 for pervious surfaces (Huber et al.,1988) |
| Dstore Imperv | Depression storage for impervious areas (mm) | 0.03-0.25 (Sun et al., 2012; Tsihrintzis & Hamid 1998) |
| Dstore Imperv | Depression storage for pervious areas (mm) | 0.25-0.5 (Sun et al., 2012; Tsihrintzis & Hamid 1998) |
| Infiltration parameters (Green-Ampt) | Suction Head: 316 mm, Conductivity (K): 0.6 mm/hr, Initial Deficit: 0.21 | (Nasrin et al., 2016, 2017) |

After selecting the parameters and assigning initial values to all of them, a percentage of uncertainty was assigned to the parameters as suggested by James et al., 2010 and considering their sources. The uncertainty percentage helped to redefine the mean using the limits of parameters as are calculated based on the Equation 5-1 and 5-2:

$$VUpper = Vcurrent \times (1 + Vf) \qquad (5-1)$$

$$VLower = VCurrent \times \left[\frac{1}{1 + Vf}\right] \qquad (5-2)$$

where

$V_{Current}$ = Value of pre calibration parameter in PCSWMM and $V_f$ = Fraction representing the percentage of variability calculated for the range. Assigned uncertainties percentages for the selected parameters are listed in Table 5-11.

Table 5-11-Assigned Uncertainty Values to Parameters in PCSWMM model

| Notation | Calibration Parameter | Uncertainty percentage (%) |
|---|---|---|
| Slope | Sub-catchment slope | 25 |
| Width | Sub-catchment width | 100 |
| Impervious percentage | Impervious percentage | 25 |
| N perv | Manning's roughness for pervious area | 25 |
| N Imperv | Manning's roughness for impervious area | 25 |
| Geom1 | Geometry and Maximum Pipe depth | 25 |
| Roughness | Pipes Manning Roughness | 25 |

For the calibration process, the collected stormwater data was used for the whole period starting from 5/13/2019 and ending on 7/14/2020. The process was done applying an event-based method and for comparisons. The parameters were adjusted for the area in the model based on the catchment's outlet maximum peaks. To assess the model performance during calibration process, the Nash-Sutcliffe coefficient (NS), Relative Error (RE), and Coefficient of Determination (R2) were used.

The ENS is a commonly used goodness-of-fit measure and is suitable for reflecting the trends and overall fit of a flow hydrograph (Coutu et al., 2012). The calculation is shown in Equation (5-3).

$$E_{NS} = 1 - \left( \frac{\sum_{i=1}^{N} \left| Q_{obs}^{i} - Q_{simu}^{i} \right|^2}{\sum_{i=1}^{N} \left| Q_{obs}^{i} - \overline{Q_{obs}^{i}} \right|^2} \right) \qquad (5\text{-}3)$$

where Qobs and Qsimu address to the observed measured flows and model simulated flows respectively. N defines the number of observations. The coefficient should be close to 1 for a good fit.

RE can be calculated using the Equation 5-4:

$$RE(\%) = \frac{Q_o - Q_S}{Q_o} \qquad (5-4)$$

Where RE (%) is the difference between the total observed and simulated runoff, Qs is simulated runoff, and Qo is the observed runoff. Results of the calibration for the study area is discussed in section 5.13.2.

### 5.13.1. Parameter Sensitivity Analysis in SRTC

Chosen parameters for calibration were checked using SRTC considering their impact on urban runoff generation. This was done to understand the effect of parameters on the catchment's outlet runoff. Among the listed parameters in Table 5-11, only sub-catchment width and impervious percentage were the sensitive ones and had the most significant impact on the runoff generation. As a result, while the sensitive parameters were adjusted frequently to achieve the best set of parameters, insensitive parameters were left unchanged.

### 5.13.2. Calibration Results

Figure 5-25 and 5-26 compared the observed flow data for both catchment 1 and 2 outlets during the calibration process. A good agreement achieved between the simulated flow and the observed flow data. Table 5-12 and 5-13 presents errors and peak values for the calibration process for both catchments. During the calibration and for catchment 2, maximum peak flow had only 9% relative error. Relative error was 12% for catchment 1. The values of R2 and NS confirmed a satisfactory model calibration however, the total volume showed -9% and -12% relative error. The minus explains that the simulated flow in the model was overestimated the flow during simulation. Model performance has shown during the calibration process in some chosen events in Figure 5-27.

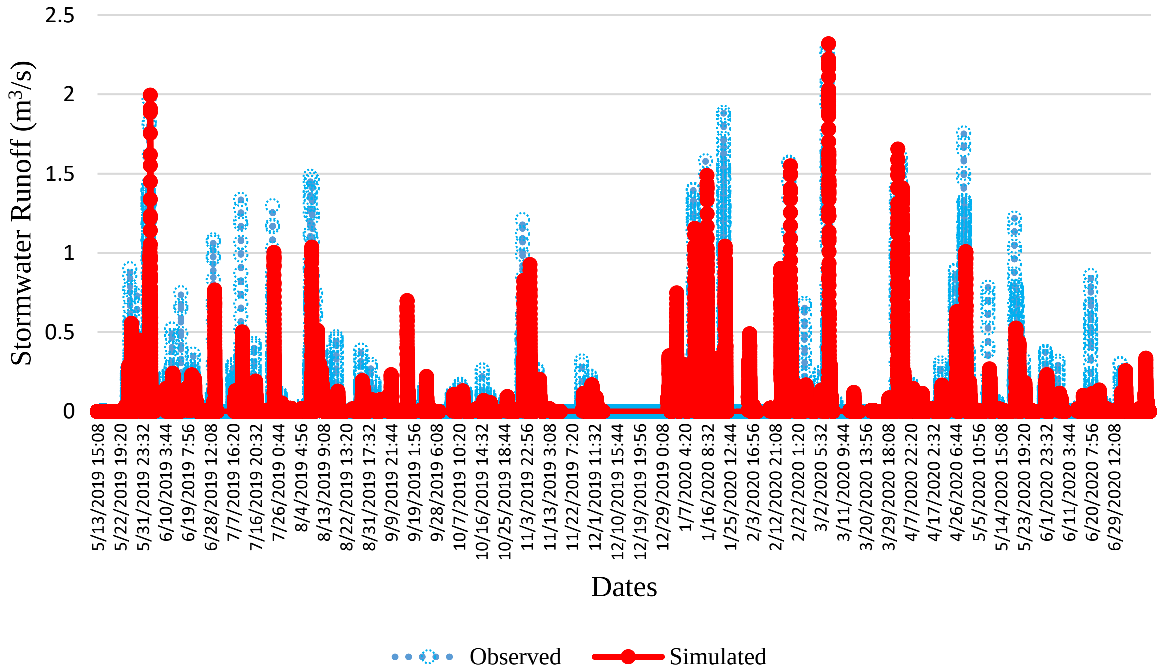


*Figure 5-25- Model Performance During the Calibration Process – Catchment 2*

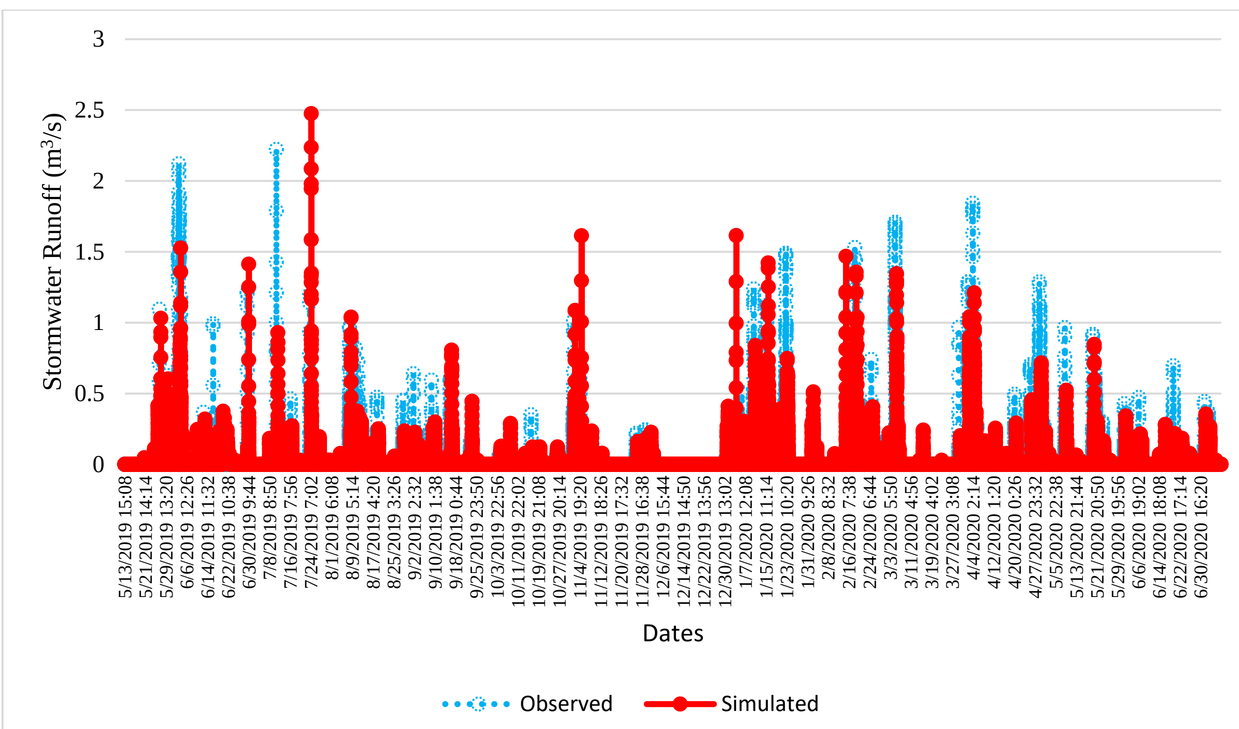


*Figure 5-26- Model Performance During the Calibration Process – Catchment 1*

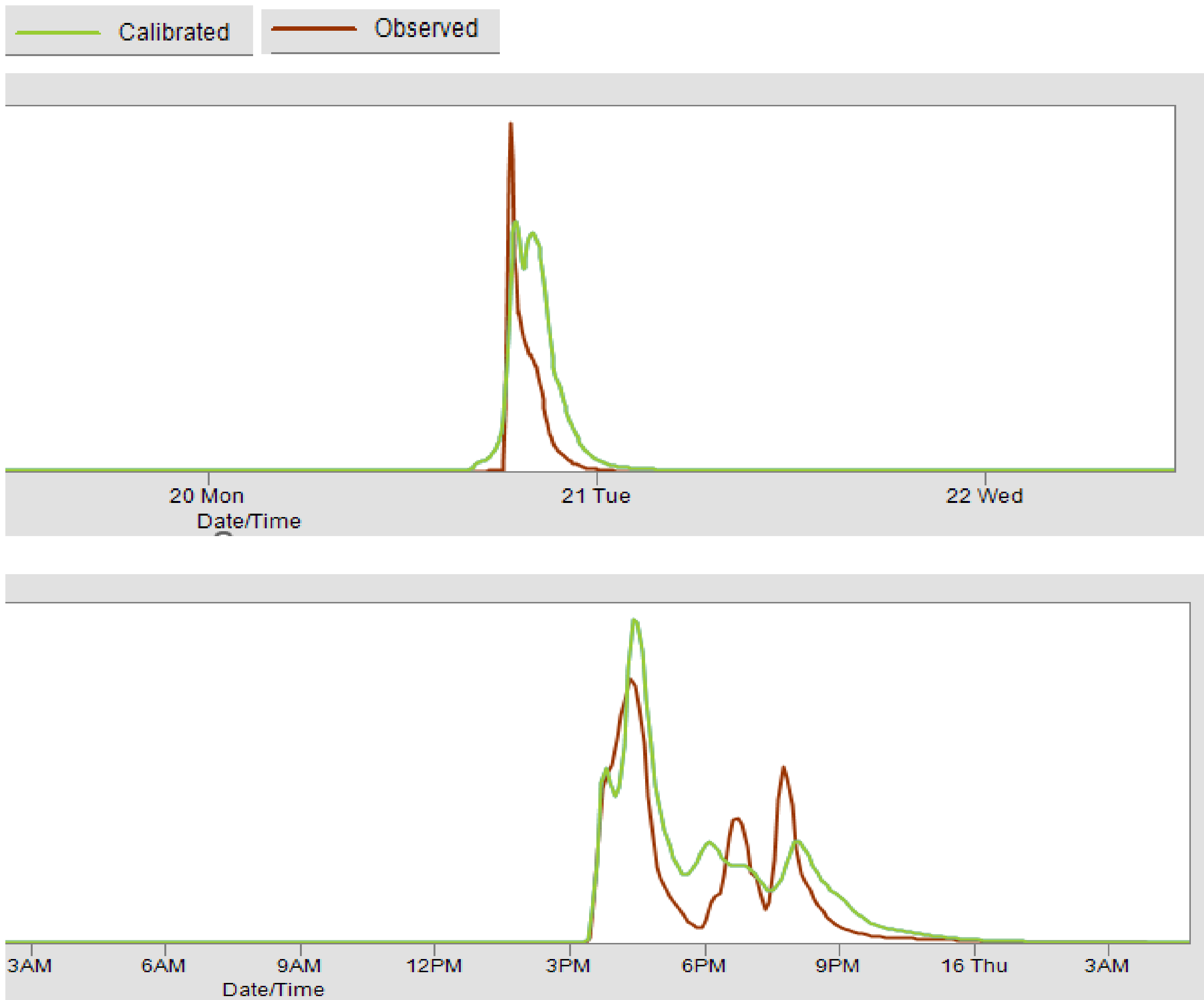


*Figure 5-27- Model Performance During the Calibration Process – Individual events*

Table 5-12-Model Performance During Calibration – Catchment 2

| Parameters | Observed | Simulated |
|---|---|---|
| Maximum Flow ($m^3/s$) | 2.29 | 2.31 |
| Minimum Flow ($m^3/s$) | 0 | 0 |
| Mean Flow ($m^3/s$) | 0.014 | 0.019 |
| Total Volume $m^3$ | 525,300 | 574,600 |
| Nash-Sutcliffe efficiency (NS) | 0.64 | |
| Coefficient of determination ($R^2$) | 0.66 | |

Table 5-13-Model Performance During Calibration – Catchment 1

| Parameters | Observed | Simulated |
|---|---|---|
| Maximum Flow ($m^3/s$) | 2.22 | 2.4 |
| Minimum Flow ($m^3/s$) | 0 | 0 |
| Mean Flow ($m^3/s$) | 0.009 | 0.012 |
| Total Volume $m^3$ | 330,900 | 370,908 |
| Nash-Sutcliffe efficiency (NS) | 0.57 | |
| Coefficient of determination ($R^2$) | 0.59 | |

The calibration process was conducted multiple times to enhance the model's accuracy, ensuring that both total runoff volume and peak flow trends were aligned with observed data. Sensitivity analysis identified sub-catchment width and impervious percentage as the most influential parameters affecting runoff volume, whereas their impact on peak flow was negligible. These parameters played a crucial role in improving the agreement between the modeled and observed total stormwater volume at the catchment outlet. Further investigation revealed that discrepancies in peak flow alignment were primarily attributed to gaps in recorded data caused by remote connection failures and potential measurement inaccuracies at high discharge levels due to sensor limitations. Given that the model was developed for long-term planning, the accuracy of total volume estimation for WSUD assessments was deemed more critical than precise short-term peak flow matching (Hosseini et al., 2020; Rossman, 2010). However, despite variations in peak flow magnitudes, the overall hydrograph shape and timing remained consistent with observed trends, reinforcing the model's reliability for long-term hydrological assessments.

## 5.14. Stormwater Model Quality Calibration

The following process of adopting the observed stormwater quality data in the stormwater modelling tools was explored for the stormwater model quality calibration, with the aim of ensuring that the simulation amounts of the model's outlets are in close proximity to the observed data.

- Music Model: The TSS concentration in the MUSIC model was adjusted using the calculated mean and standard deviation of the observed data (refer to Figure 5-19 TSS STD: 310mg/l and Mean 241mg/l) and used as input for the model to understand the amount of TSS should be used for the chosen storm events for optimization in Chapter 8.

- PCSWMM Model: the model is capable of analysing any number of water quality constituents, including accumulation, wash-off, transit, and treatment. For a stormwater pollutant quality run in the model, observed TSS was picked. Since the research area's TSS mean and standard deviation are noticeably higher than those of other Melbourne metropolitan catchments, just TSS was selected; TN and TP were not considered. This has been mentioned in section 5.8.2. as well. The quality run was done after adjusting the PCSWMM model from the quantity aspect by the sub-catchment width and impervious percentage for the urban catchments. The land use categories were altered for sub-catchments once the stormwater pollutant was selected considering the landaus abilities to generate TSS. The quality of runoff from each land use was determined by setting the parameters of the buildup and wash-off functions. The primary land use categories in the area were roads, residential areas, and open spaces. The model was run to ensure the C2 outlet stormwater quality and quantity will remain close to the observed data.

## 5.15. Conclusion

Limited stormwater quality and quantity data is available to conduct research to understand the impact of WSUD implementation in existing developed areas. Thus, there is a need to generate such data for further research. A monitoring program conducted for this research study area located in west Melbourne funded by Melbourne Water. The aim was to collect stormwater quality and quantity data for a year to calibrate and validate the stormwater modelling tools chosen for this research.

The monitoring program commenced for the research study area in May 2019 and finished in August 2020. Stormwater runoff data collected for the area every 6-minutes. Collected data was used for stormwater modelling tools calibration and validation. In total, thirty samples were collected for quality analysis during the monitoring period and for one of the catchments. Two of samples were grabbed manually and the other samples were collected by an auto-sampler. Stormwater quality analysis was done for a list of pollutant. The list was prepared according to the literature review and the most found pollutants in urban stormwater runoff. Due to the small change in pollutant range, only two samples were collected every month for rainfall events with more than 2mm.

Adopting the collected stormwater runoff data for the research study area, PCSWMM models' parameters were adjusted to represent the actual condition of the area for both quality and quantity aspects. Sensitive parameters were identified and were adjusted for the chosen catchments calculating Nash-Sutcliffe efficiency (NS) and Coefficient of determination (R2). During the calibration and for catchment 2, maximum peak flow had only 9% relative error. Relative error was 12% for catchment 1. Radio tuning was done to make the maximum peaks closer for events throughout the whole monitoring period. MUSIC model was not calibrated in this research and just was used for a few purposes like WSUD effectiveness on stormwater quantity at an allotment and stormwater quality concentration validation at the catchment scale in the study area. This is because the MUSIC model can simulate stormwater pipe to an extent and is limited to use Muskingum- Cunge routing approach and only considers travel time for pipe's network.

In order to make sure the land use data has been appropriately adjusted for the research study area to reflect the catchment's outlet TSS amount for Catchment 2, the quantity data collected for stormwater was used for both PCSWMM model calibration and a quality run in both MUSIC and PCSWMM models. Urban catchments comparison of TSS, literature review, and storm water quality data analysis led to the decision to include this pollutant in the catchment scale optimization challenge in chapter eight. Since their median quantity was so low in relation to Melbourne's other urban catchments, TN and TP were disregarded.

# 6. Chapter 6: Stormwater Scenario Modelling

## 6.1. Introduction

The effectiveness of WSUD assets at different scales to manage urban stormwater quantity and quality is still uncertain. Prior to understand the optimal location of the techniques at a developed catchment and the optimisation task it was essential to understand their impact on peak size and volume reduction at different scale as they can be implemented. This chapter has investigated the extent to which various WSUD approaches can reduce stormwater runoff volume and peak flow size for a typical allotment, chosen sub-catchment and the whole study area.

## 6.2. Stormwater Scenario Modelling at Allotment Scale

### 6.2.1. Chosen WSUD Techniques

- ***Rainwater Tanks:***

Rainwater tanks are an ancient practice in stormwater management and remain relevant to modern cities (Gomes et al., 2012). These systems collect and store rainwater to reuse as primary or alternative water sources (Fewkes 2012). Capital cities in Australia, households experienced an increase in the proportion of rainwater tanks installation during the Millennium drought with uptake reported at 15% in 2007 and 28% in 2010 (Australian Bureau of Statics 2010). For the scenario modelling in this study the percentage of house for adopting rainwater tanks and raingardens, was assumed to be 28%, refer to the Australian Bureau of Statistics report 2010. According to the available guidelines and literature review the likely tank sizes are 1 kL, 2 kL, 3 kL, 4 kL and 5 kL. Under a survey which has been done by CSIRO in Melbourne (Moglia et al., 2014) the most adopted tank sizes are 2 to 3 kL and the average size is 4.3 kL. Therefore, and for this study 3 kL and 5 kL tanks selected for this study for scenario modelling. A typical rainwater tank is shown in Appendix A.

- ***Bio-retention or Raingardens***

Bio-retentions: Bio-retentions or raingardens are devices composed of media (sand, gravel, loam) and trees or plants. They are suitable for storage, infiltration and evaporation of both direct rainfall and the runoff captured from surrounding areas and convey the excess stormwater to a pervious area. Raingardens are effective in stormwater quantity management by reducing runoff (Yang et al., 2009; Trowsdale and Simcock 2011) and providing stormwater treatment.

Rain gardens can be considered as suitable lot scale systems for stormwater management purposes. Burns et al., 2014, suggested a combination of rainwater tanks for reuse and rain gardens with infiltration for the management of flow regime at small scales. However, further research is required to investigate as how to achieve flow regime management at both catchment and land parcel scales. Raingardens are depressions which contain vegetation in an engineered soil mixture placed above a gravel drainage bed. They are suitable to store, infiltrate and evaporate of both direct rainfall and runoff captured from surrounding areas and convey the excess stormwater to the pervious area. A typical raingarden is shown in Appendix A.

- ***Vegetated Swales***

A vegetated swale is a grass-lined channel with a flat bottom which receives stormwater runoff flow via its slope's sides (Davis et al., 2012). These shallow channels have shown to be effective in stormwater quantity management. In two separate studies, around 47% and 45% reduction of mean annual runoff volume was reported by adopting swales (Ackerman and Stein 2008; Barrett 2005). A typical swale is shown in Appendix A.

- ***Permeable Pavements***

Permeable pavements reported as an efficient tool to manage flooding risk as they allow infiltration and evaporation (Freni et al., 2010; Ullate et al., 2011; Beecham et al., 2012; Argue 2013). They consist of a permeable pavement top layer and an underdrain layer. The permeability of the top layer can vary from tens to thousands of millimetres per hour (Bean et al., 2007; Kuang et al., 2011). Stormwater runoff can be drained over the top layer of the permeable pavement even with a fast rainfall with a 100 mm/hr intensity (Hsieh and Chen 2012). A typical permeable pavement is shown in Appendix A.

- ***Infiltration Trench***

Infiltration trenches can capture stormwater runoff via a filter media such as stone or gravel and infiltrate into surrounding soils and underlaying groundwater a (Siriwardene et al., 2007). In a study conducted by Yazdi and Scholz (2008), 73% and 80% reduction of mean flow volume and peak flow was observed by adopting infiltration systems. A typical infiltration trench is shown in Appendix A.

MUSIC model and PCSWMM model were used for scenario modelling at an allotment scale for this study. Specifications of the selected techniques as were adopted in the models are presented in Table 6-1 and 6-2. Several local factsheets, guidelines and procedures have been developed in Australia and other countries that suggest appropriate ranges for the key parameters of WSUD techniques (Argue 2013; Melbourne Water 2013; Rossman 2015). Key parameters for the considered WSUD assets in this study were assigned according to the stormwater modelling tools user’s manuals, existing guidelines, and literature review (Table 6-1 and 6-2).

Table 6-1- Asset's specifications and input values as adopted in the MUSIC model (MUSIC model manual guideline, Melbourne Water 2018).

| **Parameters** | **3-kL Rainwater tank -Input values** | **5-kL Rainwater tank -Input values** |
|---|---|---|
| Volume (kL) | 3 | 5 |
| Initial volume (kL) | 1 | 1 |
| Surface area ($m^2$) | 3 | 5 |
| | **Infiltration trench – Input values** | |
| Extended Detention Depth (m) | 0.01 | |
| Pond surface area ($m^2$) | 6 | |
| Filter area ($m^2$) | 6 | |
| Filter media perimeter (m) | 10 | |
| Exfiltration rate(mm/h) | 0.36 | |
| Depth of infiltration (m) | 0.5 | |
| Width (m) | 2 | |
| Length (m) | 3 | |
| | **Raingarden – Input values** | |
| Extended DD (m) | 0.3 | |
| Berm Hight (m) | 0.3 | |
| Surface area ($m^2$) | 7 | |
| Filter area ($m^2$) | 7 | |
| Filter media perimeter (m) | 12.3 | |
| Exfiltration rate(mm/h) | 0.36 | |
| Filter depth (m) | 0.5 | |
| Length (m) | 5.5 | |
| | **Vegetated swale – Input values** | |
| Berm Hight (m) | 0.2 | |
| Vegetation height (m) | 0.8 | |
| Bed slope (%) | 3 | |
| Length (m) | 5 | |
| Top width (m) | 3 | |
| Base width (m) | 2 | |
| Depth (m) | 0.5 | |

Table 6-2- Asset's specifications and input values adopted for the PCSWMM model- (Baek et al., 2015; James et al., 2010)

| | Parameters | 3-kL Rainwater tank -Input values |
|---|---|---|
| | Volume (kL) | 3 |
| | Width (m) | 0.8 |
| | Hight (m) | 2 |
| | Surface area ($m^2$) | 2 |
| | | **5-kL Rainwater tank -Input values** |
| | Volume (kL) | 5 |
| | Width (m) | 1 |
| | Hight (m) | 3 |
| | Surface area ($m^2$) | 3.14 |
| | | **Infiltration trench -Input values** |
| | Berm height (m) | 0.2 |
| | Vegetation volume | 0.15 |
| | Storage thickness (m) | 0.3 |
| | Drain coefficient | 3 |
| | Void ratio | 0.6 |
| | Seepage rate (mm/hr) | 0.36 |
| | Surface slope | 1 |
| | | **Raingarden -Input values** |
| | Berm height (m) | 0.3 |
| | Vegetation volume | 0.6 |
| | Storage thickness (m) | 7 |
| | Void ratio | 12.3 |
| | Seepage rate (mm/hr) | 0.36 |
| | Conductivity (mm/hr) | 100 |
| | | **Vegetated swale – Input values** |
| | Berm height (m) | 0.3 |
| | Vegetation volume | 0.8 |
| | Surface roughness | 0.015 |
| | Surface slope | 3 |
| | Side slope | 5 |
| | Top width (m) | 2 |
| | Length (m) | 5 |
| | | **Permeable pavement- Input values** |
| **Surface** | Berm Hight (m) | 0.04 |
| | Surface slope % | 2 |
| | Surface roughness | 0.015 |
| **Pavement** | Thickness (mm) | 80 |
| | Void ratio | 0.4 |
| | Permeability (mm/hr) | 4000 |
| **Soil** | Thickness (m) | 0.5 |
| | Porosity fraction | 0.5 |
| | Field capacity | 0.28 |
| | Conductivity (mm/hr) | 100 |
| **Storage** | Thickness | 0.45 |
| | Void ratio | 0.4 |
| | Seepage rate (mm/hr) | 0.36 |
| | Clogging factor | 0 |
| **Underdrain** | Drain coefficient (mm/hr) | 12 |
| | Drain exponent | 0.5 |
| | Drain offset height (mm) | 50 |

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### 6.2.2. Understand the size of components of a typical allotment in the study area

After selecting a suitable study area in Melbourne and to explore the effectiveness of WSUD assets at a typical allotment on stormwater runoff and peak flow size, the first step was to understand the typical allotment layout and components in the study area. According to the available Geographical Information System (GIS) layers and exploring residential properties, the average size of the allotment's components was identified (Appendix B).

The average size of a typical property in the area for the current condition presented in Table 6-3 and was included roof, paved area and driveway, and garden areas as shown in Figure 6-1. The reported average allotment size in the area was 3 to 5 people (2016 census data). For future scenario modelling and to reflect the urbanisation impact and population growth, the typical allotment was divided into two townhouses as depicted in Figure 6-1. This assumption was made based on the comment development pattern in local government areas across Melbourne. Looking into subdividing process across the Melbourne, it was understood that there has been a consistent trend of landowners and developers subdividing large lands to accommodate multiple dwellings (at least two), driven by increasing urban density and housing demand. Given this trend, it was reasonable to anticipate that the subject land could be subdivided into at least two dwellings in the future, aligning with common development practices in the region." The average size of the household components for the future condition is listed in Table 6-3. Future roof area was provided for two dwellings.

Table 6-3- Components of a typical allotment in the study area

| **Total area ($m^2$)** | **Roof area ($m^2$)** | **Paved area and driveway ($m^2$)** | **Garden and lawn area ($m^2$)** |
|---|---|---|---|
| Existing - 495 | 240 (49%) | 80 (16%) | 175 (35%) |
| Future - 495 | 300 (60%) | 90 (18%) | 105 (22%) |

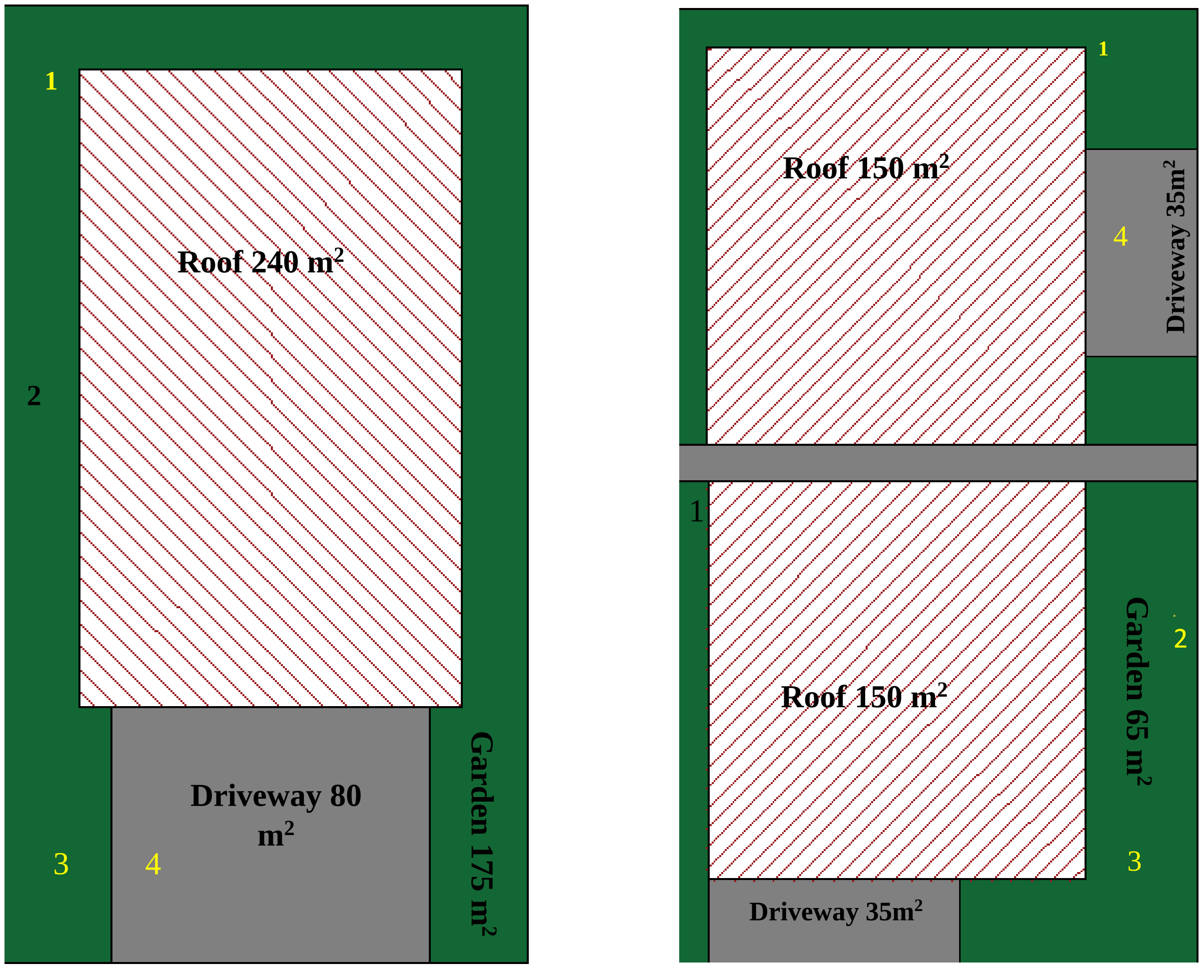


*Figure 6-1- A typical allotment current and future condition components*

1 Tank 2 Swale- Infiltration trench 3 Raingarden 4 Permeable pavement

The placement of the adopted techniques within the allotment under existing and future condition is indicated by numbers from 1 to 4.

### 6.2.3. Define suitable stormwater management target

The main aim of this chapter was to understand the reduction in overall stormwater runoff volume and peak flow with the implementation of various WSUD assets at a residential allotment scale under different development conditions. There is also a current requirement in Australia for the 1.5-year ARI flow to be reduced to pre-development levels for the purposes of protecting waterway health (The Best Practice Environmental Management Guidelines: Stormwater (Victoria Stormwater Committee, 1999)). Therefore, the stormwater target was set to achieve the highest reduction percentage for the selected allotment under the future condition to maintain the pre-development condition. Mean annual flow and peak size were used as indicators to be compared for the current and future condition scenario modelling. Reductions were compared for different ARI.

### 6.2.4. Development of scenarios for WSUD assets assessment

Scenarios were developed for selected techniques separately and in combination for current and future development condition. First, the current residential allotment condition was considered as a base case. Then, the selected techniques utilised, and scenarios were compared with the base condition. Later, the existing dwelling was divided into two townhouses to represent the future condition and was considered as the future condition base case. Again, techniques utilised, and scenarios were compared with the base case for the future impervious ratio. The mean annual runoff volume and peak flow size reduction were compared for the developed scenarios in MUSIC and PCSWMM models—list of investigated scenarios for current and future development considerations presented in Table 6-4. The treated percentage for both current and future conditions scenarios was determined in accordance with the local council's WSUD requirements. This ensures effective stormwater management within the allotment, aligning with regulatory standards and sustainability objectives for new developments at urban catchments.

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Table 6-4- Investigated scenarios for the selected allotment for current and future conditions

| No | System | Current scenarios | Future scenarios |
|---|---|---|---|
| 1 | *3-kL rainwater tank (3-kL T) | - 49% of the allotment's area (roof) was connected to the tank<br>- Usage was assumed 206 l/day for laundry, garden irrigation and toilet flush | - 60% of the allotment's area (roofs) was connected to two tanks<br>- Usage was assumed 226 l/day for laundry, garden irrigation and toilet flush per tank |
| 2 | *5-kL rainwater tank (5-kL T) | - 49% of the allotment's area (roof) was connected to the tank<br>- Usage was assumed 206 l/day for laundry, garden irrigation and toilet flush | - 60% of the allotment's area (roofs) was connected to two tanks<br>- Usage was assumed 226 l/day for laundry, garden irrigation and toilet flush per tank |
| 3 | 0.5-meter depth Infiltration trench (IT) | - 49% of the allotment's area (roof) was connected to the trench | - 60% of the allotment's area (roofs) was connected to two trenches |
| 4 | 0.5-meter depth Rain garden (RG) | - 49% of the allotment's area (roof) was connected to the raingarden | - 60% of the allotment's area (roofs) was connected to two raingardens |
| 5 | Vegetated swale (VS) | - 49% of the allotment's area (roof) was connected to the swale | - 60% of the allotment's area (roofs) was connected to two swales |
| 6 | Permeable pavement (PP) | - 49% of the allotment's area (roof) was connected to the permeable pavement | - 60% of the allotment's area (roofs) was connected to two permeable pavements |
| 7 | Combination of a 3kL-tank and a rain garden (COM1) | - 49% of the allotment's area (roof) was connected to the tank<br>-Usage was assumed 206 l/day for laundry, garden irrigation and toilet flush)<br>- Hhousehold's' driveway and paving (16%) was connected to the raingarden | - 60% of the allotment's area (roofs) was connected to two tanks<br>-Usage was assumed 226 l/day for laundry, garden irrigation and toilet flush)<br>- Hhousehold's' driveway and paving (18%) was connected to two raingardens |
| 8 | Combination of a 3kL-tank and an infiltration trench (COM2) | - 49% of the allotment's area (roof) was connected to the tank<br>-**Usage was assumed 206 l/day for laundry, garden irrigation and toilet flush)<br>- Hhousehold's' driveway and paving (16%) was connected to the infiltration trench | - 60% of the allotment's area (roofs) was connected to the tank<br>-**Usage was assumed 226 l/day for laundry, garden irrigation and toilet flush)<br>- Hhousehold's' driveway and paving (18%) was connected to two infiltration trenches |
| 9 | Combination of a rain garden and an infiltration trench (COM3) | - 49% of the allotment's area (roof) was connected to the raingarden<br>- Hhousehold's' driveway and paving (16%) was connected to the infiltration trench | - 60% of the allotment's area (roofs) was connected to two raingardens<br>- Hhousehold's' driveway and paving (18%) was connected to two infiltration trenches |
| 10 | Combination of a 3-kL tank and a rain garden and an infiltration trench (COM4) | - 49% of the allotment's area (roof) was connected to the tank and the raingarden<br>- Hhousehold's' driveway and paving (16%) was connected to the infiltration trench | - 60% of the allotment's area (roofs) was connected to two raingarden and two tanks<br>- Hhousehold's' driveway and paving (18%) was connected to two infiltration trenches |

* An additional scenario was also considered for the 3-kL and 5-kL rainwater tanks with no usage. Also, no usage assumed for rainwater tanks in PCSWMM model for combination scenarios

### 6.2.5. Stormwater models development and application

The selected residential allotment was modelled as a single node in the stormwater models representing pervious and impervious ratios. The most important parameter to estimate the stormwater runoff is the estimation of the pervious and impervious ratios (Alley and Veenhuis, 1983). There are different types of surfaces in an urban area including Directly Connected Areas (DCA) (e.g., roofs or paved areas), Indirectly Connected Areas (ICA) (e.g., disconnected roofs and gardens) and pervious areas such as parklands (Ball et al., 2016). Given the different connected, disconnected, and green areas of the selected allotment, the ratios of directly connected impervious (roof), pavement and pervious areas were calculated as 49%, 16% and 35% respectively for the current condition and 60%, 18% 21% for the future allotment condition.

Stormwater models are developed, incorporating several parameters representing urban catchments components. Therefore, to ensure reliable models' predictions, the most effective and sensitive parameters in which would affect the model's results were identified for both models through a sensitivity analysis. These were included the catchments width, impervious fraction, soil conductivity, depth of depression storage on the impervious area for the PCSWMM model and soil storage capacity and field capacity for the MUSIC model. Initial values were assigned in the models according to the literature review. Rainfall data was adopted from the closest rainfall station to the area. Table 6-5 and 6-6 presented the adjusted key parameters and resources for the PCSWMM and MUSIC models in this study.

Table 6-5- Key parameters adjusted for the typical allotment in PCSWMM model

| **Parameters** | **Description** | **Initial values – recommended by the literature or calculated** | **Final values** |
|---|---|---|---|
| Impervious fraction (%) | The ratio of the impervious section at the allotment | 64 % for current and 80% for future condition | 64 and 80 % |
| N Imperv | Manning's n for overland flow over the impervious area | 0.01–0.015 (Huber et a1.1988; Sun et al., 2012) | 0.015 |
| N perv | Manning's n for overland flow over the pervious area | 0.02–0.8 for pervious surfaces (Huber et al.,1988) | 0.2 |
| Dstore Imperv | Depression storage for impervious areas (mm) | 0.03-0.25 (Sun et al., 2012; Tsihrintzis & Hamid 1998) | 0.05 |
| Dstore Imperv | Depression storage for pervious areas (mm) | 0.25-0.5 (Sun et al., 2012; Tsihrintzis & Hamid 1998) | 0.35 |
| Infiltration parameters (Green-Ampt) | Suction Head: 316 mm, Conductivity (K): 0.6 mm/hr, Initial Deficit: 0.21 | (Nasrin et al., 2017; PCSWMM model manual) | Suction Head: 100 mm, Conductivity (K): 0.36 mm/hr, Initial Deficit: 0.21 |

Table 6-6- Key parameters adjusted for the typical allotment in the MUSIC model

| **Parameters** | **References** | **Final values** |
|---|---|---|
| Area (ha) | Available GIS layer | 495 $m^2$ |
| Zoning | WCC MUSIC guideline (Mixed area) | Residential |
| EIA/TIA (impervious and pervious percentage) | Calculated for the household for the current condition | 64 and 80% |
| Soil Storage Capacity (SSC) | Initial values from WCC MUSIC guideline | 120 |
| Soil Initial Storage (IS) | Initial values from WCC MUSIC guideline | 25 |
| Field Capacity (FC) | Initial values from WCC MUSIC guideline | 65 |

Another key parameter for the MUSIC model scenario modelling was rainwater tank usage calculation. Current water usage was estimated for the allotment using Yarra Valley Water (Ghobadi et al., 2013) and Wyndham city information and available reports. By assuming 3 people as average household size, the total household's Usage was calculated approximately 413 L/P/day for indoor and outdoor requirements. A typical household water usage is shown in Figure 6-2 for the study area. Two assumptions were considered in the MUSIC model and to allocate the usage demand for modelling rainwater tanks for current and future condition scenario modelling:

- Toilet flushing usage plus laundry and a garden requirement for current household usage for 3-kL and 5-kL tanks as 206 L/day.
- Toilet flushing usage plus laundry and a garden requirement for future household usage for 3-kL and 5-kL tanks as 226 L/day by 10% increase for the future demand.

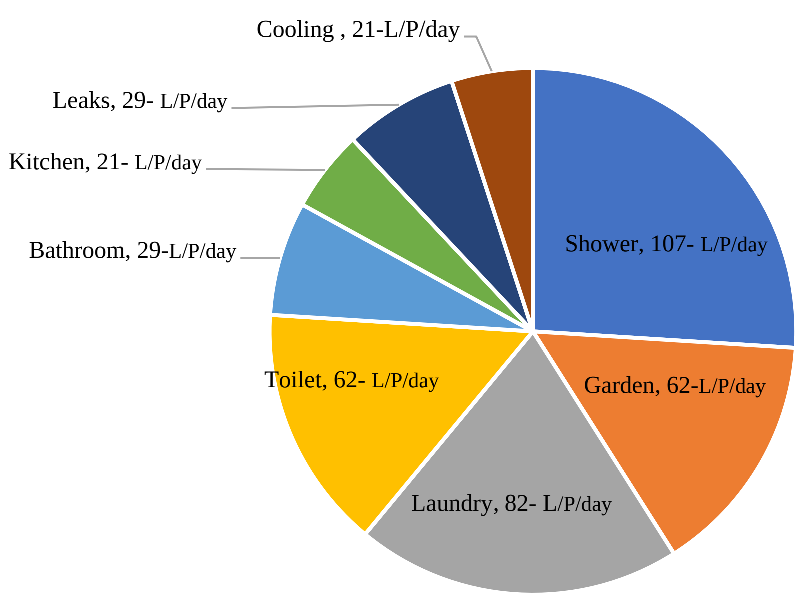


*Figure 6-2- Typical water usage for existing lots of the study area (Wyndham City Council water need, 2018)*

### 6.2.6. Scenario Modelling Results

Scenarios were simulated and run in MUSIC and PCSWMM models, as explained in Table 6-4 for selected assets individually and in combination. All the investigated scenarios were compared with a benchmark which was a base case scenario with no WSUD system under current and future development condition at the allotment. Results were assessed for mean annual flow and peak flow size reduction.

#### 6.2.6.1. Mean Annual Flow Reduction

Reduction of mean annual flow was considered in this study as one of the stormwater management indicators and for results comparison. The mean annual flow for an allotment was calculated 112 KL/year and 132 KL/year respectively for current and future condition at the selected allotment which showed at least 17% increase due to the higher impervious ratio for the future development condition. Scenarios were simulated and the results presented for mean annual flow reduction in Figure 6-3 and 6-4 for MUSIC and PCSWMM models separately. In the MUSIC model, scenarios with a combination of a 3-kL tank, a rain garden and an infiltration trench showed the most reduction percentage for the mean annual flow at the selected allotment under both current and future conditions. The least reduction percentage was achieved with the vegetated swale scenario for both current and future conditions.

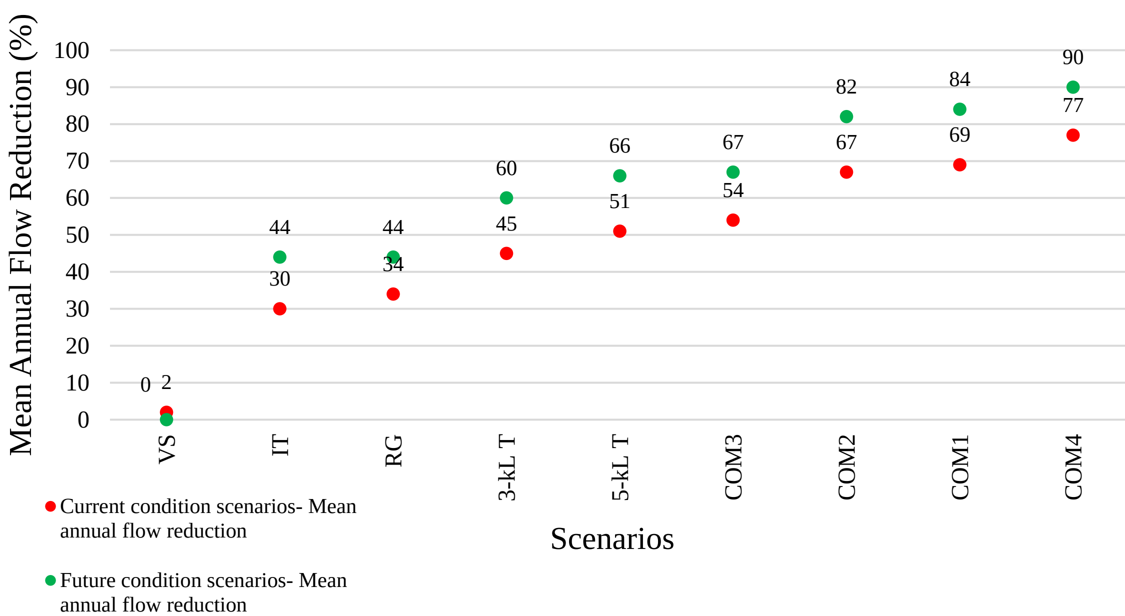


*Figure 6-3- Mean annual flow reduction percentage for current and future condition scenarios-MUSIC model*

*MUSIC doesn't represent evapotranspiration from a swale or the soil underlying it so may underestimate flow volume reductions

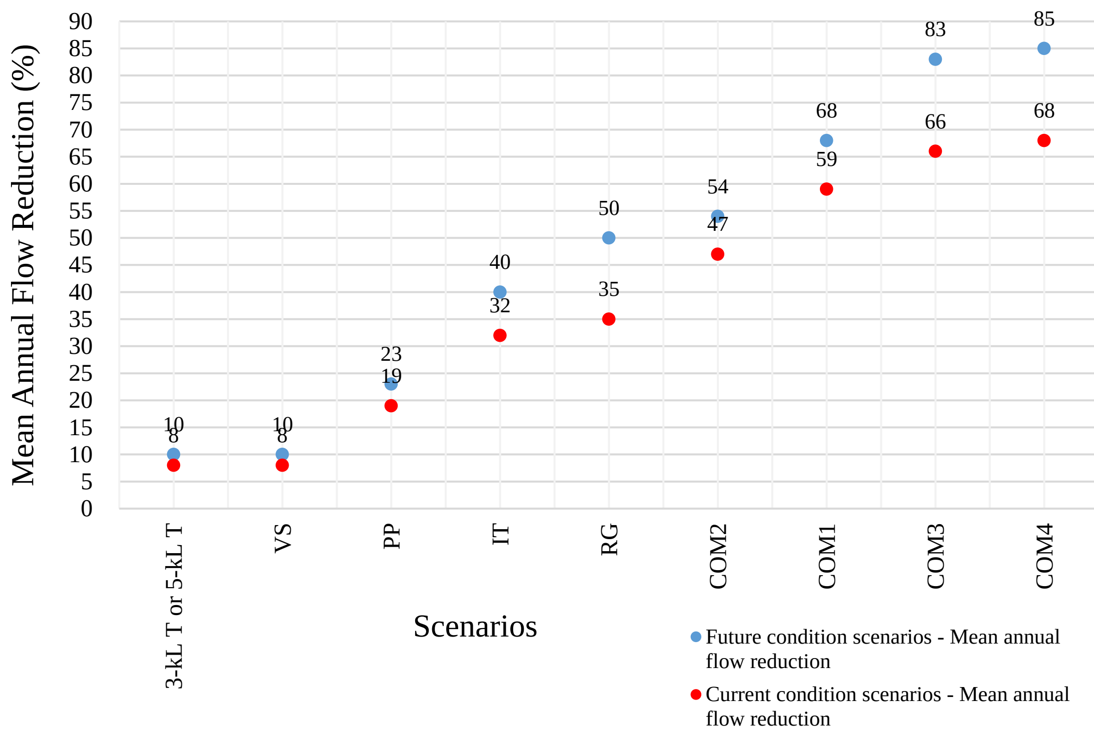


*Figure 6-4- Mean annual flow reduction percentage for current and future condition scenarios-PCSWMM model*

Similar to the MUSIC model results, the most reduction percentage for the mean annual flow at the selected allotment was achieved by a combination of a 3-kL tank, a rain garden and an infiltration trench under both current and future conditions in PCSWMM model. However, the least reduction percentage was achieved with 3-kL and 5-kL rainwater tanks and no usage. The MUSIC model results confirmed the low reduction range of mean annual flow as the PCSWMM model results for rainwater tanks and no usage.

#### 6.2.6.2. Peak Flow Size Reduction in MUSIC model

One appropriate method to understand a catchment's behaviour in terms of a flood is to undertake a partial or annual series analysis (Ball et al., 2016). The partial series is preferable for events smaller than 10% Annual Exceedance Probability (AEP); however, annual series is preferable for larger events. The partial series analysis method constructs an empirical estimate of the relationship between peaks magnitude and Annual Recurrence Interval (or the average number of exceedances per year (EY)). This method was applied in this study to find independent peaks above a threshold for the selected household in the MUSIC model. Then a suitable probability model was fitted to the selected peaks.

Figure 6-5 and Table 6-7 show the results of the current condition scenario modelling for the selected WSUD assets in the MUSIC model. Based on the results, 3-kL rainwater tank decreased the probability of peak flow exceedance for the household a minimum of up to 18% and a maximum of up to 34%. The 5-kL rainwater tank decreased this probability between 28% and 42%. Both 3-kL and 5-kL tanks with no household usage showed a different behaviour and a reduction range for different events. They showed a greater range of reduction for bigger events when there was no usage at the household. A 3-kL tank showed effectiveness to reduce the probability of peak flow exceedance for events greater than 5 years up to 10% and a 5-kL tank reduced the probability of peak flow exceedance for events greater than 5 years up to 18%.

Table 6-7- Range of peak size reduction by each system for the current condition scenario modelling in MUSIC model

| System | Reduction range for 1 to 2-year events | Reduction range for 2 to 5-year events | Reduction range for > 5-year events |
|---|---|---|---|
| 3-kL Tank | Min 26% to max 34% | Min 22% to max 26% | Min18% to max 21% |
| 5-kL Tank | Min 35% to max 42% | Min 32% to max 35% | Min 28% to max 32% |
| 3-kL Tank – no usage | Min 3% to max 4% | Min 4% to max 5% | Min 5% to max 10% |
| 5-kL Tank – no usage | Min 3% to max 8% | Min9% to max 13% | Min 13% to max 18% |
| IT – 1m depth | Min 16% to max 27% | Min 11% to max 16% | Min 5 to max 10% |
| RG – 0.5m depth | Min 50% to max 70% | Min 40 to max 50% | Min 32 to max 40% |
| VS –0.5meter length | Min 15 to max 22% | Min 12 to max 15% | Min 10 to max 12% |

Rain gardens decreased the probability of peak flow exceedance for the simulated period between 32% to 70%. Infiltration trench and vegetated swale also showed a very close range of reduction minimum up to 5% and maximum up to 27%.

In summary, the lowest reduction percentages were achieved for bigger events with greater ARI. All the assets showed more effectiveness in reducing peak flows exceedance probability for smaller events (ARI =1 to 5 years) compared to bigger events (ARI > 5 years). Comparing the individual assets reduction range, 3-kL tank with no usage, 5-kL tank with no usage, infiltration trench, vegetated swale, 3-kL tank with Usage, 5-kL tank with usage and rain garden showed the least to the most reduction percentage range for different rainfall events respectively.

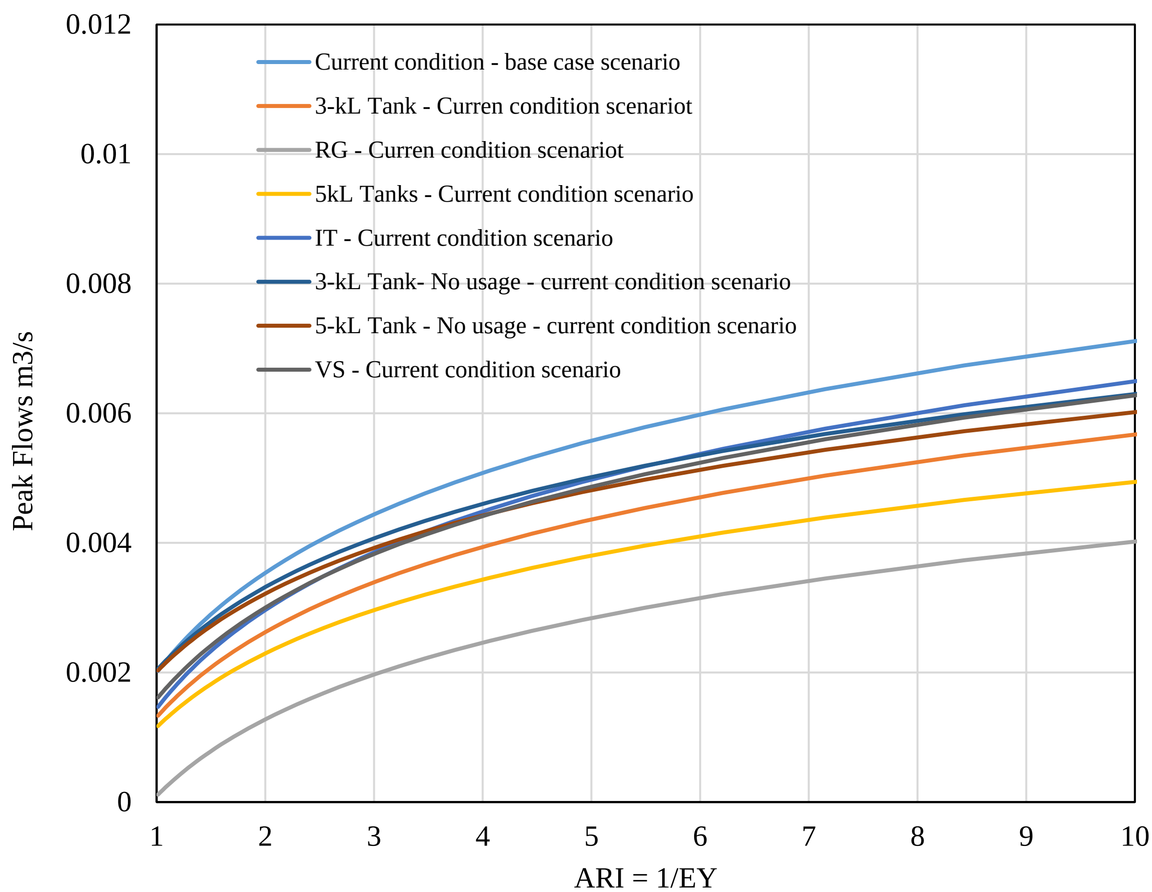


*Figure 6-5- Fitted probabilities for possible peak flows: scenario comparisons under the current condition and selected assets*

#### 6.2.6.3. Combination of assets under the current household condition

Figure 4-6 and Table 4-8 show the results of the current condition scenario modelling for a combination of a 3-kL rainwater tank, a rain garden or an infiltration trench on peak flow probability reduction. According to the results, the combination of a 3-kL tank and a rain garden showed a very close range of reduction as a combination of a 3-kL tank and an infiltration trench minimum up to 27% and maximum up to 51%. Maximum reduction was achieved in the COM3 and COM4 scenarios up to 100% for very small events with an ARI < 1.5 year and up to 50% to 60% for events with ARI> 5 years. Combination of three assets-maintained peaks discharge for the 1.5-year ARI and close to predevelopment levels at the household under current impervious ratio.

Table 6-8-Range of reduction for COM1 to COM4 assets for current condition scenario modelling

| System | Reduction percentage rage for events 1 to 2 years | Reduction percentage range for events 2 to 5 years | Reduction percentage range for events > 5 |
|---|---|---|---|
| COM1 | Min 42% to max 45% | Min 38% to max 42% | Min 35% to max 37% |
| COM2 | Min 39% to max 51% | Min 33% to max 38% | Min 27% to max 32% |
| COM3 | Min 70% to max 100% | Min 60% to max 75% | Min 37% to max 55% |
| COM4 | Min 80% to max 100% | Min 65% to max 85% | Min 50% to max 60% |

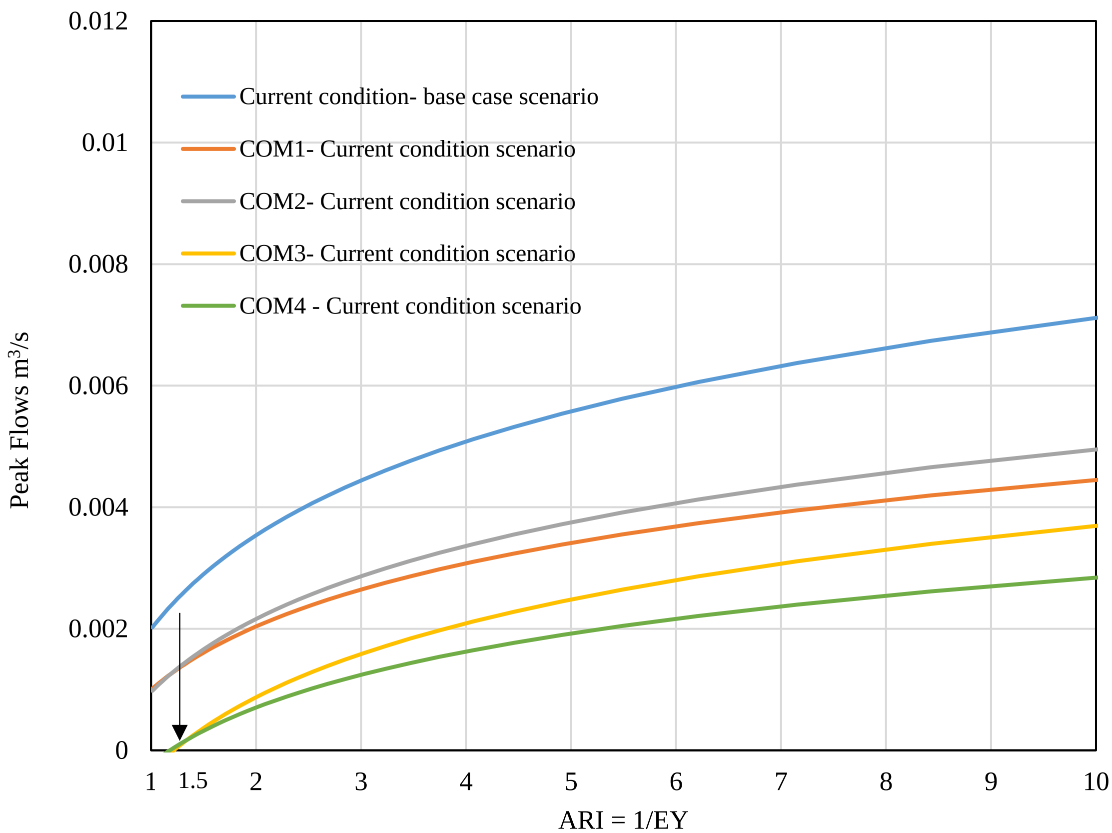


*Figure 6-6- - Fitted probabilities for possible peak flows: scenario comparisons under current condition and combination of selected assets*

#### 6.2.6.4. Individual assets under the future household condition with dual occupancy

Future condition scenario modelling results based on MUSIC model analysis and for the selected individual assets are summarised in Figure 6-7 and Table 6-9. According to the results, two 3-kL rainwater tanks decreased the probability of peak flow exceedance for the household a minimum of up to 30% and a maximum up to 65% for big events and small events, respectively. Two 5-kL tanks showed more effectiveness by reducing the probability of peak flow exceedance 10% to 15%more than 3-kL tanks for any type of event. Both 3-kL and 5-kL tanks with no usage showed a different behaviour and a reduction range for different events. Assets showed a greater range of reduction for bigger events when there was no usage. Two 3-kL tanks and two 5-kL tanks reduced the probability of peak flow exceedance for events bigger than 5 years up to 8% and 11%, respectively. The reduction range of tanks for the future condition with more impervious ratio and no usage was lower than their effectiveness in the current condition. Infiltration trenches showed a better performance than vegetated swales to reduce the probability of peak flow exceedance under the future condition with more impervious ratio compared with the current condition. Rain gardens decreased the probability of peak flow exceedance for the simulated period minimum up to 57% and maximum up to 82%.

Table 6-9- Range of peak size reduction by each system for the future condition scenario modelling in MUSIC model

| System | Reduction percentage for events 1 to 2 years | Reduction percentage for events 2 to 5 years | Reduction percentage for events > 5 |
|---|---|---|---|
| 3-kL Tank x 2 | Min 48% to max 65% | Min 38% to max 47% | Min 30% to max 38% |
| 5-kL Tank x 2 | Min 60% to max 80% | Min 49% to max 59% | Min 38% to max 48% |
| 3-kL Tank – no usage x 2 | Min 1% to max 3% | Min 3% to max 5% | Min 5% to max 8% |
| 5-kL Tank – no usage x 2 | Min 1% to max 5% | Min 5% to max 8% | Min 8% to max 11% |
| IT – 1-meter depth x 2 | Min 39% to max 57% | Min 28% to max 38% | Min 19% to max 28% |
| RG – 0.5m depth x 2 | Min 67 % to max 82% | Min 60% to max 70% | Min 57% to max 60% |
| VS – 0.5-meter length x 2 | Min 23% to max 28% | Min 20% to max 23% | Min 17% to max 20% |

In summary, in a similar manner to current condition scenarios, lower reduction percentages can be seen with the bigger events with greater ARI. All selected assets showed more effectiveness in reducing peak flows exceedances probability for smaller events (ARI =1 to 5 years) compared to bigger events (ARI > 5 years). Comparing the reduction ranges of the individual assets, 3-kL tanks with no usage, 5-kL tanks with no usage, infiltration trenches, vegetated swales, 3-kL tanks with Usage, 5-kL tanks with usage and rain gardens showed the least to the most reduction percentage range respectively. Under future condition with more impervious ratio and by adopting a combination of two assets for the townhouses, higher effectiveness was achieved in the reduction of the probability of peak flow exceedance compared with the current condition.

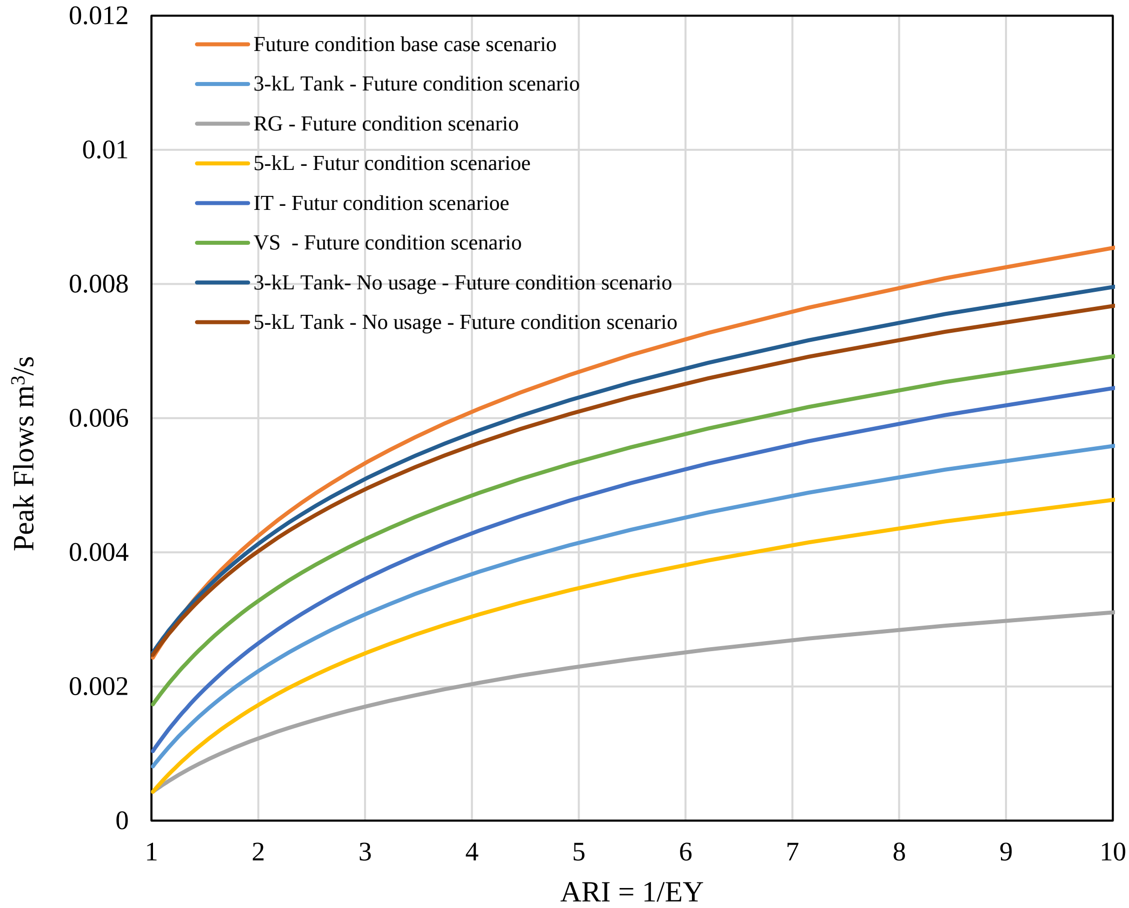


*Figure 6-7- Fitted probabilities for possible peak flows: scenario comparisons under the future condition and selected assets.*

#### 6.2.6.5. Combination of assets under the future household condition

Results of the future condition scenarios are presented in Figure 6-8 and Table 6-10. Combination of a 3-kL tank and rain garden showed a very close range of reduction to a combination of a 3-kL tank and infiltration trench minimum and maximum up to up to 40% and 80% respectively. The maximum range of reduction of the COM1 and COM2 scenarios for the future condition was 37% higher than the reduction range for the same scenarios under current condition. Besides, both combinations showed more reduction up to 35% for events with ARI > 5 years than the current condition and the same events. The maximum range of reduction was achieved under COM3 and COM4 scenarios up to 100%. Combination of three assets including a tank and a rain garden and an infiltration trench maintained peaks discharge for the 1.5-year ARI close to predevelopment levels at the household under future impervious condition. There were significant reductions in peak flows throughout the range up to 86% for the 1 in 10-year ARI.

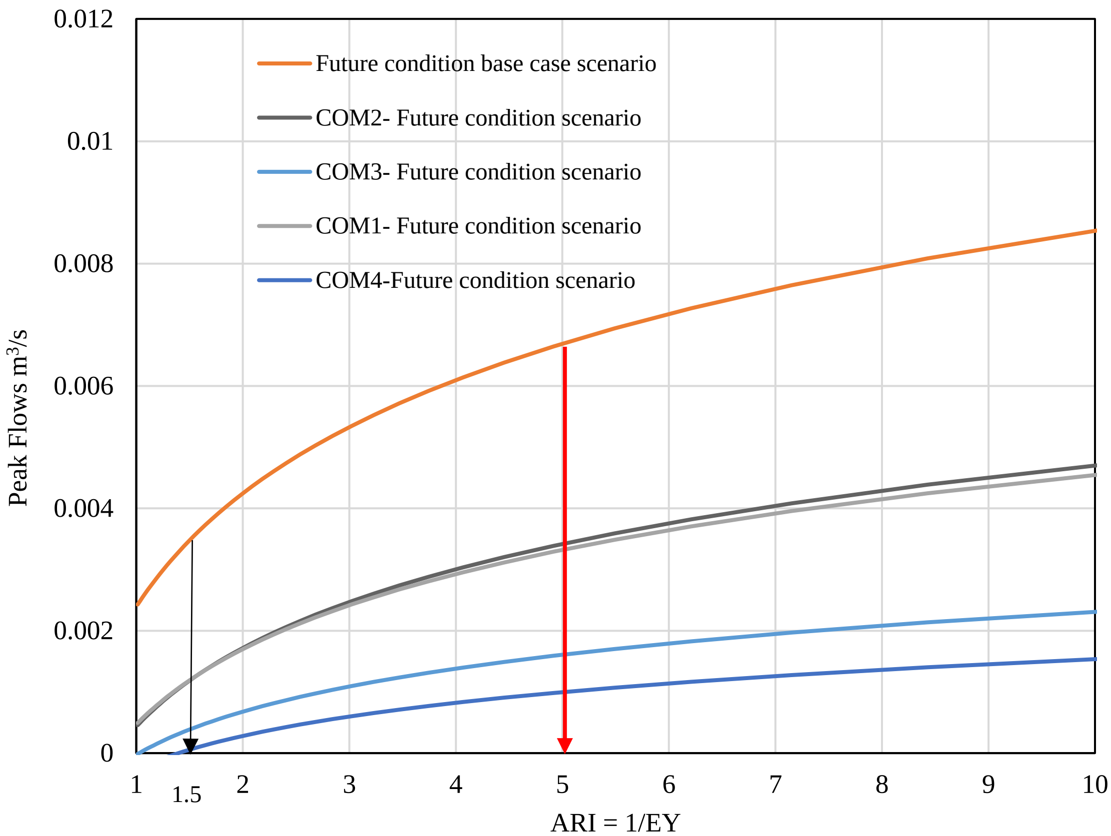


*Figure 6-8- Fitted probabilities for possible peak flows: scenario comparisons under future condition and combination of selected assets*

Table 6-10- Range of reduction for the combination of assets for future condition scenario modelling

| System | Reduction percentage for events 1 to 2 years | Reduction percentage for events 2 to 5 years | Reduction percentage for events > 5 |
|---|---|---|---|
| COM1 | Min 60% to max 80% | Min 50% to max 60% | Min 41% to max 50% |
| COM2 | Min 60% to max 80% | Min 50% to max 60% | Min 40% to max 50% |
| COM3 | Min 80 % to max 100% | Min 75% to max 84% | Min 76% to max 86% |
| COM4 | Min 90 % to max 100% | Min 85% to max 90% | Min 77% to max 86% |

In summary, using the MUSIC model, all the selected assets reduced the peak flow occurrence probability for all types of events with ARI greater or equal to 1 under the current and future condition scenarios. Given that in the future in an allotment, the impervious ratio will be greater due to the current property being replaced by two townhouses, WSUD assets are expected to reduce stormwater peak flow size significantly. Although, the rainwater tank system and no usage showed less reduction for the future condition, which stated that the performance of the tanks on peak flow reduction was less effective while the impervious ratio was higher.

#### 6.2.6.6. Peak Flow Size Reduction in PCSWMM model

Using the PCSWMM model and the adopted the chosen systems in the allotment, it was found that vegetated swales were the least effective systems to reduce the peak size for both current and future allotment conditions. At the same time, a combination of a 3-kL tank and a rain garden and an infiltration trench showed the most reduction peak size under both current and future allotment condition. Results are summarised in Figure 6-9.

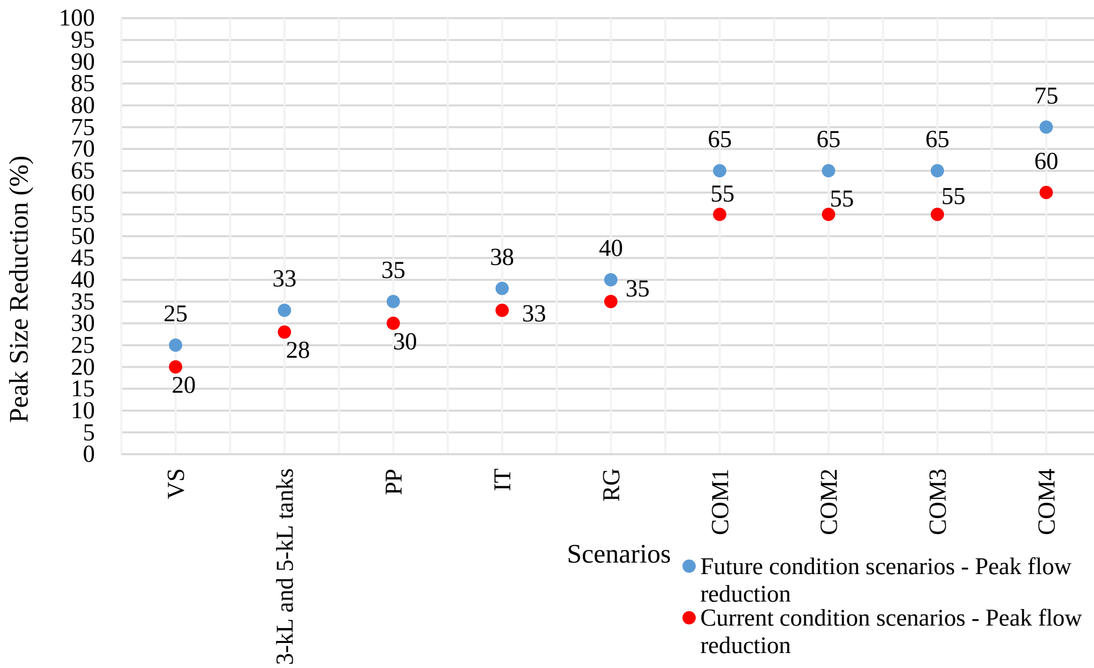


*Figure 6-9- Peak size reduction in PCSWMM model: scenario comparisons for current and future conditions*

According to the results of this study, it was found that the reduction percentage of peak size and mean annual runoff volume were similar for the selected WSUD assets in both modelling tools MUSIC and PCSWMM. Vegetated swales showed the least and the combination of three systems showed the most reduction in peak size and mean annual runoff in both modelling tools.

### 6.2.7. Life-Cycle Cost of WSUD techniques

Life cycle costing is a "process to determine the sum of all expenses associated with a product or project, including acquisition, installation, operation, maintenance, refurbishment, and discarding and disposal costs" (Standards Australia, 1999, p. 4). The LCC often provides essential input into the best stormwater system selection where stormwater management systems needed to be evaluated (Taylor 2005). General life cycle diagram has been shown in Figure 6-10.

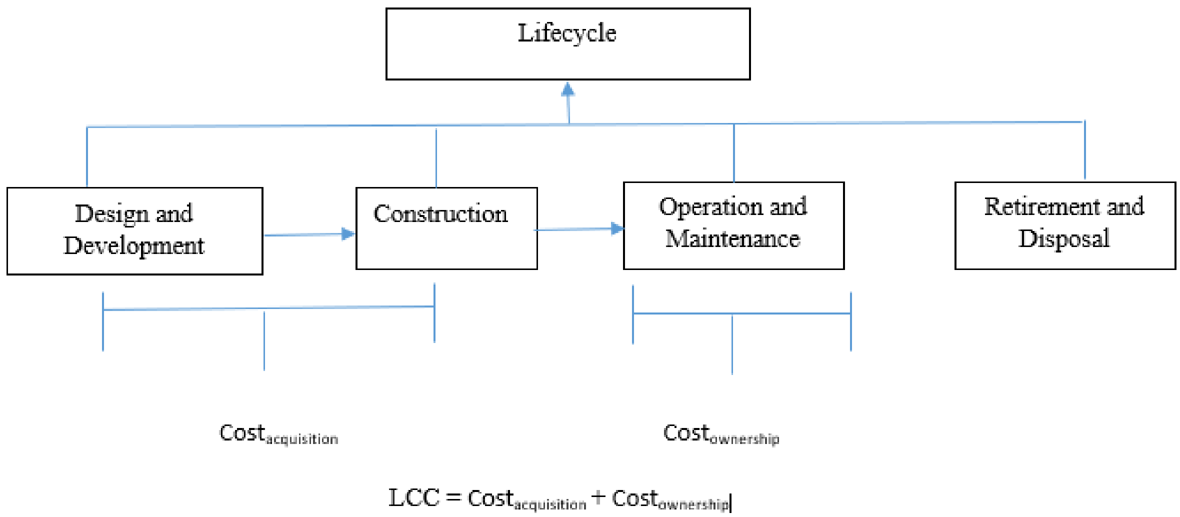


*Figure 6-10- General LCC diagram*

Following key stages defined for LCC by Commonwealth of Australia, 2001.

- Initial concept
- Asset requirements and documentation
- Asset construction
- Asset Operation
- Asset Usage
- Asset renewal process
- Asset disposal

LCC is defined as the sum of WSUD assets acquisition, operation, maintenance, conservation, and decommission in the assets' entire life span (AS/NZA 2005). LCC is a useful tool for engineering design and environmental management (Emblemsvåg 2003). Analysis of LCC is an essential tool to make cost-efficient choices. LCC represents in a Net Present Value (NPV) form specified with a discounting tool incorporated into the Life Cycle Cost Analysis method (LCCA) (Barringer, 2003). The discount range, which can vary from 1 to 15%, converts future values to present values as monetary value is not constant. Obtained NPV can quantify the impact of time on future costs after the discounting process. Blanchard's "Systems and engineering" discount rate formula is shown in Equation 6-1:

$$NPV = FV/(1 + r)n \qquad (6-1)$$

where NPV is the net present value, FV is the future value, n is the number of intervals between the present and future transaction (years), and r is estimated discount rate.

WSUD techniques usually designed with an infinite life span assuming no decommissioning period (Lloyd et al., 2002); therefore, the disposal phase in Figure 6-10 can be ignored. The maintenance phase is difficult to predict and forms the major component of LCC (Mitchell 2006). Other components of LCC, including design and construction phases, can be identified from the literature review and stormwater management expert experience.

### 6.2.8. LCC of Chosen WSUD techniques

Chosen WSUD techniques for this study includes rainwater tank, bio-retention, vegetated swales, permeable pavement, and infiltration trench. This section has discussed the LCC of the chosen techniques based on the literature review.

- ***LCC Rainwater tanks***

Typical annual maintenance cost for above ground tanks reported $70 by Kuczera and Coombes (2003). Operation and maintenance costs of a 22kL tank used for toilet flushing and garden irrigation reported $101.43/year (Gardner et al., 2003). A rainwater tank pump electricity uses 3,000kw h/ML with a cost of approximately $410.40 ML (Gardner, et al., 2004; Grant and Hallmann, 2003). The other associated cost for rainwater tanks is separate reticulate supply to toilets which can vary from $400 to $1,050 based on the tank's size (Cardno BLH 2002). Plumbing costs associated with rainwater tanks were listed by Boubli and Kassim (2003) as pipework, fitting and slab, electric and soundproof enclosure installation, supply pump and reticulation equipment. Retrofitting rainwater tanks to some existing residential properties was explored by Lane (2004) in three case studies in the Snowy Monaro region. It was concluded that $3,000 to $4,000 would be enough to retrofit a 9kL rainwater tank. This was included purchase and tank supply, pump purchase, plumbing material and labour, and the electrician cost. Typical rainwater tank supply cost was estimated by Coombes 2002 as: $540 and $860 for Aqua plate 4.5 and 9kL tanks respectively. $440 and $640 for 4.5 and 9kL Galvanised iron tanks. $670 and $1150 for 4.5 and 9kL Polymer tanks and $1,300 and $1,800 for 4.5 and 9kL concrete tanks. Total rainwater tank installation cost for a 5-10kL tank was indicated by Coombes 2002 of around $1,600 including pump installation, pressure control, and fittings. Up-to-date tanks purchases prices are available from suppliers' websites.

- ***LCC bio-retention or Raingardens***

The initial cost for bio-retention or raingardens were indicated based on a summary of cost relationships for Best Management Practice structures (BMPs) by Taylor 2005. The study explored 60 agencies across Australia to create a cost database for LCC analysis (Ira, Vesely & Krausse 2008). The cost range between $125-$150 and $225-$275 was stated for areas greater than and less than 100m$^2$. The systems' typical annual maintenance cost reported 5-7% of the construction cost by the Centre for Watershed Protection (CWP, 1998) and U.S EPA. (2001). For grassed raingardens construction and in residential areas, a cost of $135 per liner was suggested by Taylor (2005). Maintenance cost was also indicated $2.5/m$^2$ similar to vegetated swales for mature systems. Taylor (2005) suggested raingardens and swales costs as follow:

- Total construction cost for both raingardens and swales equal to 85% of the total acquisition cost
- Annual maintenance cost equal to 4.3% of the total acquisition cost
- Renewal and decommissioning costs equal 2% and 39% total acquisition cost (per annum).

The construction cost of a 3m wide and 1m deep bioretention trench was reported $410/m or 137/m$^2$ of surface area (URS 2003). The estimate was included the systems excavation, geofabric installation, drainage pipe installation, and media vegetation.

Suggested typical construction cost for swales by Taylor (2005) is 4.5/m$^2$, including earthwork, labour and media preparation. In Melbourne, Indigenous Gardens Pty Ltd suggested a construction cost of about $15 to 20/m$^2$ for the systems including labour, Indigenous vegetation, and earthwork Bryant (2003).

- ***LCC vegetated swales***

Suggested cost of maintenance for grassed swales by Lloyd et al., (2002) is ~$2.5/m$^2$/yr. However, it can reduce if regular mowing is done. A routine maintenance cost for the systems starts at 9/m$^2$/yr and reduces to 1.5/m$^2$/yr.

Bryant (2003), reported a cost of 120/m2 for the swales, including systems excavation, planting, soil, maintenance and irrigation. Some BMP size/cost relationship was developed by Lloyd et al., (2002) for a combination of wetlands and vegetated swales as follow:

- Construction cost ($) = 343,913 * Ln (Surface treatment area of BMP's in ha) + 738,607.
- Landscaping maintenance cost ($ annual) = 9,842.2 * (Surface treatment area of BMP's in ha)$^{0.4303}$

Estimated unit rate construction cost of vegetated swales by URS (2003) is $10/m$^2$ and $18/m$^2$ (if the turf is used). The costs are with no sub-soil drainage, and an additional cost of $10/m$^2$ is required for the sub-soil drainage.

- ***LCC permeable pavements***

Cost information for five types of permeable pavements was reported by Boral (2003) In New South Wales, Australia as following:

- Pavements allowing infiltration: $111/m$^2$
- Pavements allowing water collection and over sealed sub-grade: 119/m$^2$
- Mixed pavements with standard pavers: 98/m$^2$
- Asphalt pavements: 67/m$^2$
- Pavements with concrete slabs: 90/m$^2$

Typical annual maintenance of permeable pavements reported $9,700/ha by Taylor (2005) in California. Approximate cost of pavements blocks supply in Sydney estimated $30/m$^2$-$50/m$^2$ by URS (2003). The total construction cost, including systems excavation, blocks supply and installation, geofabric liner installation, sand, and gravel installation, was estimated 98.4/m$^2$.

- ***LCC Infiltration trenches***

The cost of construction for Infiltration trenches estimated $46-48 per linear meter by Earthtech Engineering Pty Ltd (2003) in Melbourne. Reported typical annual maintenance cost for the systems is 5-20% of construction cost (The Centre for Watershed Protection (CWP, 1998) and U.S EPA. (2001)).

The suggested construction cost of an infiltration trench by Taylor (2005) and for a trench with 1 m width and 1 m depth is \$60-80/m3. The unit rate of the construction cost of similar trenches in Sydney estimated \$138/m (URS 2003). The estimated cost included: systems excavation, geofabric liner installation, pipe installation, installation of filter layer and gravel layer and the fertiliser and topsoil.

Renewal/ adaption and decommissioning costs of the systems estimated at 4.1% and 35% of the total acquisition cost (Taylor 2005).

#### 6.2.8.1. Adopted LCC for the chosen techniques in this research

Typical design stages and LCC for chosen WSUD for this research are presented in Table 6-11. Size of assets for the typical allotment in this research were small between 5 to 7 m$^2$. Assumed total acquisition, establishment and maintenance costs are based on the literature review and a discussion with WSUD construction experts.

Table 6-11- Costs of chosen WSUD for an allotment

| Parameters | Raingarden | Vegetated swale | Infiltration trench | Permeable pavement | Rainwater tanks |
|---|---|---|---|---|---|
| | 5-7m$^2$ | 5-7m$^2$ | 5-7m$^2$ | 5-7m$^2$ | 3 and 5kL |
| Acquisition | \$ 2,300 | \$ 3,000 | \$ 3,000 | \$ 3,000 | \$ 3,700-\$ 4,300 |
| Establishment | \$ 2000 | \$ 2,200 | \$ 3,000 | \$ 3,000 | \$ 200 |
| Annual Maintenance | \$ 2000 | \$ 2,200 | \$ 3,000 | \$ 3,000 | \$ 100 |
| Total | \$ 6,300 | \$ 7,400 | \$ 9,000 | \$ 9,000 | \$ 4,000-\$ 4,600 |
| Life Span (Year) | 30 | 50 | 30 | 30 | 25 |
| NVP | \$ 27,300 | \$ 32,000 | \$ 38,000 | \$ 38,000 | \$ 12,191 |

Plotting the LCC for considered scenarios at the selected allotment, rainwater tanks would be the most suitable cheapest systems to manage urban stormwater volume, and peak size at an allotment scale as the urban areas develop too fast (Figure 6-11).

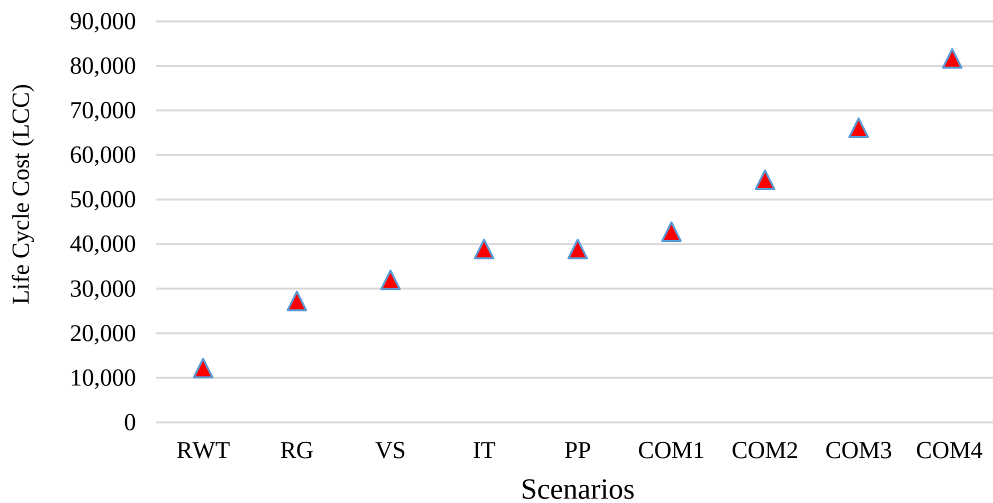


*Figure 6-11- LCC of considered scenarios for an allotment*

### 6.2.9. Conclusion

Stormwater runoffs mean annual volume and peak flow size reduction was investigated for a residential dwelling redeveloped with (a) a single dwelling and (b) redeveloped with two dwellings. In the condition with redevelopment with two dwellings, the allotment's impervious ratio was increased due to the replacement of the single dwelling with two or more townhouses. The residential allotment was selected from an urban catchment in West Melbourne. Five WSUD assets and combinations with these including rainwater tanks, infiltration trench, rain garden, and vegetated swales were selected for scenario modelling. The results of this study showed that vegetated swales and rainwater tanks without reuse were not all that effective compared with the other chosen systems to reduce the mean annual runoff volume and peak size at the allotment under both existing and future development condition with a greater impervious fraction.

Peak flow rates for storm events with an ARI of 1 in 5 years corresponding to the typical design standard for the piped drainage system were reduced by 90% for the future redeveloped condition. It was also observed that smaller events up to a 1 in 1-year ARI were usually eliminated, reducing runoff discharges' frequency, and causing reduced disturbance to the waterways. A combination of three assets including a tank, a rain garden and an infiltration trench-maintained peaks discharge for the 1.5-year ARI close to the household's predevelopment levels under future developed conditions. Besides, there were significant reductions in peak flows throughout the storm events with an ARI of 1 in 10 up to 86%. Total mean annual runoff volume reductions were observed up to 90% with three WSUD assets for the future developed condition. While they had the highest costs, the combination of a rainwater tank, rain garden and infiltration trench assets provided the most reduction in stormwater runoff volume and peak flow size undertaking MUSIC and PCSWMM modelling tools. The reduction percentage of peak size and mean annual runoff volume were similar for the selected WSUD assets in modelling tools MUSIC and PCSWMM. An LCC analysis was undertaken using the MUSIC model to rank the considered scenarios. The LCC analysis ranked the scenarios for implementation at the selected allotment from the least to the costliest one.

Effectiveness of the chosen WSUD assets was confirmed at a residential allotment scale in this study to manage the future urbanization impact, urban floods and waterway health. High reduction percent of peak flow and runoff volume will be enough to maintain the allotments' pre-development condition under future development condition. However, the systems LCC and the maintenance process should be considered essential criteria for the system implementation at a residential allotment scale.

In the "future" condition the allotment impervious ratio was increased as a result of the replacement of the single dwelling with two or more units. The typical residential property was configured to represent an allotment within an urban catchment in West Melbourne. Five WSUD techniques including rainwater tanks, infiltration trench, rain garden, vegetated swales and permeable pavement were selected for assessment. The results of this study showed that selected WSUD assets are effective to reduce the magnitude of peak flow and stormwater runoff volume for the selected allotment, even though the impervious ratio was increased for future condition up to 80%. On average, a total peak flow size reduction up to 100% was achieved for very small rainfall events with an Annual Recurrence Interval (ARI) equal to 1 in the allotment under current and future conditions. Also, total stormwater runoff volume reductions between 10% to 75% were achieved with various combinations of WSUD approaches.

## 6.3. Stormwater Scenario Modelling at Sub-catchment and Catchment Scale

### 6.3.1. Chosen WSUD Techniques

In this chapter, **rainwater tanks and rain gardens** were selected as the **two primary WSUD techniques** for stormwater modelling using **PCSWMM and MUSIC**. This selection was informed by a **comprehensive literature review** and the findings outlined in **Section 6.2** of this research. According to **Dale Browne (Pers. Comm., 2018)**, these treatment methods are the **most implemented solutions** for **infill developments and retrofit projects in Melbourne**, making them the most **practical and effective choices** for this analysis.

### 6.3.2. Study Area Simulation

- ***<u>PCSWMM model</u>***

Data for all of the stormwater pipes, pits, and the chosen catchment was inputted into the PCSWMM software as shapefile or excel file format in order to model the existing area including catchment one and two (C1 and C2). The data was obtained from Wyndham City Council. The catchment borders and the area's existing stormwater management system pits and pipelines are shown in Figure 6-12 from the PCSWMM model.

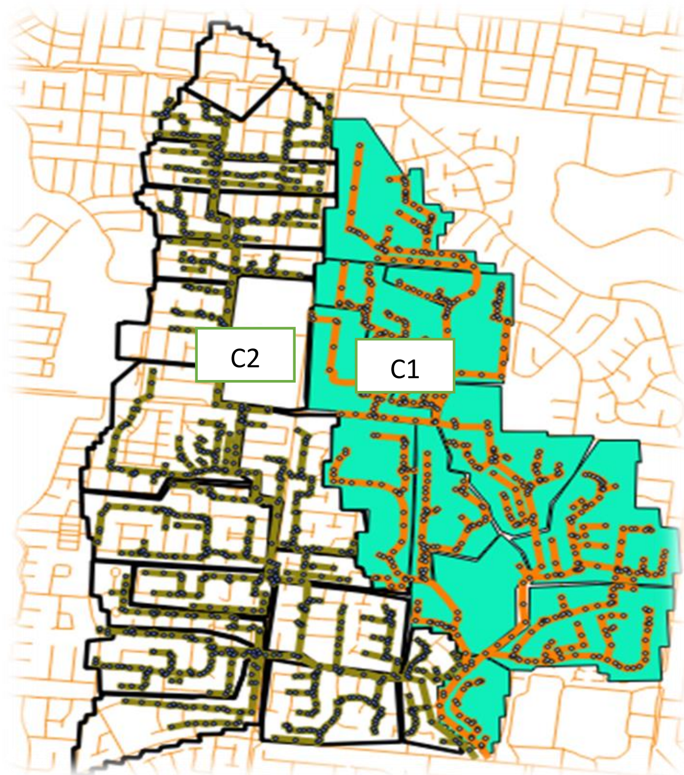


*Figure 6-12- Catchment 1(C1) and Catchment 2 (C2) boundaries, stormwater pipes and pits*

The area was divided into thirty-six smaller sub-catchments. Crucial factors for delineating sub-catchments included the direction of existing main stormwater pipes, natural waterways, and the slope of the catchment. In order to determine the optimal size for sub-catchments, three different approaches were tested for Catchment one (C1): treating the entire area as one catchment, dividing it into fourteen sub-catchments, and further dividing it into 118 sub-catchments. The results of this analysis are depicted in Figure 6-13, indicating that dividing the area into smaller sub-catchments leads to smoother changes in peak flows over time at the catchment outlet. Table 6-12 and 6-13 present the small sub-catchments areas and impervious percentage. Table 6-14 outlines the catchment parameters utilized within the PCSWMM model.

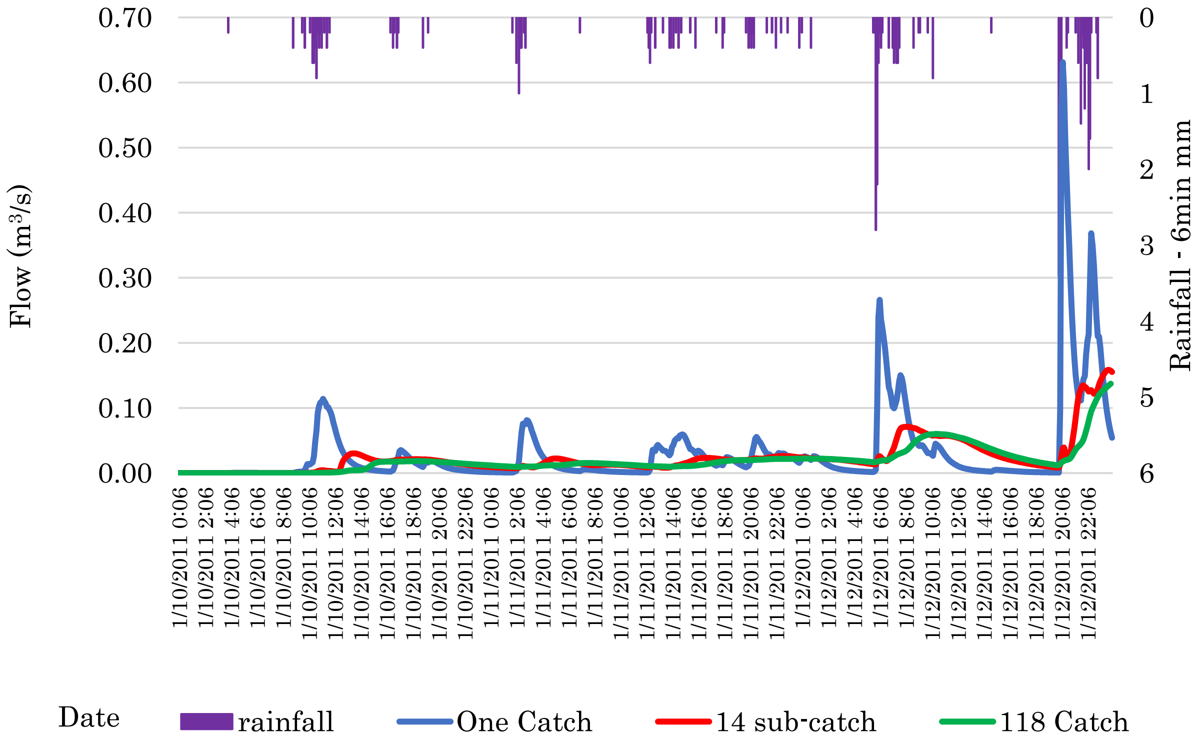


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*Figure 6-13- - Changes in flow peak at the Catchment 1 outlet pipe*

2436

Table 6-12- C1 small sub-catchments areas and impervious percentage

| Sub-catchment name | Area(ha) | Impervious percentage |
|---|---|---|
| C1-1 | 27.17 | 0.55 |
| C1-2 | 4.8 | 0.77 |
| C1-3 | 14.16 | 0.73 |
| C1-4 | 12.17 | 0.37 |
| C1-5 | 6.18 | 0.80 |
| C1-6 | 17.1 | 0.64 |
| C1-7-1 | 12 | 0.47 |
| C1-7-2 | 13.5 | 0.90 |
| C1-8 | 15.53 | 0.90 |
| C1-9 | 22.75 | 0.75 |
| C1-10 | 30.37 | 0.69 |
| C1-11 | 18.21 | 0.57 |
| C1-12 | 26.73 | 0.41 |
| C1-13 | 10.38 | 0.53 |

Table 6-13- C 2 small sub-catchments information and impervious percentage parameters

| Sub-catchment name | Area(ha) | Impervious percentage |
|---|---|---|
| C2-1 | 11.95 | 0.78 |
| C2-2 | 5.16 | 0.45 |
| C2-3 | 13.12 | 0.7 |
| C2-4 | 6.73 | 0.65 |
| C2-5 | 7.75 | 0.55 |
| C2-6 | 11.31 | 0.73 |
| C2-7 | 6.76 | 0.6 |
| C2-8 | 4.64 | 0.8 |
| C2-9 | 6.22 | 0.75 |
| C2-10 | 21.84 | 0.8 |
| C2-11 | 15.00 | 0.65 |
| C2-12 | 31.82 | 0.6 |
| C2-13 | 23.76 | 0.55 |
| C2-14 | 27.61 | 0.45 |
| C2-15 | 5.99 | 0.45 |
| C2-16 | 11.96 | 0.7 |
| C2-17 | 20.96 | 0.73 |
| C2-18 | 10.2 | 0.65 |
| C2-19 | 19.76 | 0.6 |
| C2-20 | 22 | 0.55 |
| C2-21 | 20.1 | 0.63 |
| C2-22 | 12.15 | 0.68 |

Table 6-14- Sub-Catchments parameters and references in PCSWMM model

| Parameter | Description | Amount and Reference (s) |
|---|---|---|
| N Imperv | Manning's n for overland flow over impervious area | 0.01–0.015 (Huber et a1l.1988; Sun et al. 2012) |
| N perv | Manning's n for overland flow over pervious area | 0.02–0.8 for pervious surfaces (Huber et 1l.1988) |
| Dstore Imperv | Depression storage for impervious areas (mm) | 0.03-0.25 (Sun et al. 2012; Tsihrintzis & Hamid 1998) |
| Dstore Imperv | Depression storage for pervious areas (mm) | 0.25-0.5 (Sun et al. 2012; Tsihrintzis & Hamid 1998) |
| Infiltration parameters (Green-Ampt) | Suction Head: 316 mm, Conductivity (K): 0.6 mm/hr, Initial Deficit: 0.21 | (Nasrin et al. 2017) |

Adjustment to the impervious area ratio and the parameter values listed in the table above were conducted in accordance with methodologies outlined in the literature review (refer to Section 5.9) and within the parameter ranges established by previous research. These parameters were subsequently refined through calibration procedures specific to the study area, as detailed in Chapter 5, using collected actual data. However, sensitivity analysis ultimately revealed that only two parameters—**impervious fraction** and **catchment width**—had a significant impact on the model results compared to soil parameters, manning's value and depression storage.

- ***MUSIC Model***

Like the PCSWMM model, thirty-six sub-catchments were defined as urban nodes in MUSIC model and the same impervious ratios and areas were employed for all of them according to the amounts were used in PCSWMM model. Figure 6-14 shows three of the small sub-catchments as modelled in MUSIC model as an urban node.

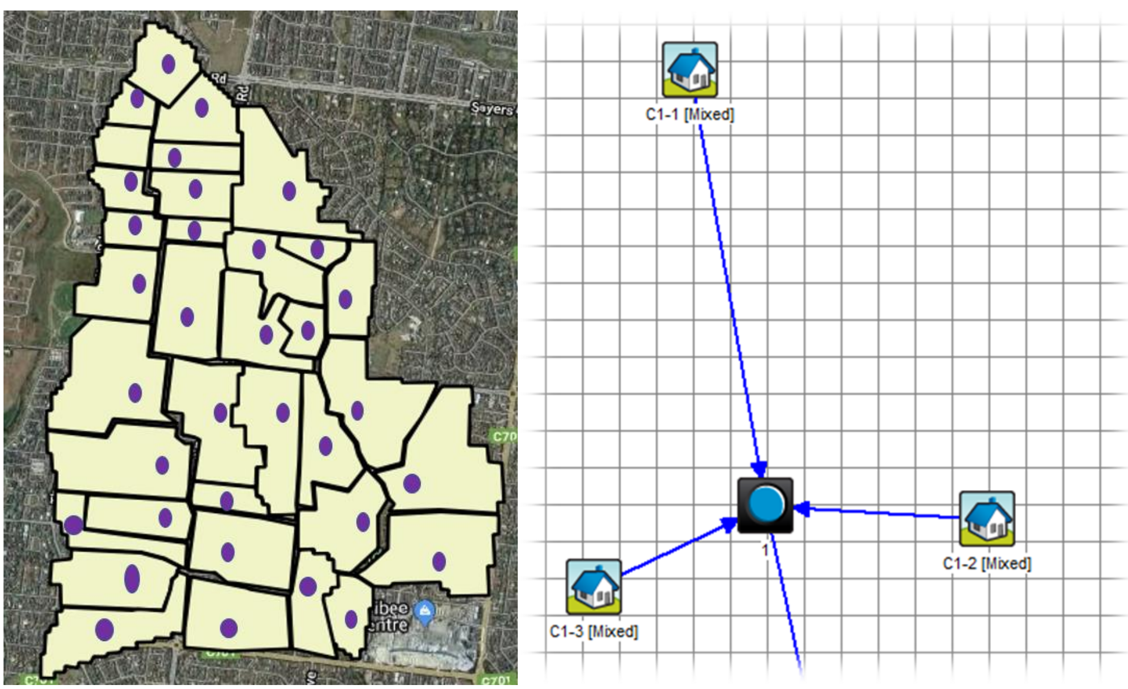



*Figure 6-14- Small sub-catchments as defined in MUSIC model*

The relevant parameters for urban nodes in MUSIC model were adopted using Melbourne Water and Wyndham City Council MUSIC guidelines. Land use have been assumed as mixed for the sub-catchments and important parameters for model set up were sub-catchments area (ha), impervious and pervious fractions, soil parameters, rainfall and evapotranspiration. To simulate the MUSIC model scenarios, 6 minutes rainfall data was used from Laverton station and evapotranspiration is derived from BoM website. Soil parameters were applied as was validated and explained in next section.

- ***<u>MUSIC Model Development for Soil Parameters Adjustment</u>***

MUSIC model was also used for soils' parameter adjustment in the area. Chosen study area (catchment C1 and C2 with an area of 570 ha) is a portion of a larger catchment that spans 1653 ha and drains out into an open drain downstream. The area includes almost 11 sub-catchments according to the WCC and MW available information and some limited flow data was available at the downstream of the water way (location highlighted in the Figure 6-15.). The available data received from Southern Rural Water company including 6 minutes flow data for a period of approximately 7 years 4/06/2010 to 30/06/2017. The data was imported into the MUSIC model simulating the whole area to validate the soil parameters.

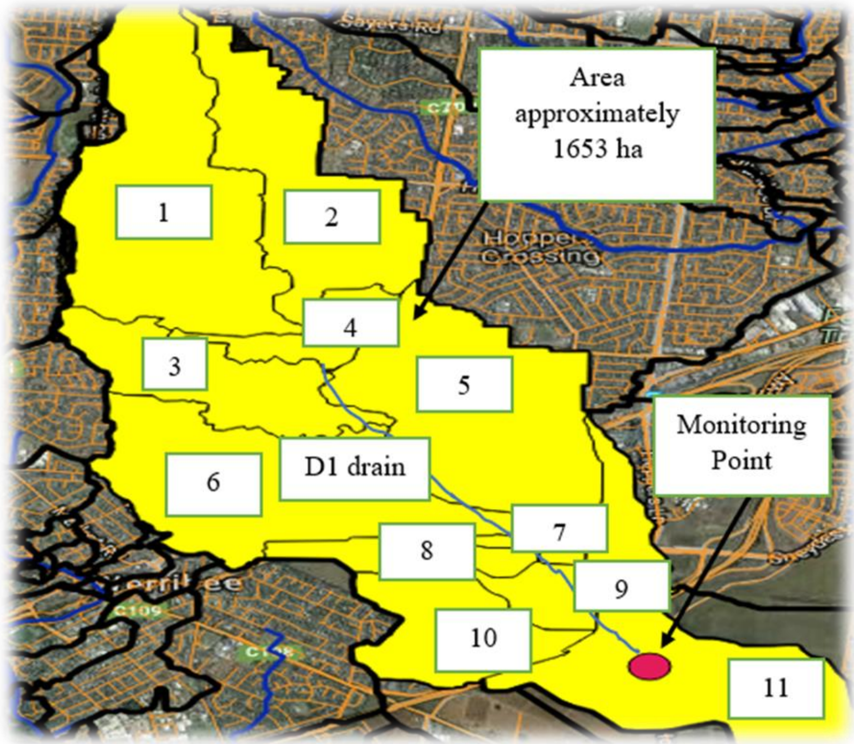



*Figure 6-15- Sub-catchments boundaries for MUSIC model configuration*

The MUSIC model parameters and considerations for validation process are listed in Table 6-15. Initial values adopted according to the literature review and references are also presented in the table for the validation process. Adjusted parameters in the MUSIC model were used for the study area stormwater modelling scenario running at different scales.

Table 6-15- List of MUSIC parameters and considerations

| Parameters | References |
|---|---|
| Area (ha) | MW and WCC GIS layers |
| Zoning | WCC MUSIC guideline (Mixed area) |
| EIA/TIA (impervious and pervious percentage) | Applied accordingly based on the method has been explained in this section and changed through the calibration process |
| Soil Storage Capacity (SSC) | Initial values from WCC MUSIC guideline |
| Soil Initial Storage (IS) | Initial values from WCC MUSIC guideline |
| Field Capacity (FC) | Initial values from WCC MUSIC guideline |
| Flow data (SRW) | 6 minutes from 4-06-2010 to 30-06-2017 |
| Rainfall and evapotranspiration | BoM for Laverton station (IDCJAC0009_087031_1800) |

Previous research indicates that the effective impervious fraction (ratio of EIA/TIA which consider directly connected area), soil storage capacity, field capacity and soil initial storage are the most influential parameters on the model validation in MUSIC software (Dotto 2009). For this study, the validation process was done manually. At the first stage the EIA:TIA ratio and corresponding effective impervious fraction was changing to understand their level of effectiveness on the model results. The best fit for the observed and simulated data achieved by changing effective impervious fraction. At next stage, soil parameters including storage capacity, initial storage and field capacity were changed to check the model sensitivity and their effectiveness level on the results. According to the several runs which have been done for this task, changing the soil parameters didn't improve the fit further, comparing the simulated and observed data. The model was most sensitive to the ratio of impervious and pervious fraction changes than the soil parameters. Therefore, best fit for the area was achieved by adjusting effective impervious fraction only and soil parameter kept as their initial amounts. This indicates that the data available is primarily smaller events generating mostly impervious area runoff and that there is limited data available for larger events generating pervious area runoff.

To check the accuracy of prediction of the MUSIC model simulated flow, the Nash–Sutcliffe coefficient of efficiency ($E_{NS}$) was used. The ENS is a commonly used goodness-of-fit measure and is suitable for reflecting the trends and overall fit of a flow hydrograph (Coutu et al. 2012). The calculation is shown in Equation 6-2.

$$E_{NS} = 1 - \left( \frac{\sum_{i=1}^{N} \left| Q_{obs}^{i} - Q_{simu}^{i} \right|^2}{\sum_{i=1}^{N} \left| Q_{obs}^{i} - \overline{Q_{obs}^{i}} \right|^2} \right) \qquad (6\text{-}2)$$

where Qobs and Qsimu address to the observed measured flows and model simulated flows respectively. N defines the number of observations. The coefficient should be close to 1 for a good fit. According to the results for this study, the ENS was calculated as 0.85. The results of the MUSIC model simulation and derived parameters for the validation are presented in Table 6-16. In addition, Figure 6-16 presents the simulated data and the observed data comparison as exported from MUSIC model. Field capacity, soil storage capacity and initial storage will be tested for the study area.

Table 6-16- Achieved parameters in MUSIC model for Area 1 for Validation

| **Parameters** | **Amount** | | | | | |
|---|---|---|---|---|---|---|
| Field Capacity (mm) | 50 mm | | | | | |
| Soil storage capacity | 125 mm | | | | | |
| Initial storage (% of capacity) | 25 | | | | | |
| Impervious area % | C1= 60 | C2=65 | C3=79 | C4=69 | C5=66 | C6=59 |
| | C7=15 | C8=2 | C9=20 | C10=10 | C11=63 | |

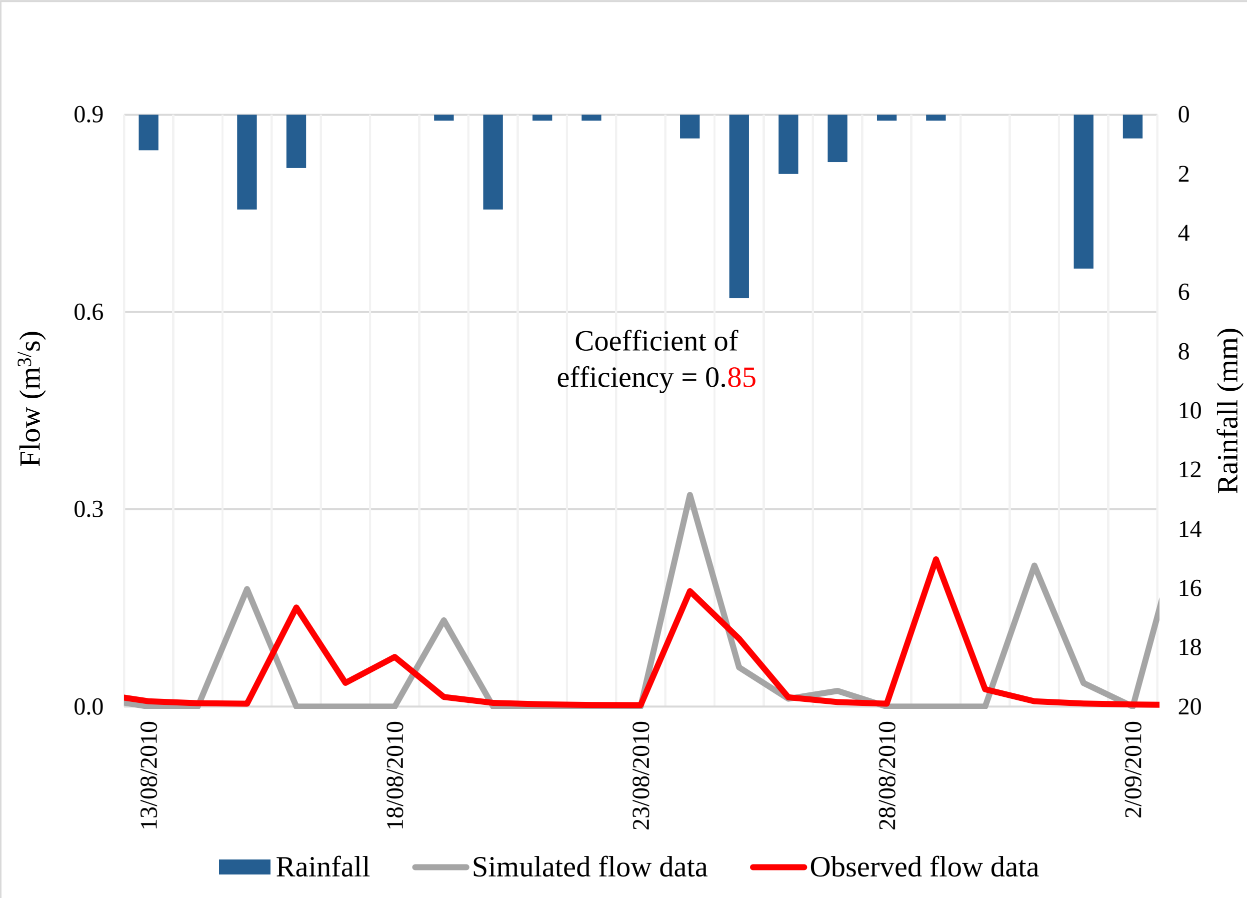



*Figure 6-16- Comparison of the observed and simulated data at the monitoring point – MUSIC model*

The coefficient of efficiency (0.85) confirmed the reliability of the overall calibration process, indicating that the sensitive parameter (impervious fraction) was appropriately adjusted to reflect the actual site conditions. However, due to potential errors in the collected data, some limitations in the MUSIC model were acknowledged. Given that the model was designed for long-term planning, ensuring the accuracy of total volume estimation for WSUD assessments was prioritized overachieving precise short-term peak flow alignment (Hosseini et al., 2020; Rossman, 2010).

### 6.3.3. Define Suitable Stormwater Management Target

The aim was to achieve an understanding of the largest reduction percentage for the selected techniques under the current conditions in a sub-catchment and the whole catchment. Therefore, the same objectives were considered as explained in section 2.8 of this thesis and examined in an allotment scale modelling. Mean annual flow and peak size were utilized as indicators to be examined.

### 6.3.4. Development of scenarios for WSUD assets assessment

Adopting chosen systems (i.e., rainwater tanks and raingardens- section 6.3.1) some scenarios were developed to be run in MUSIC and PCSWMM models. A base case scenario was considered with no system and just simulating exiting condition for the area and the scenarios with rainwater tanks and raingardens were compared to the base case. Percentage of the system's adoption assumed the same for MUSIC and PCSWMM models as was explained according to ABS report 2013. Table 6-17 and 6-18 explained the scenarios have been considered for the scenario modelling in MUSIC and PCSWMM models.

Capital cities in Australia, households experienced an increase in the proportion of rainwater tanks installation during 2007 and 2010 (15% in 2007 and 28% in 2010) (Australian Bureau of Statics 2010). For the scenario modelling in this study the percentage of house for adopting rainwater tanks and raingardens in the area, was assumed to be 28% refer to the Australian Bureau of Statics report 2010. According to the available guidelines and literature review the likely tank sizes are 1 kL, 2 kL, 3 kL, 4 kL and 5 kL. Under a survey which has been done by CSIRO in Melbourne (Moglia et al. 2015) the most adopted tank sizes are 2 to 3 kL and the average size is 4.3 kL. Therefore, and for this study 3 kL and 5 kL tanks selected for this study for scenario modelling. A typical rainwater tank is shown in Appendix A.

Rain gardens can be considered as suitable lot scale systems for stormwater management purposes. Burns et al. 2014, suggested a combination of rainwater tanks for reuse and rain gardens with infiltration for the management of flow regime at small scales. Raingardens are depressions which contain vegetation in an engineered soil mixture placed above a gravel drainage bed. They are suitable for store, infiltrate and evaporate of both direct rainfall and runoff captured from surrounding areas and convey the excess stormwater to the pervious area. A typical raingarden is shown in Appendix A.

Table 6-17- Considered scenarios for modelling using PCSWMM model

| Systems | Size | Explanation | Stormwater management indicators | Selected Events |
|---|---|---|---|---|
| Rainwater tanks | 3 kL and 5 kL | To collect runoff from 28% of roof's areas | Runoff volume and Runoff peak size | 6-minute rainfall records available from 1/12/2010 to 1/12/2011 for a continuous simulation (one year) |
| Raingardens | RG | To treat 28% of the sub-catchment's areas | Runoff volume and Runoff peak size | Single storm event: using 50% and 10% Annual Exceedance Probability (AEP) rainfall and Australian Rainfall Runoff (ARR) storm patterns |

Table 6-18- Considered scenarios for rainwater tanks and raingardens modelling in MUSIC model

| Systems | Size | Explanation | Stormwater management indicator | Selected scales | Selected Events |
|---|---|---|---|---|---|
| Rainwater tanks | 3 kL and 5 kL | To collect runoff from 28% of roof's areas | Runoff peak | - One allotment (area of 0.055 ha)<br>- One small sub-catchment (area of 27 ha)<br>- All sub-catchments (570 ha) | 6-minute rainfall records available from 7/12/2001 to 31/12/2017 for a continuous simulation |
| Raingardens | 0.5m depth | To treat 28% of the sub-catchment's areas | | - One allotment (area of 0.055 ha)<br>- One small sub-catchment (area of 27 ha)<br>- All sub-catchments (570 ha) | |

- ***Selected events***

Laverton station is the closest rainfall station to the study area. For scenario running in MUSIC model all the 17 years rainfall data in 6-minutes interval was used for continuous simulation. One year from December 2010 until December 2011 were selected for a continuous simulation in PCSWMM model. To understanding the effectiveness of the selected systems to reduce the size of peak, some single events were considered. For the single storm events, storm patterns reviewed according to the Australian Rainfall Runoff (ARR) book 2016 and patterns selected from BoM website (ARR Data Hub) for the study area. Storm designed manually in PCSWMM model for two different Annual Exceedance Probability (AEP) including 10% and 50%. Table 6-17 shows the considered scenarios for modelling in the study area using PCSWMM model.

- ***MUSIC Model Reuse Demand Allocation***

Current water usage estimated using Yara Valley Water (Ghobadi et al. 2013) and Wyndham city information. By assuming 3 people as average household size for the study area the total household's usage is approximately 413 L/P/day for indoor and outdoor. Based on the current usage typical household water usage is shown in Figure 6-17. In MUSIC model and to allocate usage demand for modelling rainwater tanks below scenario is considered:

- Toilet flushing plus laundry and garden requirement (206 L/household/day)

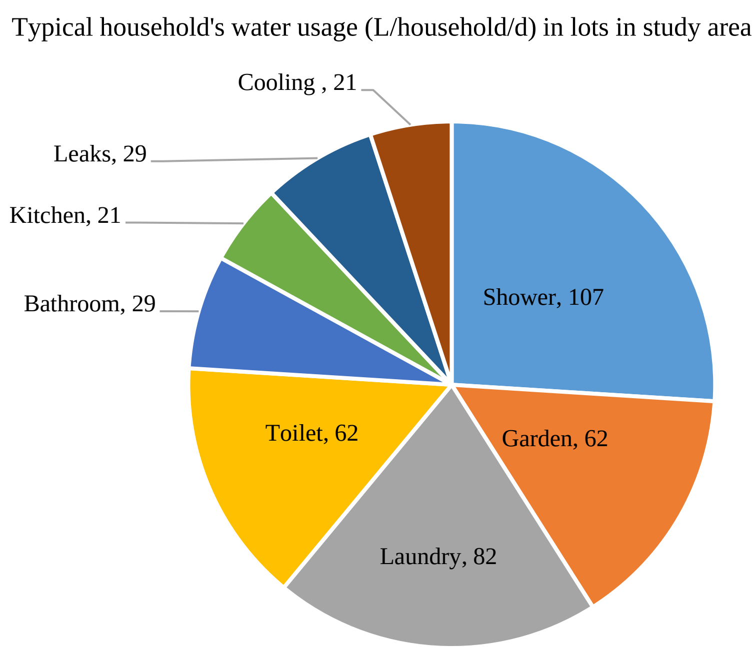



*Figure 6-17- Typical water usage for existing lots in study area (Wyndham City Council water need, 2018)*

- ***MUSIC Model Scenario Modelling Methodology***

To understand the effectiveness of rainwater tanks and raingardens on flow peak reduction under considered scenarios in the study area, partial series analysis method has been adopted using MUSIC model results. One appropriate method to understand the catchment's behaviour in terms of flood is to undertake a partial or annual series analysis (Ball et al. 2016). The partial series is preferable for smaller events with 10% AEP however annual series is preferable for bigger events. The partial series analysis method constructs an empirical estimate of the relationship between peak magnitude and ARI (or average number of exceedances per year (EY)). In this study the method helped to find independent peaks above some threshold for the selected scales and then fitting a probability model to those peaks. The following steps described the method as applied in this study:

- Considered scenarios were simulated in MUSIC model for different selected scales. Base cases were developed, and later rainwater tanks and raingardens were adopted accordingly for each scale to be run.
- Flows were exported for each simulated scenario from MUSIC model at a 6-minute timestep interval for the running period from 2001-2017. The reason for exporting simulated flows in 6-minutes interval was to ensure that the peak flows were represented adequately.
- Using the exported flows data, maximum daily peak flows were identified using excel spreadsheet and were plotted against the time assuming three days gap to ensure the plotted peaks were independent.
- Considering available recorded data, which was 17 years, 47 peaks were selected above a specific threshold for each scenario for analysis.
- The selected peaks were ranked from the largest to smallest and each one assigned an individual rank.
- Using the equation 6-3 expected exceedance flows were plotted for each scenario:

$$EY = \frac{n + 0.2}{i - 0.4} \qquad (6-3)$$

where EY is the number of exceedances per year, $n$ is the number of years of record, which is 17 for this study, and $i$ is the rank for peaks.

- A statistical distribution was fitted to the plotted data for each scenario to compare the results for each and every scenario and for different scales.
- Several statistical distributions were tried to find the best fit to the data. Log Pearson III was found to provide the best fit based on several goodness of fit statistics (Appendix B).
- Probability fits were plotted and were compared.

Results of the scenario modelling are presented in next sections for one sub-catchment and all sub-catchments.

The study has shown that OSD can reduce a 99% chance of increased runoff to less than 8% when a land parcel is in the upper reaches of a catchment. In the lower portions of the same catchment however, the same OSD has a 72% change of increasing runoff, compared to a 58% chance without.

### 6.3.5. PCSWMM Model Results

As previously stated, 3 kL and 5 kL tanks were selected to collect stormwater runoff from 28% of the existing allotment's roofs. The number of tanks assigned is calculated by calculating the number of existing allotments and multiplying by 28%. The results for runoff volume and runoff peak size reductions are reported in the sections below, under the defined scenarios in Table 6-17. The PCSWMM model does not allow for the insertion of a household demand to control rainwater tank outflow, therefore excess runoff drains into the pervious area. Figure 6-18 shows a schematic diagram of the system configuration as it behaves in the model. (Walsh et al., 2014).

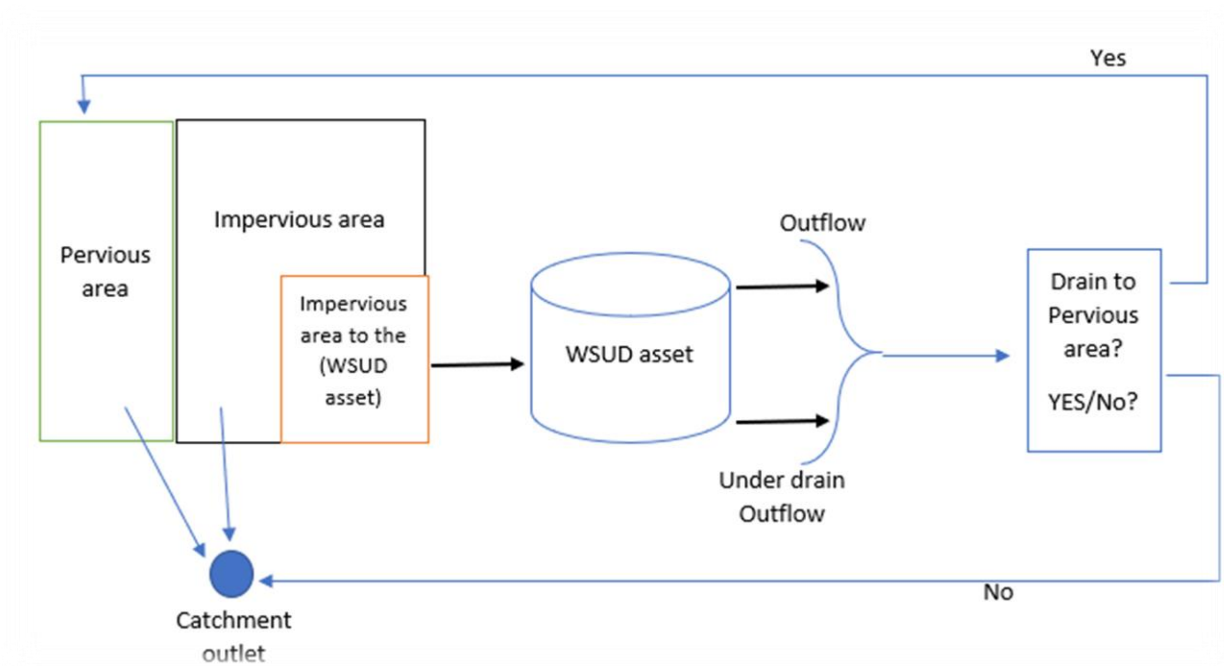



*Figure 6-18- Schematic diagram of the rainwater tanks and raingardens representation in PCSWMM model*

#### 6.3.5.1. Rainwater Tanks Runoff volume reduction

According to the results, systems reduced the runoff volume between 30 to 35% based on the catchments sizes in the area over the one year. However, the size of the thanks, did not affect significantly for runoff volume reduction. Table 6-19 and 6-20 and Figures 6-19 and 6-20 present the results.

Table 6-19- Runoff volume comparison for 3 kL and 5kL rainwater tanks in small sub-catchments

| Sub-Catchments | Area ha | Impervious ratio | Runoff Volume No Tank ($m^3$) | Runoff Volume with RWT (collecting runoff from 28% roof's areas) 3 kL | Runoff Volume with RWT (collecting runoff from 28% roof's areas) 5 kL | Difference in volume % |
|---|---|---|---|---|---|---|
| C1-1 | 27.17 | 0.55 | 9,130 | 1,434 | 1,430 | 33 |
| C1-2 | 4.8 | 0.77 | 2,338 | 390 | 385 | 35 |
| C1-3 | 14.16 | 0.73 | 6,455 | 1,095 | 1,085 | 30 |
| C1-4 | 12.17 | 0.37 | 2,635 | 399 | 390 | 35 |
| C1-5 | 6.18 | 0.80 | 3,108 | 560 | 555 | 30 |
| C1-6 | 17.1 | 0.64 | 6,763 | 1,160 | 1,140 | 30 |
| C1-7-1 | 11.75 | 0.47 | 3,325 | 472 | 470 | 35 |
| C1-7-2 | 13.44 | 0.90 | 7,668 | 1,479 | 1,459 | 25 |
| C1-8 | 15.53 | 0.75 | 7,291 | 1,235 | 1,230 | 34 |
| C1-9 | 22.75 | 0.69 | 9,763 | 1,664 | 1,644 | 30 |
| C1-10 | 30.37 | 0.57 | 10,600 | 1,845 | 1,840 | 25 |
| C1-11 | 18.21 | 0.41 | 4,418 | 635 | 630 | 37 |
| C1-12 | 26.73 | 0.535 | 8,625 | 1,366 | 1,356 | 32 |
| C1-13 | 10.38 | 0.50 | 3,144 | 505 | 500 | 32 |

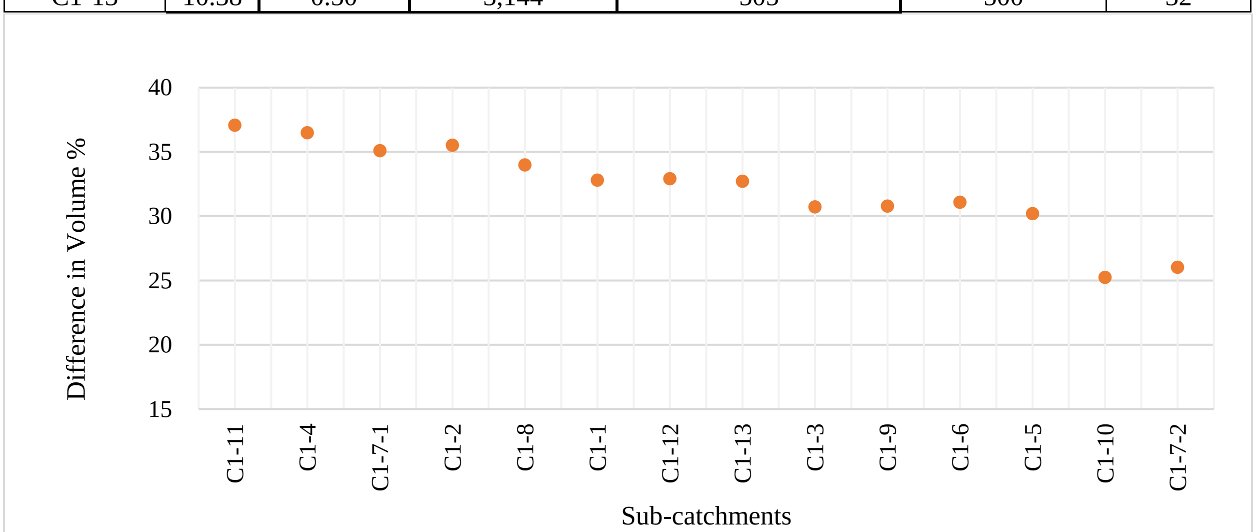



*Figure 6-19- Runoff volume comparison for small sub-catchments – C1 for the selected year with 3 kL and 5kL tanks*

Table 6-20- Runoff volume comparison for 3 kL and 5kL rainwater tanks in small sub-catchments – C2 for selected year

| Sub-Catchments | Area ha | Impervious ratio | Runoff Volume No Tank ($m^3$) | Runoff Volume with RWT (collecting runoff from 28% roof's areas) 3 kL | Runoff Volume with RWT (collecting runoff from 28% roof's areas) 5 kL | Difference in volume % |
|---|---|---|---|---|---|---|
| C2-1 | 11.95 | 0.78 | 5,313 | 3,560 | 3,560 | 33 |
| C2-2 | 5.16 | 0.45 | 2,529 | 1,644 | 1,644 | 35 |
| C2-3 | 13.12 | 0.7 | 5,729 | 3,781 | 3,781 | 34 |
| C2-4 | 6.73 | 0.65 | 3,927 | 2,749 | 2,749 | 30 |
| C2-5 | 7.75 | 0.55 | 3,140 | 2,041 | 2,041 | 35 |
| C2-6 | 11.31 | 0.73 | 4,940 | 3,310 | 3,310 | 33 |
| C2-7 | 6.76 | 0.6 | 2,741 | 1,864 | 1,864 | 32 |
| C2-8 | 4.64 | 0.8 | 2,136 | 1,474 | 1,474 | 31 |
| C2-9 | 6.22 | 0.75 | 1,939 | 1,338 | 1,338 | 31 |
| C2-10 | 21.84 | 0.8 | 2,872 | 1,953 | 1,953 | 32 |
| C2-11 | 15.00 | 0.65 | 4,906 | 3,385 | 3,385 | 31 |
| C2-12 | 31.82 | 0.6 | 11,660 | 8,045 | 8,045 | 31 |
| C2-13 | 23.76 | 0.55 | 12,750 | 8,543 | 8,543 | 33 |
| C2-14 | 27.61 | 0.45 | 15,250 | 9,913 | 9,913 | 35 |
| C2-15 | 5.99 | 0.45 | 1,961 | 1,333 | 1,333 | 32 |
| C2-16 | 11.96 | 0.7 | 6,140 | 4,237 | 4,237 | 31 |
| C2-17 | 20.96 | 0.73 | 11,090 | 7,652 | 7,652 | 31 |
| C2-18 | 10.2 | 0.65 | 6,984 | 4,889 | 4,889 | 30 |
| C2-19 | 19.76 | 0.6 | 12,090 | 8,463 | 8,463 | 30 |
| C2-20 | 22 | 0.55 | 10,450 | 7,315 | 7,315 | 30 |
| C2-21 | 20.1 | 0.63 | 8,324 | 5,577 | 5,577 | 33 |
| C2-22 | 12.15 | 0.68 | 4,166 | 2,833 | 2,833 | 32 |

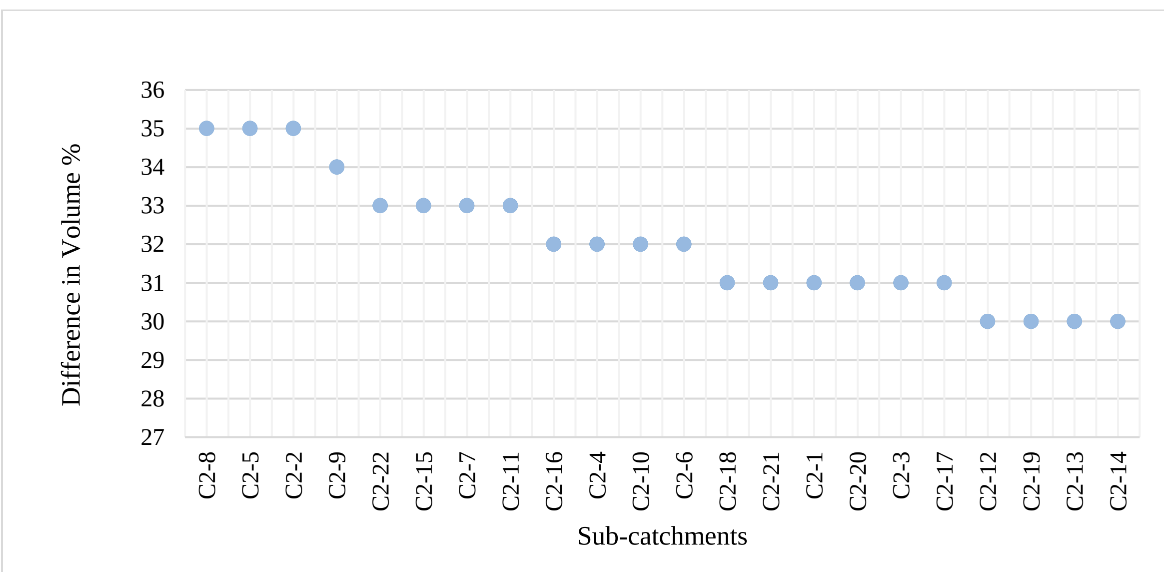



*Figure 6-20- Runoff volume comparison for small sub-catchments- C2- for the selected year with 3 kL and 5kL tanks*

#### 6.3.5.2. Rainwater Tanks Peak flow reduction

In PCSWMM model and according to the Figure 6-18, collected runoff in tanks can be drained to the pervious area under different delay times including 0, 12, 24 and 36 hours which is called drain time. This represents a ‘leaky tank’ system where a portion of the tank is allowed to drain to a lawn or garden area, so it remains available to provide storage during storm events with a high level of certainty (Figure 6-18). To understand rainwater tanks effectiveness on peak flow reduction at the catchment outlet 3 kL tanks evaluated by adopting some sub-scenarios explained in Table 6-21 using different draining delay times. In PCSWMM model the rainwater tanks underdrain flow can be governed by the submerged orifice equation as shown in Equation (6-4) (Walsh et al. 2014).

$$Q = C\,(D - H_d)^{n} \qquad (6\text{-}4)$$

where, C is draining coefficient, D is the stored water height, Hd is the drain offset, and n is the drain exponent. C coefficient can be calculated based on Equation (6-5).

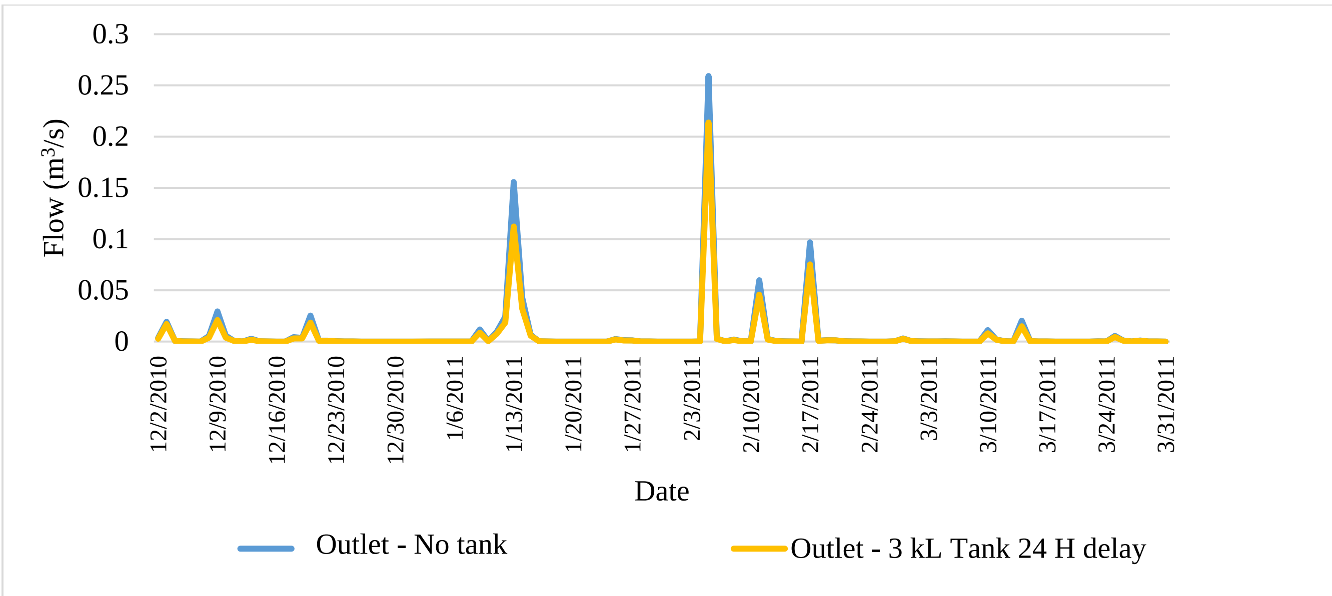
$$C = 2\,(25.4D)^{0.5} \quad /T \qquad (6\text{-}5)$$

where D and T are tank depth and time of drain in millimetre and hour respectively. The drain time impacts on the underdrain flow so, three different delay times are tested to compare the peak flow size reduction at the C1 outlet.

Figure 6-21 shows the results from the sub-scenarios and the reduction in flow obtained with 24 hours delay for draining. Based on the results, peak size can be reduced between 15% at the catchment outlet with 24 hours delay in draining compared to the scenario with no rainwater tank. 5 kL tanks and raingardens with 0.5 depth showed the same results as 3 kL tanks.

Table 6-21- Considered scenarios for 3 kL rainwater tank to compare C1 outlet peak flow reduction.

| Tanks size | Roof area treated % | Average roof areas $m^2$ | Drain Delay (HR) |
|---|---|---|---|
| 3 kL | 28 | 240 | 0 |
| | | | 12 |
| | | | 24 |



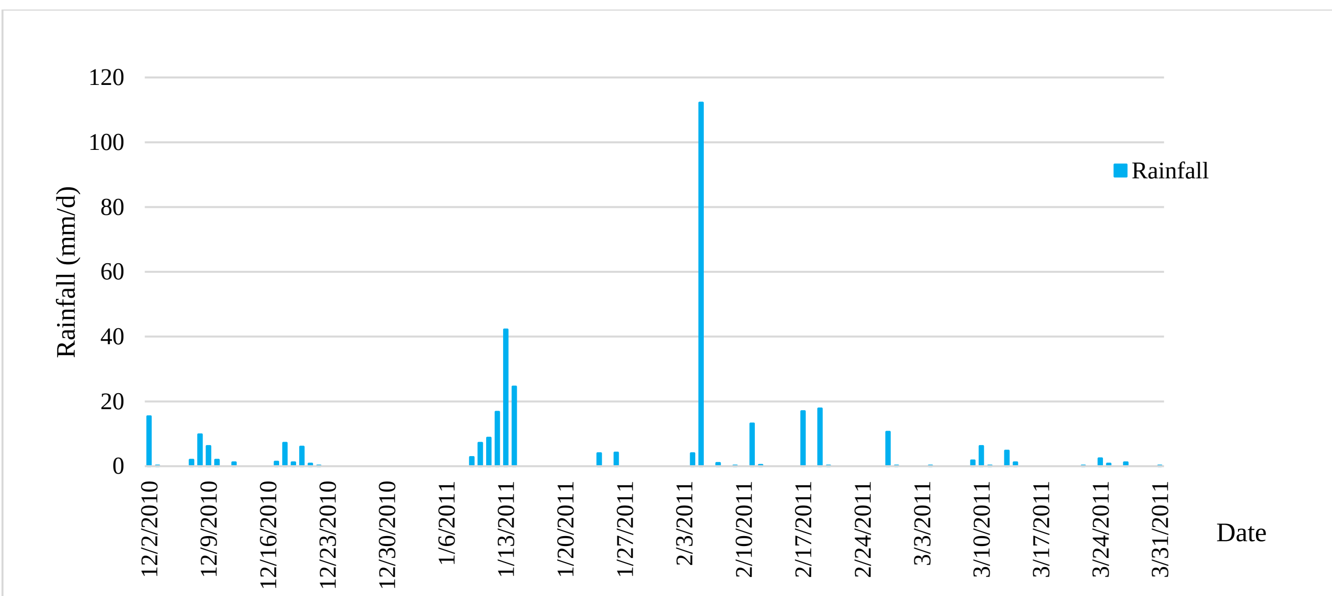




*Figure 6-21- Comparison of sub-scenarios for peak size reduction by adopting 3 kL rainwater at the sub-catchments- for one-year months*

#### 6.3.5.3. Rainwater Tanks Peak flow reduction under single storm events

Another approach which has been applied in this study to understand the rainwater tanks effectiveness for peak flow reduction in small sub-catchments is single storm event simulation. This approach uses rainfall data from different events by Annual Exceedance Probability (AEP) which is available for the study area via BoM website. The method simulates single short storm events. In this study ARR storm patterns are used to understand the study area storm pattern (Ball et al. 2016; http://data.arr-software.org/). For 10% and 50% AEP 10 patterns created by applying the rainfall amounts in PCSWMM model. Rainfall information was derived from Bureau of Meteorology (BoM) website and IFD design rainfall intensity charts using the location of the study area longitude and latitude. This method is highly dependent on the rainfall amount. Table 6-22 presents the selected rain events to simulate single events in PCSWMM model.

Table 6-22- Selected rain events for LID-scenario evaluation

| Duration (Min) | Total rainfall (mm) – 50% AEP | Total rainfall (mm) – 10% AEP |
|---|---|---|
| 10 | 7.29 | 12.8 |
| 15 | 8.85 | 15.6 |
| 45 | 13.7 | 23.6 |
| 60 | 15.1 | 26.3 |
| 90 | 17.4 | 29.9 |
| 120 | 19.2 | 32.7 |
| 180 | 22.1 | 37.1 |
| 360 | 28.2 | 46.6 |
| 720 | 35.6 | 59.1 |
| 1,440 | 74.6 | 43.7 |

Storms created in PCSWMM model and according to the results 3 kL and 5 kL rainwater tanks reduced the flow rate up to 15 % for 50% AEP storm events and up to 18% for 10% AEP storm events respectively. Figure 6-22 to 6-25 show the results for the peak flow reduction at sub-catchments in C1 and C2 by adopting 3 kL and 5kL rainwater tanks.

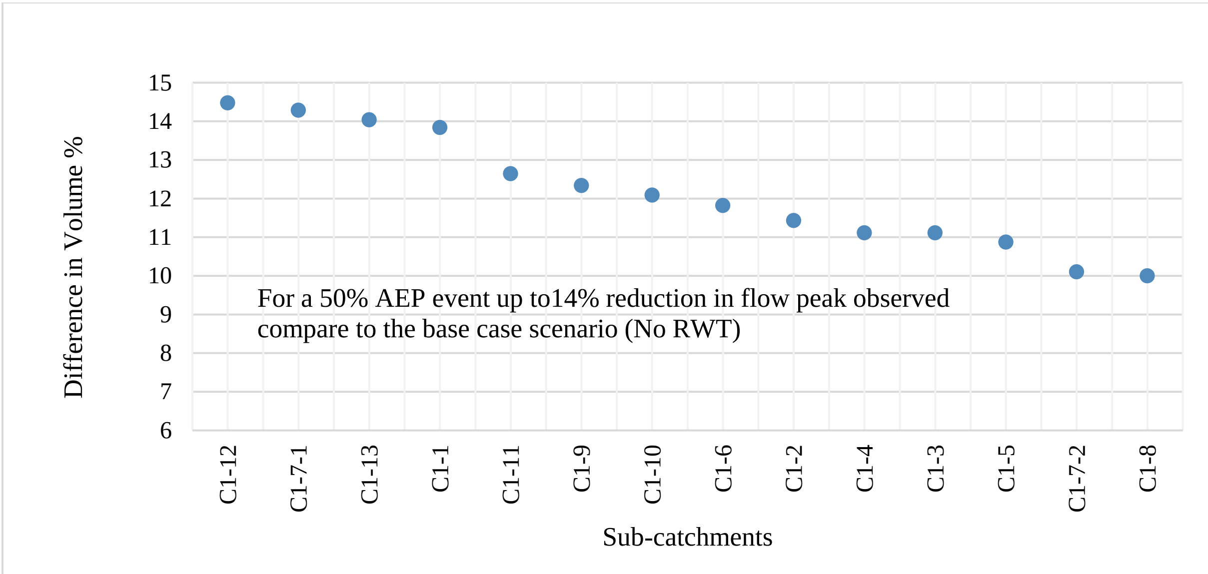



*Figure 6-22- Difference in peak size for simulated 50% AEP event – C1 with 3kL and 5kL tanks*

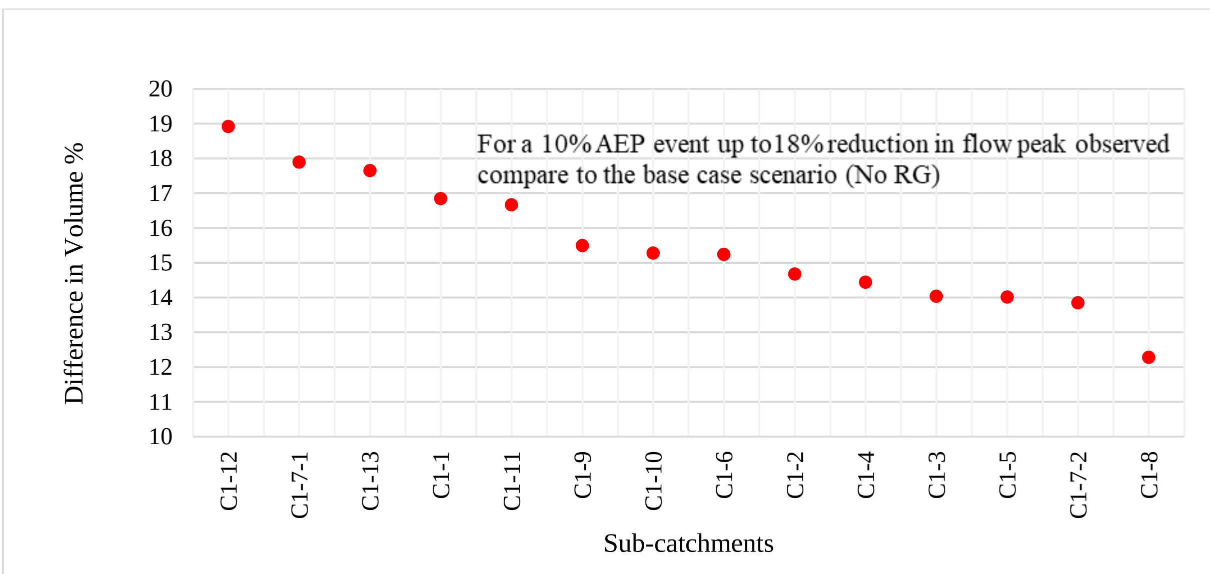



*Figure 6-23- Difference in peak size for simulated 10% AEP event – C1 with 3kL and 5 kL tanks*

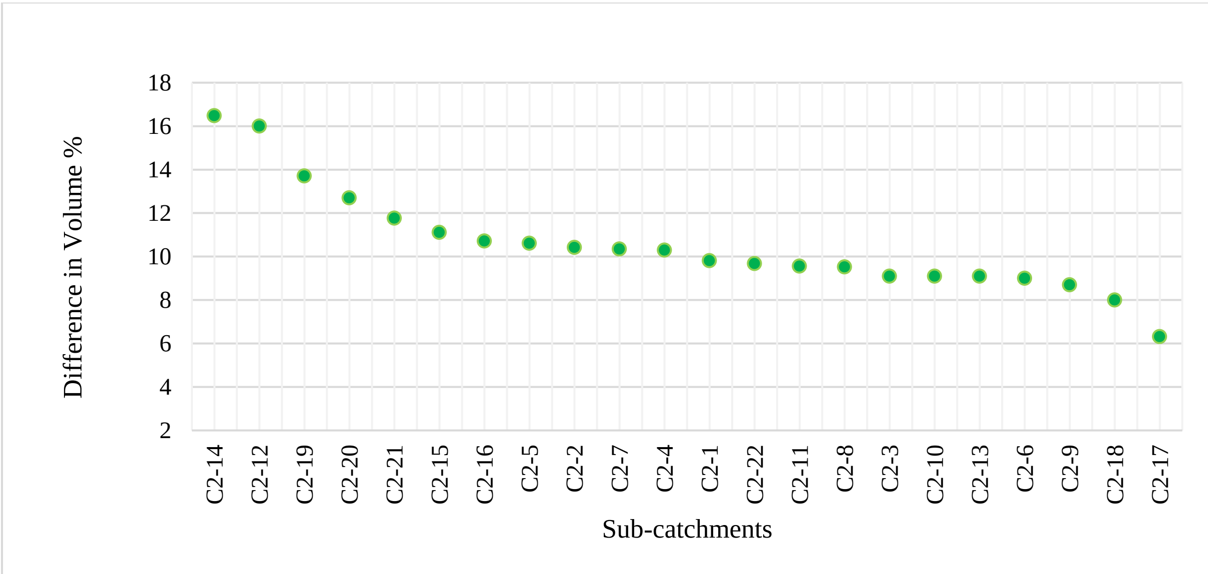



*Figure 6-24- Difference in peak size for simulated 10% AEP event – C2 with 3kL and 5kL tanks*

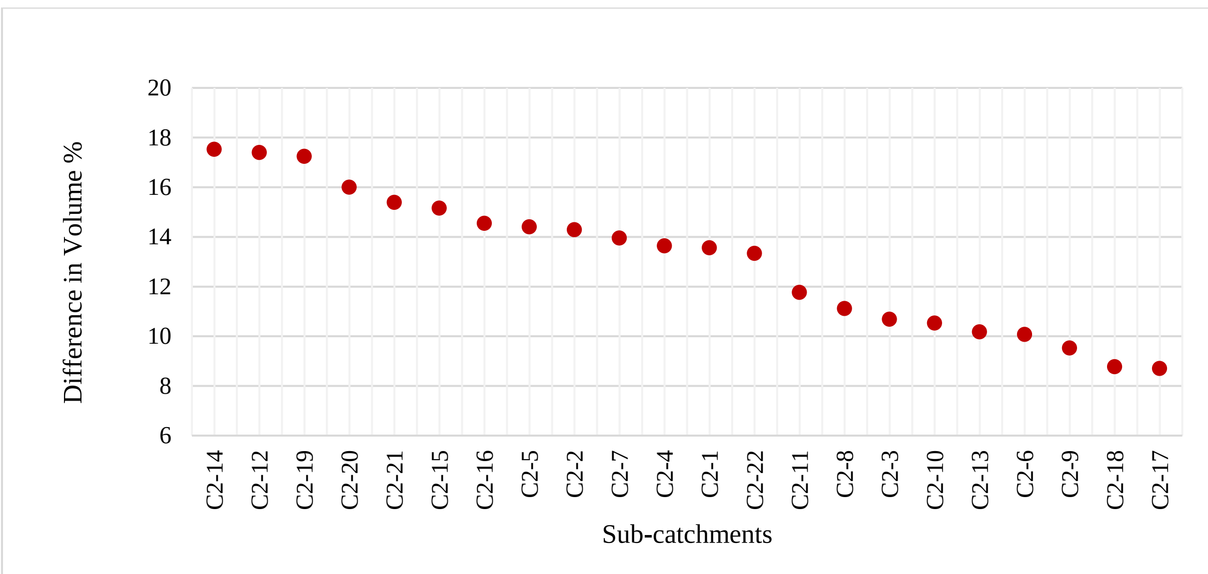



*Figure 6-25- Difference in peak size for simulated 50% AEP event – C2 with 3kL and 5kL tanks*

#### 6.3.5.4. Raingardens adoption and results

Raingardens are depressions which contain vegetation in an engineered soil mixture placed above a gravel drainage bed. They are suitable for store, infiltrate and evaporate of both direct rainfall and runoff captured from surrounding areas and convey the excess stormwater to the pervious area or the stormwater pipe network. Table 6-23 presents the specification of the adopted raingardens in PCSWMM model in the area to treat 28% of the total impervious area. Parameters were applied according to the PCSWMM user manual and literature review at this stage and optimal size of the systems will be finalised at next stage. The schematic representation of the raingarden systems in PCSWMM model can be explained according to the figure presented in Appendix A. The number of raingardens applied in the area by counting the existing number of households and multiply by 28%.

Table 6-23- Raingarden's specifications adopted in PCSWMM model

| **Soil type** | **Thickness (mm)** | **Porosity fraction** | **Field capacity** | **Wilting point** |
|---|---|---|---|---|
| Silt Loam | 500 | 0.5 | 0.28 | 0.17 |
| **Conductivity (mm/hr)** | **Suction head (m)** | **Seepage rate (mm/hr)** | **Berm Hight (mm)** | **Vegetation volume** |
| 100 | 0.170 | 3.6 | 152 | 0.25 |

Runoff volume reduced up to 35% by adopting raingardens to treat the 28% of the total impervious area. Results for the sub-catchments runoff volume reduction in C1 and C2 is presented in Table 6-24 to 6-25 and Figure 6-26 and 6-27.

Table 6-24- Runoff volume comparison for small sub-catchments without and with raingardens- C1 for one year

| Sub-Catchments | Area ha | Impervious ratio | Runoff Volume No raingarden ($m^3$) | Runoff Volume with raingarden (28%) | Difference in volume % |
|---|---|---|---|---|---|
| C1-1 | 27.17 | 0.55 | 2,128 | 1,434 | 33 |
| C1-2 | 4.8 | 0.77 | 597 | 480 | 20 |
| C1-3 | 14.16 | 0.73 | 1,566 | 1,094 | 30 |
| C1-4 | 12.17 | 0.37 | 614 | 399 | 35 |
| C1-5 | 6.18 | 0.80 | 795 | 560 | 30 |
| C1-6 | 17.1 | 0.64 | 1,654 | 1,161 | 30 |
| C1-7-1 | 11.75 | 0.47 | 724 | 470 | 35 |
| C1-7-2 | 13.44 | 0.90 | 1,972 | 1,476 | 25 |
| C1-8 | 15.53 | 0.75 | 1,863 | 1,233 | 34 |
| C1-9 | 22.75 | 0.69 | 2,375 | 1,663 | 30 |
| C1-10 | 30.37 | 0.57 | 2,461 | 1,840 | 25 |
| C1-11 | 18.21 | 0.41 | 1,001 | 650 | 35 |
| C1-12 | 26.73 | 0.535 | 2,021 | 1,300 | 36 |
| C1-13 | 10.38 | 0.50 | 743 | 504 | 32 |

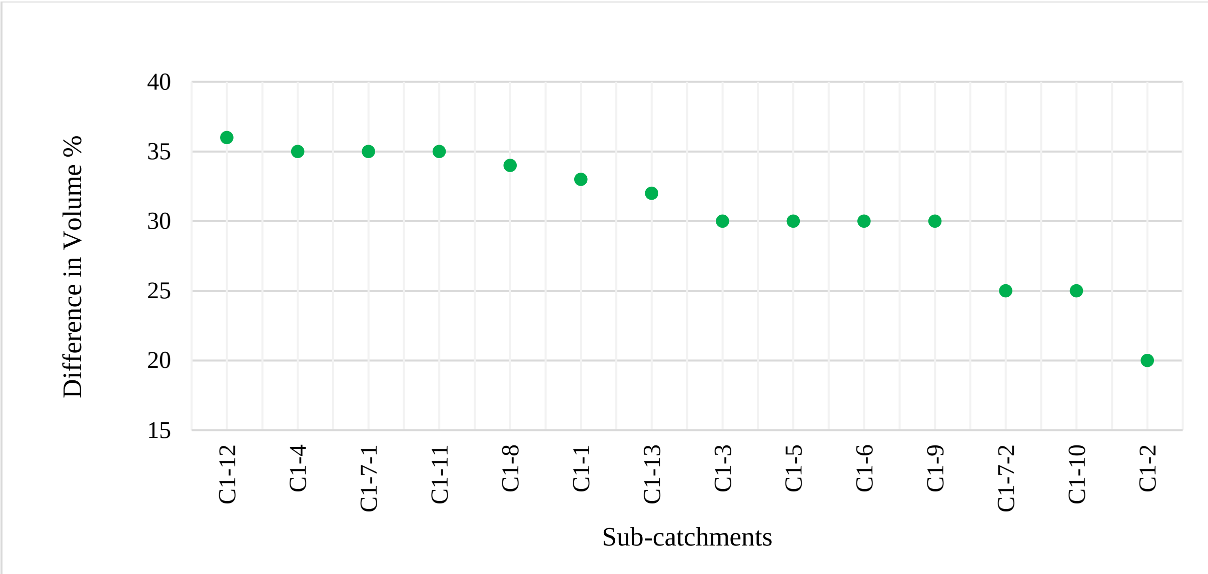



*Figure 6-26- Runoff volume comparison for small sub-catchments without and with raingardens- C2 for one year*

Table 6-25- Runoff volume comparison for small sub-catchments without and with raingardens- C2 for one year

| Sub-Catchments | Area ha | Impervious ratio | Runoff Volume No raingarden ($m^3$) | Runoff Volume with raingarden (28%) | Difference in volume % |
|---|---|---|---|---|---|
| C2-1 | 11.95 | 0.78 | 5,313 | 3,560 | 33 |
| C2-2 | 5.16 | 0.45 | 2,529 | 1,770 | 30 |
| C2-3 | 13.12 | 0.7 | 5,729 | 3,896 | 32 |
| C2-4 | 6.73 | 0.65 | 3,927 | 2,631 | 33 |
| C2-5 | 7.75 | 0.55 | 3,140 | 2,167 | 31 |
| C2-6 | 11.31 | 0.73 | 4,940 | 3,260 | 34 |
| C2-7 | 6.76 | 0.6 | 2,741 | 1,919 | 30 |
| C2-8 | 4.64 | 0.8 | 2,136 | 1,431 | 33 |
| C2-9 | 6.22 | 0.75 | 1,939 | 1,319 | 32 |
| C2-10 | 21.84 | 0.8 | 2,872 | 1,982 | 31 |
| C2-11 | 15.00 | 0.65 | 4,906 | 3,385 | 31 |
| C2-12 | 31.82 | 0.6 | 11,660 | 7,812 | 33 |
| C2-13 | 23.76 | 0.55 | 12,750 | 8,415 | 34 |
| C2-14 | 27.61 | 0.45 | 15,250 | 10,370 | 32 |
| C2-15 | 5.99 | 0.45 | 1,961 | 1,333 | 32 |
| C2-16 | 11.96 | 0.7 | 6,140 | 4,298 | 30 |
| C2-17 | 20.96 | 0.73 | 11,090 | 7,763 | 30 |
| C2-18 | 10.2 | 0.65 | 6,984 | 4,540 | 35 |
| C2-19 | 19.76 | 0.6 | 12,090 | 8,463 | 30 |
| C2-20 | 22 | 0.55 | 10,450 | 7,315 | 30 |
| C2-21 | 20.1 | 0.63 | 8,324 | 5,744 | 31 |
| C2-22 | 12.15 | 0.68 | 4,166 | 2,916 | 30 |

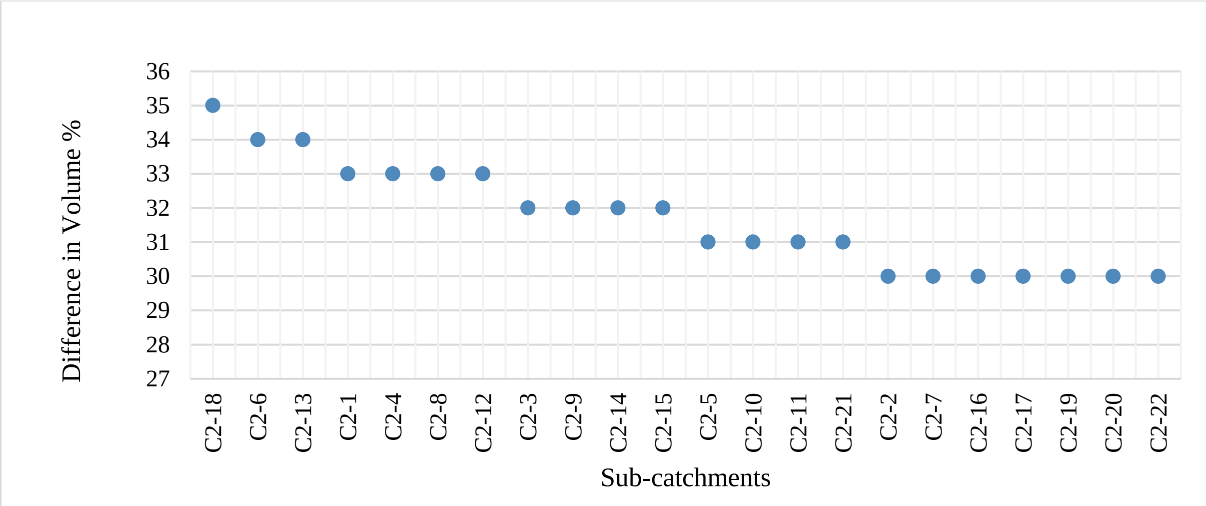



*Figure 6-27- Runoff volume comparison for small sub-catchments without and with raingardens-C2 for over one year with raingardens*

### 6.3.6. MUSIC Model Results

#### 6.3.6.1. Results for one sub-catchment

The selected sub-catchment has an area of 27 ha. By adopting the POT methodology as the selected allotment probability fits were plotted for the selected sub-catchment. Appendix B explains how the raingardens were adopted throughout the selected sub-catchment in MUSIC model. Figure 6-28 shows the results for the selected sub-catchment with and without rainwater tanks and raingardens. Based on the graph, adopting 3-kL rainwater tanks to collect the stormwater from only 28% of the roofs in the selected sub-catchment reduced the peak flow exceedance probabilities minimum up to 3% and maximum up to 8%. 5-kL rainwater tanks also showed the same reduction range. Moreover, raingardens decreased the probability minimum up to 11% and maximum up to 18%. Scenarios were compared with a base case with no rainwater tanks and raingardens. Raingardens showed more effectiveness in peak flow reduction at the selected sub-catchment compared to the rainwater tanks. In addition, they showed 3% to 5% more effectiveness in probability reduction for big events.

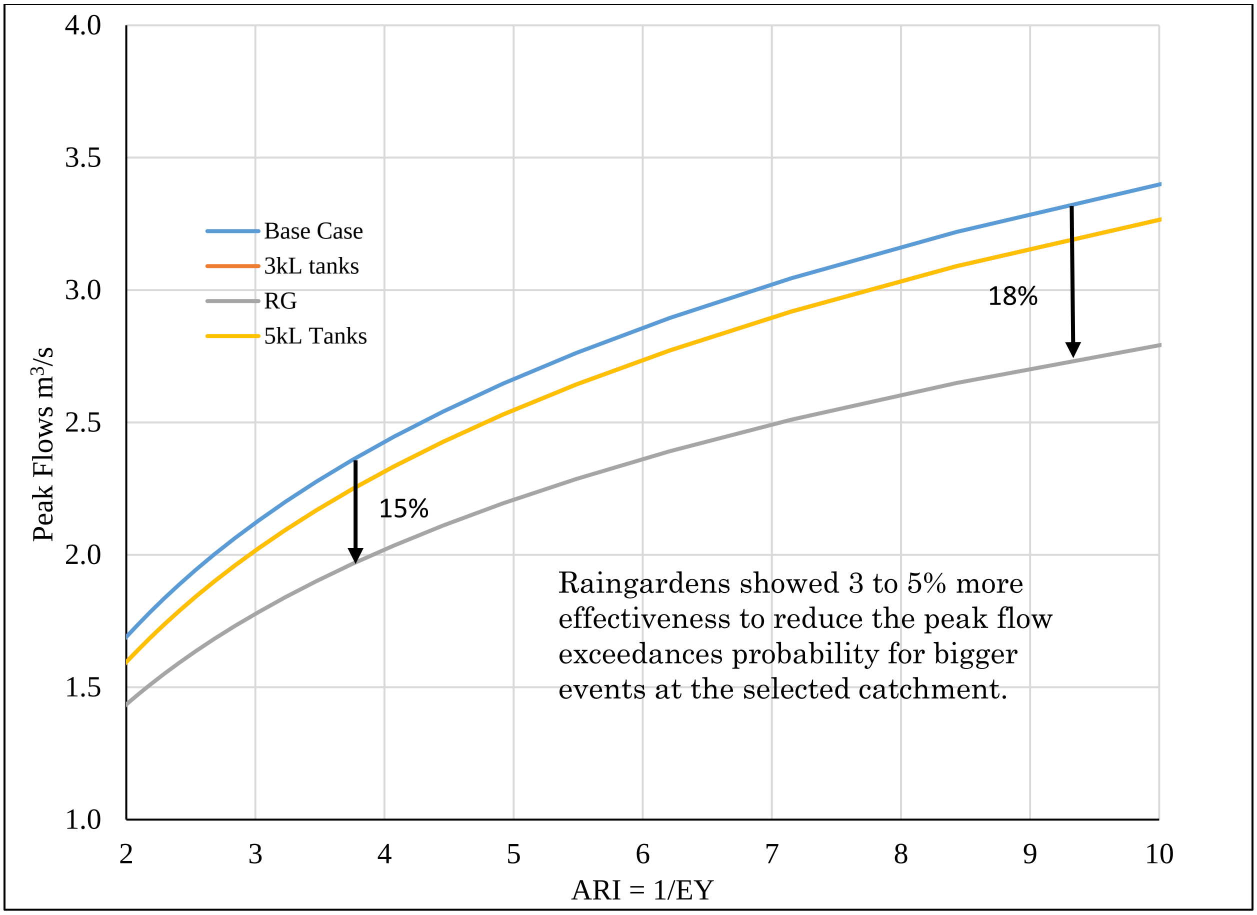



*Figure 6-28- Flow exceedance probability plotted fits– Scenario comparisons (with and without rainwater tanks and raingardens for a selected sub-catchment)*

#### 6.3.6.2. Results for all sub-catchments

By adopting POT methodology, probability fits plots for all small sub-catchments under explained scenarios in section 6.3.4. Figure 6-29 shows the results for the whole area with and without rainwater tanks and raingardens.

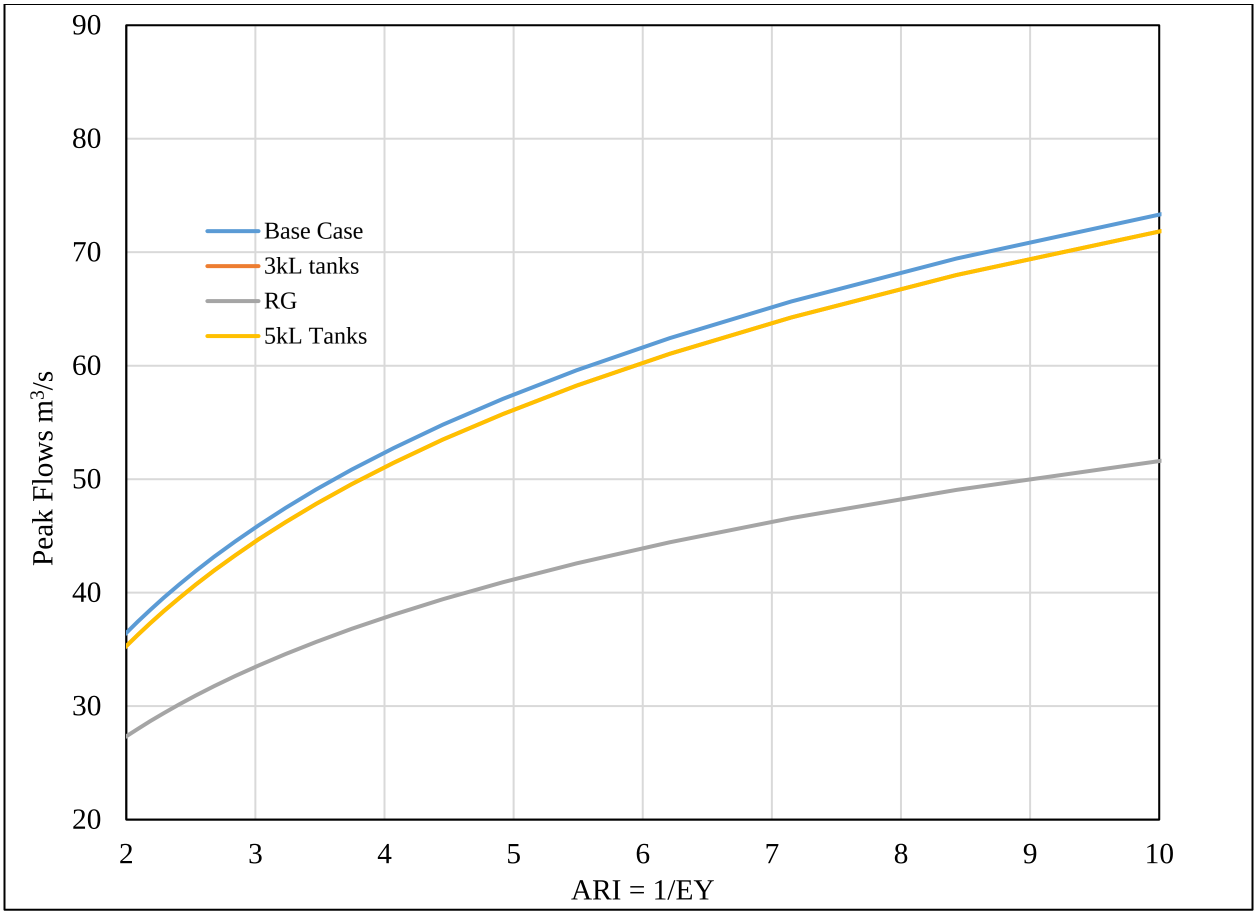



*Figure 6-29- Flow exceedance plotted probabilities per year – Scenario comparisons (with and without rainwater tanks and raingardens for the whole area)*

According to the graph and by adopting rainwater tanks for the whole area, ignoring the size of the tanks, probability of peak flow exceedance reduced minimum up to 2% and maximum up to 6%. In contrast, raingarden reduced the probability minimum by 18% and maximum up to30%. Scenarios were compared with a base case with no rainwater tanks and raingardens. According to the results, it was understood that raingardens showed more effectiveness in peak flows exceedances probability reduction at the whole area compared to the rainwater tanks.

In summary, rainwater tanks and raingardens reduced the probability of peak flow exceedance for the simulated period in all the selected scales. However, rainwater tanks were more effective for reduction at allotment scale and for small events at catchment scales. However, raingardens showed more effectiveness for reduction at the catchment scales and for all range of small and big events.

Totally, Raingardens showed more effectiveness to reduce the probability of peak flow exceedance for all the selected scales.

It should be mentioned that these results are based on the modelling assumptions for this study and are initial findings. Results could be different with other modelling assumptions or by adopting higher percentage of the rainwater tanks to treat more roofs at the catchments.

### 6.3.7. Conclusion

To effectively address the research questions, following the comprehensive literature review and WSUD selection, the primary and most critical step was to evaluate the effectiveness of the selected techniques in managing stormwater quantity across different spatial scales, including allotment and sub-catchment levels. This focus was essential, as existing research has predominantly emphasized stormwater quality rather than quantity, often neglecting real-world catchment conditions and multi-scale assessments. Given the rapid urbanization and increasing pressures of climate change, a deeper understanding of WSUD performance under these dynamic conditions was missed and imperative (according to the literature review). Therefore, this chapter aimed to provide a rigorous assessment of various WSUD techniques, simulating actual urban conditions to offer meaningful insights for sustainable stormwater management in evolving urban landscapes particularly for the quantity aspect.

In summary, the effectiveness of rainwater tanks and raingardens were evaluated on stormwater runoff volume and peak flow size reduction for the study area using MUSIC and PCSWMM models. Initial results are summarised in Figure 6-30 Table 6-26 and 6-27. Reduction of runoff volume and peak flow were used as performance indicators, and results for the indicators were compared with and without rainwater tanks and raingardens.

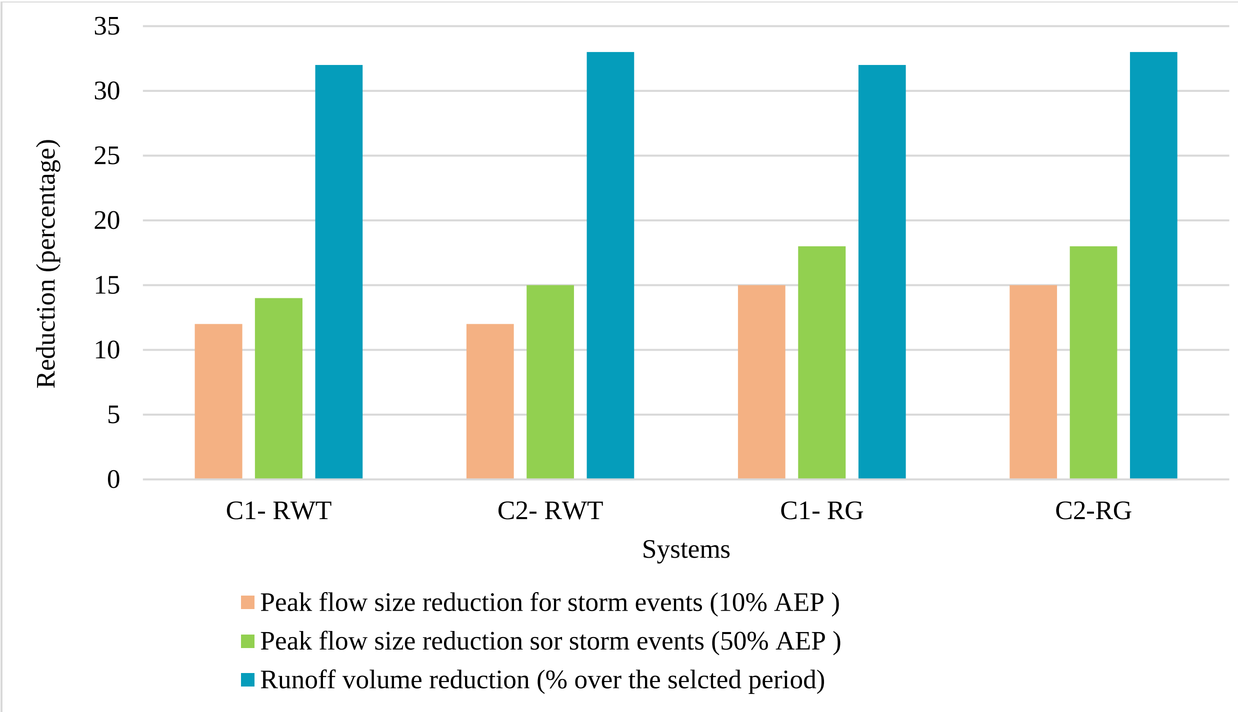



*Figure 6-30- PCSWMM model results for rainwater tanks and raingardens under different storm events*

Table 6-26- Summary of results for stormwater quantity scenario modelling – PCSWMM Model

| **Modelling tool** | **WSUD system(s)** | **Area treated** | **Peak flow size reduction** | **Runoff volume reduction** |
|---|---|---|---|---|
| PCSWMM | 3 and 5 kL rainwater tanks | 28% | Up to 15% at a catchment outlet | Up to 30% over the four months continuous simulation and up to 14% under single storm events |
| | 0.5 m depth raingardens | | Up to 15% at a catchment outlet | Up to 30% over the four months continuous simulation and up to 18% under single storm events |

Table 6-27- Summary of results for stormwater quantity scenario modelling – MUSIC Model

| Modelling tool | WSUD system(s) | Area treated | Peak flow exceedances probability |
|---|---|---|---|
| MUSIC | 3 kL and 5 kL rainwater tanks | 28% | Average up to 7% at catchment scale and 26% at allotment scale |
| | 0.5 m depth raingardens | | Average up to 54% at allotment scale and 29% at catchment scale |

Different climate scenarios were modelled using continuous simulation and individual events. A wet period was considered from 1/12/2010 to 31/03/2011 for a continuous simulation based on the Laverton rainfall station located close to the study area. For the single storm events, short term patterns were exported from BoM Hub data website. In addition, the maximum rainfall was extracted from rainfall IFD data system from BoM website for the study area for 10 % and 50 % AEP events. To define the single storm events, 10 different storm patterns for each exceedance probability were considered.

By adopting the rainwater tanks and raingardens for only 28% of the total areas in small sub-catchments, they reduced the runoff volume up to 30% over the selected period. In addition, 3 kL and 5 kL tanks reduced the peak flow size up to 15% for the selected period at the catchment's outlet. Moreover, raingardens along with 3 kL and 5kL tanks reduced the peak flow for the area up to 18 % for the 10 % AEP events and up to 14% for 50% AEP events.

For the MUSIC model, partial flow analysis method was applied to understand the effectiveness of rainwater tanks and raingardens on the flow peak size reduction for the scales including one sub-catchments and all sub-catchments. Possible flows plotted for the selected areas to understand the catchment behaviour under different rainfall amounts. Probability fits were plotted and compared for defined scenarios. Adopting rainwater tanks in allotment scale showed significant changes to reduce the runoff peak size compared to the selected sub-catchment and all sub-catchments. However, raingardens were better at reducing the runoff peak size for two scales. There was not a significant difference between 3 kL tanks reduction compared to 5kL tanks at a catchment scale. The results in MUSIC showed that raingardens can be more beneficial for peak flow reduction in both allotment and catchment scales for different range of ARI. However, by connecting more roofs to the rainwater tanks the chance of peak flow reduction may increase at the catchment scale.

A review of literature found limited studies on the effectiveness of the rainwater tanks and raingardens at a catchment scale particularly under existing condition, but a good agreement was found between the results of the current study and those in literature. Hardy et al. 2004 investigated the impacts of spatially distributed rainwater tanks at an estate level. The study explored how decentralized rainwater harvesting influences stormwater management, water supply sustainability, and urban hydrology. The research highlighted that rainwater tanks can significantly reduce peak stormwater flows, lessen the burden on drainage infrastructure, and contribute to localized water security. By using PCSWMM model, Walsh et al. 2014 reported maximum long-term watershed volumetric reductions between 10.1% and 12.4% which were observed for the period of analysis (1948–2011) with a range of rainwater harvest storage sizes (227 L barrels to 7571 L cisterns). Also, up to 15% peak flow rate reduction reported for an urban catchment in Brazil by only adopting raingardens to treat 10% of the area (Gonçalves et al. 2018).

# 7. Chapter 7: WSUD techniques Parameters Adopted in PCSWMM Model

## 7.1. Introduction

This chapter delves into the representation of WSUD techniques in the PCSWMM modell, covering design parameters, size, and physical characteristics necessary in PCSWMM. The majority of defining parameters were not chosen for optimization, so they had to be configured in accordance with design standards. The use of multiple WSUD design guidelines was necessary, given the absence of a single comprehensive guideline that encompasses all the required parameters for PCSWMM.

## 7.2. WSUD Representation in PCSWMM Model

WSUD stormwater techniques can be depicted in models in various ways. For instance, the physical processes within the LID control can be represented, or they can be demonstrated using aggregate properties such as curve number (CN) (Ahiablame et al., 2012). Another method is to develop a model for WSUD measures and integrate them into open-source models. Damodaram et al., (2010) demonstrated this by incorporating curve numbers, and Zhang (2009) created physically based algorithms to represent green roofs, porous pavement, and bioretention in SWMM.

Several models, including SWMM, now incorporate a built-in WSUD techniques and LID toolbox for simulating the systems. Zhou (2014) emphasizes the importance of accurately representing the techniques in both modelling and physical design. They highlight that underestimating the complexity of WSUD functionality can result in underachieving their performance and unmet expectations. The next part examines the results of studies employing computer models to analyse the hydrological processes in LID and WSUD techniques.

### 7.2.1. Hydrology

Xiao et al., (2007) specifically focused on lot-level techniques using a model they created. Their findings indicated that the heightened percolation to groundwater had a more significant impact than evapotranspiration. While this increased percolation could aid in groundwater recharge, it is crucial to exercise caution to prevent groundwater contamination when directing runoff to WSUD techniques from paved surfaces in areas with highly permeable soil.

In Matlab, Gilroy and McCuen (2009) created a model to replicate the spatial and temporal characteristics of rainfall and runoff. Their aim was to assess the efficacy of bioretention cells and cisterns in lot-sized Catchment. The study revealed that the LID controls performed significantly better during a one-year storm compared to a two-year storm. To enhance the available storage for such events, the LID controls could be arranged in series along the same flow path.

The design of Low Impact Development (LID) controls has varying impacts on peak flow rate and runoff volume. During the design phase, it is crucial to consider both the timing and volume of runoff. Achieving optimal LID performance is essential, as the addition of extra LID measures may yield diminishing returns, as noted by Gilroy and McCuen (2009).

In a study using Storm Water Management Model (SWMM), Qin et al., (2013) evaluated the effectiveness of swales, permeable pavements, and green roofs in a Chinese catchment prone to heavy summer rainfall. The findings suggested that swales were not efficient in reducing flood volumes because they received runoff from an area that was too large, leading to quick overflow. Permeable pavements and green roofs, on the other hand, were identified as effective in reducing flood volumes for precipitation events ranging from 70 mm to 140 mm. Li et al., (2017) conducted SWMM simulations to compare the performance of bioretention units, green roofs, permeable pavement, low-elevation greenbelt, and rain barrels.

Their study demonstrated that bioretention cells could reduce peak flow by 36% and total runoff by 39%, green roofs reduced peak flow by 25% and total runoff by 30%, permeable pavement reduced peak flow by 18% and total runoff by 23%, low-elevation greenbelt reduced peak flow by 26% and total runoff by 30%, and rain barrels were able to reduce peak flow and total runoff by 6% and 11%, respectively.

Palla and Gnecco (2015) investigated LID controls at the catchment scale using SWMM and discovered that hydrological performance was linearly dependent on the reduction of impervious area. They reported that a reduction greater than 5% was necessary to achieve noticeable benefits. The retention capacity of LID measures drove improvements in hydrological performance.

Simpson and Roesner (2018) employed SWMM to evaluate whether LID measures alone could preserve predevelopment hydrology. It was determined that while LID measures can restore predevelopment hydrology, extensive implementation is necessary to manage a 100-year storm, making it economically impractical to do so. Zhang et al., (2016) estimated the hydrological effects of common LID controls by employing the Soil Conservation Service (SCS) method to simulate runoff-generating processes for various rainfall frequencies. They utilized the simulation results to examine factors influencing LID performance, including the impact of LID controls on reducing runoff and increasing baseflow.

Trinh and Chui (2013) employed MIKE SHE for simulations and emphasized the significant roles of groundwater and evapotranspiration in hydrological systems. They highlighted that proper planning and design of LID controls in an urban catchment can alter the shape of the outlet hydrograph. Stewart et al., (2017) created a HYDRUS-2D/3D model for a bioretention cell, employing it to establish a mass balance for reducing stormwater return flow, evaluate the impact of Low Impact Development (LID) measures on subsurface water dynamics, and assess the model's sensitivity to measured soil properties. The model illustrated that the bioretention cell effectively decreased stormwater return flows into the sewer system and introduced a new groundwater dynamic through the additional exfiltration from the bioretention unit.

Schmitter et al., (2016) developed an integrated model to assess the effects of implementing green roofs in Singapore, highlighting positive impacts on flood protection. This research emphasizes the effectiveness of green roofs in climates with two monsoon seasons. Sun et al., (2014) modelled and analysed the application of Low Impact Development (LID) controls in a parking lot in Kansas, showcasing that LID measures effectively controlled stormwater during small rainfall events but exhibited limitations during flooding events. Chaosakul et al., (2013) simulated both single and multiple Low Impact Development (LID) controls in an urban village in Thailand to assess the benefits in terms of stormwater quantity and quality. The study found that the combination of rain barrels and bioretention cells resulted in the most significant reduction in surface flooding. However, the implementation costs associated with this approach were likely deemed impractical for the region.

Semadeni-Davies et al., (2008) employed the Model of Urban Sewers (MOUSE), a commercial tool from the Danish Hydrological Institute, to simulate the combined sewer system in Helsingborg, Sweden. Their study demonstrated that, under future climate scenarios, the implementation of Low Impact Development (LID) controls, along with the separation of stormwater from combined sewers, could potentially reduce or eliminate combined sewer overflows (CSO). Freni et al., (2010) devised their own model to assess the efficacy of decentralized Low Impact Development (LID) controls in comparison to traditional centralized stormwater controls. Their aim was to analyse approaches for enhancing stormwater management practices. They examined an urban catchment at the University of Palermo in Italy and discovered that storage tanks linked to centralized systems were more successful in reducing Combined Sewer Overflow (CSO) volume and pollutant load, as storage tanks directly impact potential CSO volume. Distributed infiltration techniques may be more effective for soils with high infiltration rates; however, the efficiency of these controls can be compromised over time due to clogging, necessitating regular maintenance.

Integrating a combination of distributed and centralized stormwater management controls can be practical and efficient. Massoudieh et al., (2017) similarly devised their own modelling framework to evaluate the hydrological and water quality processes in Low Impact Development (LID) controls and applied it to various LID studies.

Stovin et al., (2013) developed a GIS-based tool for modelling and analysing retrofit Low Impact Development (LID) controls. LID measures were represented by disconnecting sections of catchments from the sewer system, either by routing them to pervious areas or by developing pervious areas, rather than individually modelling each LID measure. They applied the model to three catchments in the London Tideway Improvements area. Testing scenarios, such as disconnecting downspouts from sewers that carry stormwater away from storm sewers, proved to be an effective method for assessing the potential for LID implementation. They concluded that large-scale disconnection would be challenging and expensive to implement, proposing instead that LID controls should be employed as a tool alongside conventional sewer system (Stovin et al., 2013).

Satellite imagery proves to be an invaluable tool for GIS-based models. In Khin et al., (2016), high-resolution satellite imagery from WorldView-2, combined with a two-stage classification method, was employed to acquire land use cover types, and derive hydrologic parameters for modelling LID performance. The utilization of satellite imagery offers an automated approach for obtaining land cover information when modelling LID techniques within the urban drainage system.

The classified image served as the foundation for creating three LID scenarios within detailed distributed hydrologic models using PCSWMM. Essential hydrologic parameters for each sub-catchment, including width, area, percent imperviousness, overland flow path, Manning’s n value, and depression storage value, were derived from the classified results and incorporated into the

GIS software. This approach proves highly advantageous in situations where high-quality GIS data for land cover types may be lacking.

Computer modelling tools play a crucial role in aiding the evaluation and assessment of feasible LID options. These tools enable the identification of cost-effective LID placement options within a watershed, addressing the restoration and protection of environmental quality in both developed and developing urban areas (Lee et al., 2012).

Chui et al., (2016) integrated SWMM with Matlab to assess the hydrological performance and cost-effectiveness of bioretention, green roofs, and porous pavements, aiming to identify optimal designs. LID controls were modelled as vertical layers, with their movements and water balances computed for each layer during simulation. Matlab was programmed to automate the analysis of numerous SWMM output files. This approach allowed for the calculation of unit costs and dimensions for the optimal designs of various LID measures.

Liu et al., (2016) employed L-THIA-LID 2.1, coupled with an optimization tool, to simulate the effects of climate change and land use change on hydrology and water quality in an Indiana watershed. Their objective was to identify the optimal selection and placement of LID practices. The study found that, by 2050, land use changes had a more significant impact on runoff volume and pollutant loads compared to the anticipated effects of climate change.

Son et al., (2017) introduced a LID-based district unit planning model (LID-DP) and validated it through simulation tests conducted in a Korean city. The results showcased the effectiveness of LIDs in mitigating runoff and positively influencing water quality, particularly in an area prone to heavy rainfall events during the summer months.

Non-structural LID practices, such as clustered development, were investigated using computer simulations. Brander et al., (2004) compared conventional curvilinear, urban cluster, coving, and new urbanism development methods, both with and without infiltration-based LID controls. Their model, Infiltration Patch, is a spreadsheet-based extension of the National Resources Conservation Services SCS CN method. Like many other studies on LID controls, the effectiveness of these measures was more pronounced for smaller storms. Cluster development, which allows more open spaces, proved to be the most effective in reducing runoff. Williams and Wise (2006) concluded that cluster development is valuable for mitigating peak flows and runoff volume, especially when coupled with LID controls to preserve natural hydrological patterns.

### 7.2.2. Water Quality

As mentioned earlier, water quality considerations related to LIDs are not as commonly modelled as hydrology, and experimentation is often more prevalent in such studies. Ahiablame et al., (2012) highlight the need for further research on characterizing runoff water quality for different land uses. Guo et al., (2014) developed the Water Quality Capture Optimization and Statistic Model (WQ-COSM) to assist in determining the water quality capture volume for the design of LID controls. Chen and Lin (2015) employed the SUSTAIN model to simulate LID measures and assess water quality performance in a watershed in Taiwan. The study aimed to identify best practices for application in the watershed. The effectiveness of grass swales, bioretention, and pervious pavements was compared for the removal of TP, SS, TN, and BOD. The findings indicated that permeable pavements delivered the most significant reduction in pollution and runoff.

Mao et al., (2017) utilized SUSTAIN to assess the ecological benefits of LID measures in a Chinese city. Their evaluation of annual pollutant loads, including COD, SS, TN, and TP, revealed that LIDs achieved a reduction of over 60% in pollutant loads. Seo et al., (2017) developed a procedure to incorporate LID practices into SWAT, allowing them to model the impacts of LIDs on both water quantity and quality. Their model demonstrated how LID controls can effectively reduce pollutant loads across various land uses.

Carbone et al., (2014) employed laboratory experiments to validate their k-C* model, designed for simulating permeable pavement systems. The model accurately predicted TSS concentration in runoff for various permeable pavement types. Li et al., (2017) utilized SWMM to analyse the pollutant removal capabilities of LIDs. Bioretention was found to reduce COD, SS, TN, and TP by 28.3%, 34.5%, 36.1%, and 33.7%, respectively. Green roofs reduced these pollutants by 19.1%, 22.9%, 24.6%, and 22.6%, while permeable pavements achieved reductions of 13.3%, 16.0%, 17.4%, and 15.8%.

### 7.2.3. Multi- Criteria Modelling

The use of spatial multi-criteria analysis is crucial in the planning and execution of LIDs. There is a wealth of literature exploring various methods and models to assist in the selection, sizing, and placement of LID measures. Jia et al., (2013) introduced a multi-criteria index system for guiding the selection of LID controls during the planning phase.

The criteria consider specific site characteristics, site suitability, economic feasibility of LID implementation, and the performance of runoff controls. Regarding models that aid in the selection of LID controls, Charlesworth et al., (2016) designed a large-scale, site-specific model to identify the optimal treatment train.

The model integrates ArcGIS maps to enhance the decision-making process for selecting appropriate LID controls. This model proves valuable in the initial planning phases by pinpointing optimal locations for LID implementation. Additionally, Johnson and Sample (2017) created the BMP Checker, simplifying the identification of site locations for Best Management Practices (BMP) devices.

The BMP Checker utilizes site characteristics such as slope, seasonal high water table depth, soil types, and catchment size, comparing them to specific constraints. Through this analysis, the model offers recommendations on suitable and unsuitable Best Management Practices (BMPs) for implementation. In a similar vein, Joyce et al., (2017) employed scale-dependent data and the Interconnected Channel and Pond Routing model to create a multi-scale modelling platform for assessing drainage infrastructure. Tailored for coastal regions, this model underscored the importance of factors like rainfall type, sub-daily rainfall patterns, and groundwater analysis in the comprehensive evaluation of LID implementation.

## 7.3. Design Strategy

The size, physical specifications, and placement of the systems as modelled in the PCSWMM model for allotments and sub-catchment scales were discussed in this chapter. The major goal was to identify and discuss the constraints of the optimisation strategies that were modelled. The remaining parameters were chosen based on the findings of the literature review and the design guidelines. Appendix A contains cross-section sketches of the techniques for reference.

### 7.4. Rainwater tanks (Rain Barrels)

Although the PCSWMM model uses the term “rain barrel,” this thesis uses the term “rainwater tank” because it is more often used in Australia. The most commonly used rainwater tank sizes in Melbourne range from 2 to 3 kL, but the overall average capacity, influenced by larger tanks, is 4.3 kL., as stated in section 6.2.1. (Moglia et al., 2014). As a result, 3 kL and 5 kL tanks were selected for scenario modelling in allotments in this study. The PCSWMM model does not include a utilisation choice for rainwater tanks, although flow can be drained out of the technique (Walsh et al., 2014). Equation 7-1 governs the outflow from the tanks:

$$q = C(D - Hd)^n \qquad (7-1)$$

where, $H_d$ is the drain height in mm, D is the stored water height in the drainage layer in mm and q is the outflow velocity through the underdrain in mm/hr. C and n are adjustable and determine the flow rates. The value for n has been given 0.5 by Rossman (2015) as a typical value for the tanks. C is a function of drain time (T) and the stored water depth in the tank (D). The drain time is the required time to drain out the stored depth in the tank. C can be calculated using Equation 7-2:

$$C = \frac{2(D^{\,0.5})}{T} \qquad (7-2)$$

D and T have been defined in units of inches and hours in the PCSWMM model. To calculate the C value and using values for D and T in mm, equation 7-2 was modified to Equation 7-3.

$$C = \frac{2[25.4(D)]^{\,0.5}}{T} \qquad (7-3)$$

Using the abovementioned equation, rainwater tanks parameters were calculated and shown in Table 7-1. The drain delay parameters were set to 24 hours assuming the homeowners are unlikely to drain the tanks within 24 hours after any rainfall. The number of tanks per houses was optimized. Then the number of tanks was determined by multiplying the number of tanks per households by the number of households that implement systems in each sub-catchment. Drain offset height was set as zero assuming the orifice will be at the bottom of the tank. Captured stormwater from the tank's outlet pipe any overflow was routed to pervious areas. Rainwater tanks implementation parameters are shown in Table 7-2.

Table 7-1-Rainwater Tanks Design Parameters

| **Parameters** | **Description** | **Value** | **Source** |
|---|---|---|---|
| Storage layer | | | |
| Height (mm) | Height of the tank | 2000 for 3kL<br>3000 for 5kL | Real tank measurement |
| Underdrain | | | |
| Drain coefficient I | Parameter that can be adjusted and determine the flow rate. Equation 6-1. | 18.78 for 3kL<br>23 for 5kL | - |
| Drain exponent (n) | Parameter that can be adjusted and determine the flow rate. Equation 6-1. | 0.5 | Rossman (2015) |
| Drain offset height (mm) | The height of the drain above the bottom of the storage layer, $H$d | 0 | Design choice |
| Drain delay (hours) | The dry period time required for the normal rain barrel to be opened | 24 | Assumption |

Table 7-2- Rainwater Tanks Implementation Parameter

| **Parameters** | **Description** | **Chosen Value 3kL** | **Chosen Value 5kL** | **Source** |
|---|---|---|---|---|
| No. of Tanks | The number of tanks in allotments | 1 | 1 | - |
| Area of each tank ($m^2$) | Total surface area of a tank | 3kL:0.15 | 5kL:0.25 | Volume divided by height |
| Treated Impervious Area | Allotment impervious area directed to the tank | Roof area | Roof area | Table 7-11 |
| Percent Initially Saturated | Percent of the rainwater tank which is filled at the start of simulation | 0 | 0 | - |

## 7.5. Raingardens

Raingardens are similar to bio-retention techniques except they don't have an underlying drainage layer. Implementation of bio-retention is much easier in a new development area while raingarden's implementation and retrofitting would be of household's interest in developed areas as bio-retention techniques design and construction is more complex. The research study area is well-developed and there were not many available spaces for new development therefore, raingardens were considered for this research for adoption on household and street scales for stormwater quality and quantity management. It was assumed that the systems will be designed to be slightly depressed relative to the landscapes around them to increase the surface storage. This will provide larger depression storage and collect runoff from their surrounded surfaces.

The roughness coefficient of the techniques was set based on the SWMM user's manual (Rossman, 2015) according to the ground cover that will be used for them in the future. The systems won't be used to transmit overland flow therefore, their roughness and slope can be set to zero (CHI support, 2018).

There are three defined layers for the raingardens in PCSWMM model including soil, storage, and surface layers. The soil layer parameters were adopted using the recommended parameters in the literature review for Australia. The soil of the research study area is identified as heavy clay so should be imported due to the poor drainage characteristics of the current underlying soil. The storage layer parameters represent the bottom storage layer which was assumed as crushed stone or gravel. Seepage rate or the saturated hydraulic conductivity of the surrounding area is the only adjustable parameters for the raingardens. Clogging factors were set to zero as they would not be a significant factor for the performance of the system (DPLG 2010; Richards et al., 1949). Surface layer parameters also were set using available raingarden's data according to the local government guidelines and existing systems. Table 6-3 has been summarized the raingardens design parameters for the layers. Table 7-4 presented the parameters for raingardens implementation in PCSWMM model.

Table 7-3- Raingardens Design Parameters

| Parameters | Description | Value | Source |
|---|---|---|---|
| **Surface layer** | | | |
| Storage depth (mm) | The height of the surface depression storage | 200 | Existing local guidelines, PCSWMM model Use Manual |
| Vegetation volume (fraction) | The fraction of the volume within the storage depth which is occupied by vegetation | 0.15 | |
| Surface roughness (Manning's n) | Roughness for overland flow on the surface of the WSUD | 0 | |
| Surface slope (%) | Slope of the WSUD surface | 0 | |
| **Soil Layer** | | | |
| Thickness (mm) | Height of soil layer | 0.5 | Existing local guidelines, PCSWMM model Use Manual |
| Porosity (volume fraction) | The volume of pore space divided by the total volume | 0.5 | |
| Field Capacity (volume fraction) | The volume of pore water remaining in the soil after the soil has drained fully | 0.28 | |
| Wilting point (volume fraction) | The volume of pore water relative to the total volume of well dried soil. | 0.17 | |
| Conductivity (mm/hr) | Saturated hydraulic conductivity for the soil layer | 100 | |
| Conductivity slope | Slope of the curve of the log graph of conductivity versus the soil moisture content | 6 | |
| Suction Head (mm) | The average soil capillary suction along the wetting front | 0.17 | |
| **Soil Layer** | | | |
| Height (mm) | Height of the storage layer | N/A | Existing local guidelines, PCSWMM model Use Manual |
| Void ratio (V voids/V solids) | The volume of void space relative to the volume of soils layer | N/A | |
| Seepage rate (mm/hr) | The maximum rate that water is allowed to infiltrate into native soils | 3.6 | |
| Clogging factor | The volume of runoff needed to clog the bot tom layer divided by the void volume. | 0 | |

Table 7-4- Raingardens Implementation Parameters

| Parameters | Description | Chosen Value | Source |
|---|---|---|---|
| No. of Raingardens | The number of raingardens in allotment | 1 | - |
| Area of each tank ($m^2$) | Total surface area of a raingarden | 5-7 $m^2$ | - |
| Surface width of each unit (m) | The width of the outflow side of each raingarden unit | 5 | Design choice based on available space |
| Percent Initially Saturated | The initial condition of the unit's soil in terms of water content. The underlying storage zone is assumed to be dry | 50 | - |
| Percent Impervious area treated | The percentage of the impervious area in an allotment whose runoff is directed to raingarden | Roof area | Table 6-11 |

## 7.6. Infiltration Trench

Infiltration trenches which are shallow depressions, facilitate infiltration and are filled with stone for temporary runoff storage. They can easily be incorporated into a landscape to capture stormwater runoff to allow to infiltrate to the surrounding area which significantly reduces the urban stormwater volume and flow rate (Woods-Ballard et al., 2007). These techniques are suitable to be implemented in areas with underlying soils which have poor hydrologic properties. Design parameters of infiltration trenches in PCSWMM model are shown in Table 7- 5. There are the layers for the techniques in the model including surface, storage, and underdrain.

Table 7-5- Infiltration Trench Design Parameters

| Parameters | Description | Value | Source |
|---|---|---|---|
| **Surface layer** | | | |
| Berm Height (mm) | The height of the surface depression storage | 300 | Existing local guidelines, PCSWMM model Use Manual |
| Vegetation Volume (fraction) | The fraction of the volume within the storage depth which is occupied by vegetation | 0 | |
| Surface Roughness (Manning's n) | Roughness for overland flow on the surface of the WSUD | 0 | |
| Surface Slope (%) | Slope of the WSUD surface | 0 | |
| **Storage Layer** | | | |
| Thickness (mm) | Thickness of the storage | 1000 | Existing local guidelines, PCSWMM model Use Manual |
| Void Ratio | The volume of void space relative to the volume of soils layer | 0.4 | |
| Seepage Rate (mm/hr) | The maximum rate that water is allowed to infiltrate into native soils | 3.6 | |
| Clogging Factor | The volume of runoff needed to clog the bottom layer divided by the void volume. | 0 | |
| **Underdrain** | | | |
| Drain coefficient I | Used to determine the flow rate through the underdrain as a function of stored water height | 0.83 | Existing local guidelines, PCSWMM model Use Manual |
| Drain exponent (n) | Makes the drain act as an orifice | 0.5 | |
| Drain offset height (mm) | Height of underdrain piping about the bottom of the storage layer | 150 | |

The implementation parameters for infiltration trenches are presented in Table 7-6. Two parameters were optimized for the techniques which are the number of units and the area.

Table 7-6- Infiltration Implementation Parameters

| Parameters | Description | Chosen Value | Source |
|---|---|---|---|
| No. of trenches | The number of infiltration trench in a sub-catchment | Optimized | - |
| Area of each tank ($m^2$) | Total surface area of a unit | 5-6 $m^2$ | - |
| Surface width of each unit (m) | The width of the outflow side of each WSUD unit | 2 | Design choice based on available space |
| Percent Initially Saturated | The initial condition of the unit's soil in terms of water content. The underlying storage zone is assumed to be dry | 10 | - |
| Percent Impervious area treated | The percentage of the impervious area in allotment whose runoff is directed to this WSUD type | Driveway area | Table 7-11 |

## 7.7. Permeable Pavements

Permeable pavements reduce the urban stormwater runoff as they include an underlying soil layer and let the runoff drain into the layer than becoming runoff. They can be implemented in driveways, low traffic roads, and parking lots. They have been considered in this research as a retrofit to paved driveways. The average size of a paved driveway was estimated at $80m^2$ according to Google Earth for the research study area. Therefore, the techniques can be implemented in driveways to convert some part of the impervious areas to a pervious area to reduce urban stormwater runoff.

Permeable pavements which are also known as porous pavements have an additional layer to be defined in the PCSWMM model compared with other WSUD techniques. The parameters for the systems are outlined in Table 7-7. Clogging can be a frequent concern with permeable pavements however, it was set as zero as the actual impact of clogging cannot be seen in the model due to the short simulation time. It was assumed that the techniques storage is made up of three layers including bedding, stone base and subbase with a total depth of 500mm (McPhee, 2019).

Table 7-7- Permeable Pavement Design Parameters

| Parameters | Description | Value | Source |
|---|---|---|---|
| **Surface layer** | | | |
| Storage depth (mm) | The height of the surface depression storage | 4 | Existing local guidelines, PCSWMM model Use Manual; McPhee, 2019) |
| Vegetation volume (fraction) | The fraction of the volume within the storage depth which is occupied by vegetation | 0 | |
| Surface roughness (Manning’s n) | Roughness for overland flow on the surface of the WSUD | 0.014 | |
| Surface Slope (%) | Slope of the WSUD surface | 2 | |
| **Storage Layer** | | | |
| Thickness (mm) | Thickness of the surface | 80 | Existing local guidelines, PCSWMM model Use Manual; McPhee, 2019) |
| Void Ratio | Related to materials used | 0.4 | |
| Permeability (mm/hr) | Permeability through the paving joints | 0.9 | |
| Impervious surface fraction | The fraction of the area of the permeable pavement that is impervious | 4000 | |
| Clogging Factor | The amount of pavement void volumes of runoff to completely clog the pavement | 100 | |
| **Underdrain** | | | |
| Drain coefficient I | Used to determine the flow rate through the underdrain as a function of stored water height | 12 | Existing local guidelines, PCSWMM model Use Manual; McPhee, 2019) |
| Drain exponent (n) | Makes the drain act as an orifice | 0.5 | |
| Drain offset height (mm) | Height of underdrain piping about the bottom of the storage layer | 50 | |
| **Storage Layer** | | | |
| Height (mm) | Height of the storage layer | 500 | Existing local guidelines, PCSWMM model Use Manual; McPhee, 2019) |
| Void Ratio | The volume of void space relative to the volume of soils layer | 0.4 | |
| Seepage Rate (mm/hr) | The maximum rate that water is allowed to infiltrate into native soils | 0.36 | |
| Clogging factor | The volume of runoff needed to clog the bottom layer divided by the void volume. | 0 | |

Parameters for the techniques Implementation is presented in Table 7-8. The number of the techniques was optimized equal to the household's numbers adopting them.

Table 7-8- Permeable Pavement Implementation Parameters

| Parameters | Description | Chosen Value | Source |
|---|---|---|---|
| No. of Permeable Pavements | The number of permeable pavements in an allotment | 1 | - |
| Area of each pavement ($m^2$) | Total surface area of a unit | 65 | Design choice |
| Surface width of each unit (m) | The width of the outflow side of each WSUD unit | 6 | Estimation |

## 7.8. Vegetated Swales

Vegetated swales are open and shallow channels that prompt infiltration of the stormwater into the ground while they filter the urban stormwater runoff and improve the quality. They include a vegetation part, usually thick grass, to trap pollutants and reduce the stormwater runoff velocity. They can be implemented in new developments or be considered as a retrofit. The techniques can be implemented in parking lots or along with a road in place of curb and gutter systems. They have been designed and modelled to be retrofitted in sub-catchments in this study. There is one layer for them in the PCSWMM model as the surface layer. Table 7-9 presented the design parameters for vegetated swales and Table 7-10 presented the implementation parameters of the system. The number of the swales and their area was optimized according to the number of allotments in sub-catchments.

Table 7-9- Vegetated Swales Design Parameters

| Parameters | Description | Value | Source |
|---|---|---|---|
| **Surface layer** | | | |
| Berm Height (mm) | The height of the surface depression storage | 100 | Existing local guidelines, PCSWMM model Use Manual; McPhee, 2019) |
| Vegetation volume (fraction) | The fraction of the volume within the storage depth which is occupied by vegetation | 0.5 | |
| Surface roughness (Manning's n) | Roughness for overland flow on the surface of the WSUD | 0.014 | |
| Surface Slope (%) | Slope of the WSUD surface | 2 | |
| Swale Side Slope (run/rise) | Side slopes | 3:1 | |

Table 7-10- Vegetated Swales Implementation Parameters

| Parameters | Description | Chosen Value | Source |
|---|---|---|---|
| No. of swales | The number of swales in an allotment | 1 | - |
| Area of each pavement ($m^2$) | Total surface area of a unit | 5-7$m^2$ | - |
| Surface width of each unit (m) | The width of the outflow side of each WSUD unit | 3 | Design choice |

## 7.9. WSUD Combination

The combination of the chosen WSUD techniques and the percent impervious treated individually or in a combination in allotments is outlined in Table 7-11. The table breaks down the treated runoff percentage which is routed to each technique in the chosen typical allotment. The rainfall that falls on each sub-catchment and the runoff percentage forms the technique's inflow. Overflows from the WSUD techniques were directed to the previous areas or the allotments outlets. There was no treatment train assumed meaning the overflows from techniques cannot be directed to other technique.

It's impossible for WSUD techniques to collect all generated runoff so, the total percentage of the captured stormwater is not equal to 100%. The presented values were considered as estimated roof size, driveway area and available spaces for the allotments in sub-catchments.

Table 7-11-Percentage of routed runoff to the chosen WSUD techniques from impervious areas

| Techniques | Impervious Treated (%) | | | | | | Explanation |
|---|---|---|---|---|---|---|---|
| | RWT | RG | IT | PP | VS | Sum | |
| Rainwater tanks (RWT) | 49 | 0 | 0 | 0 | 0 | 49 | Capturing entire roof |
| Raingarden (RG) | | 49 | | | | 49 | Capturing entire roof |
| Infiltration trench (IT) | | | 16 | | | 16 | Capturing driveway |
| Permeable Pavement (PP) | | | | 16 | | 16 | Capturing driveway |
| Vegetated Swale (VS) | | | | 16 | | 16 | Capturing driveway |
| RWT+RG | 24 | 25 | | | | 49 | RWT routed to pervious area and RG capturing half of roof |
| RWT+IT | 24 | | 25 | | | 49 | RWT routed to pervious area and IT capturing half of roof |
| RWT+PP | 24 | | | 25 | | 49 | Capture the roof |
| RWT+VS | 29 | | | | 20 | 49 | Capture the roof |
| IT+PP | | | 20 | 16 | | 36 | IT from roof and PP for the driveway |
| RG+PP | | 20 | | 16 | | 36 | RG from roof and PP for the driveway |
| RWT+RG+IT | 25 | 20 | 16 | | | 61 | Capture roof and driveway |
| RWT+RG+PP | 25 | 20 | | 16 | | 61 | Capture roof and driveway |
| RWT+RG+IT+PP | 25 | 20 | 8 | 8 | | 61 | Capture roof and driveway |

## 7.10. WSUD Chosen for Sub-catchments

Considering the impacts of chosen WSUD techniques at the typical allotment on peak flow size and stormwater runoff volume reduction, literature review and common WSUD techniques in Australia's urban catchments, permeable pavement, infiltration trench, and bioretention techniques were examined in optimisation task in this study. Technique's specifications are summarised as they were adopted in sub-catchments in Table 7-12.

Table 7-12- Specification of WSUD techniques adopted in sub-catchments

| Layer | Parameter | Bioretention | Infiltration Trench | Permeable Pavement |
|---|---|---|---|---|
| Surface | Area ($m^2$) | 45 | 46 | 460 |
| Growing media | Depth (cm) | 50 | - | - |
| | Porosity | 0.5 | - | - |
| | Conductivity (cm/h) | 100 | - | - |
| | Suction head (cm) | 0.17 | - | - |
| Pavement | Thickness (cm) | - | - | 11 |
| | Void ratio | - | - | 0.2 |
| | Permeability (mm/h) | - | - | 4000 |
| Storage | Height (cm) | 30 | 90 | 31 |
| | Void ratio | 0.5 | 0.5 | 0.7 |
| | Conductivity (cm/h) | 3.6 | 3.6 | 3.6 |
| Underdrain | Drain coefficient (cm/h) | 1.3 | 1.3 | 12 |
| | Drain exponent | 0.5 | 0.5 | 0.5 |

This chapter discussed the techniques specifications, their application and the number of systems applied in sub-catchments.

## 7.11. Conclusion

Accurately simulating the chosen WSUD techniques under real-world urban conditions was one fundamental aspect of this study. Implementing these techniques at different spatial scales to manage stormwater runoff effectively required a robust modelling approach. This chapter provides a comprehensive examination of how WSUD techniques are integrated into the PCSWMM model, ensuring that their representation aligns with real-world hydrological and hydraulic behaviour.
A critical focus of this chapter was the translation of WSUD design parameters, including size, physical constraints, and operational requirements, into the PCSWMM modelling framework. The chapter outlines the methodology for constructing simulation scenarios based on developed case studies, ensuring that the modelled performance of these techniques reflects actual urban drainage conditions as closely as possible.

Furthermore, the chapter presents a systematic review of the hydrological and water quality modelling methods incorporated into PCSWMM, highlighting the key factors that influence stormwater quantity and pollutant load estimation. It also details the combination of stormwater control measures (SCMs) and their effectiveness at both allotment and sub-catchment scales, offering insights into their feasibility and optimization.

Ultimately, this chapter serves as a valuable reference for future research focused on stormwater management in existing urban environments, providing guidance on how to simulate WSUD techniques in a manner that closely mirrors real-world performance. By refining the representation of these techniques within PCSWMM, this study contributes to more reliable and effective urban stormwater modelling, facilitating improved planning and decision-making for sustainable drainage solutions.

# 8. Chapter 8: Optimisation

## 8.1. Introduction

Implementing WSUD techniques in urban areas involves several critical considerations, including the selection, quantity, size, and strategic placement of these measures within a sub-catchment to achieve optimal benefits. The hydrologic characteristics of urban catchments can vary significantly, influencing the performance of WSUD techniques depending on their location. Therefore, a key objective of this study's optimization process was to identify the most effective combination and placement of WSUD techniques to maximize runoff reduction and pollutant load mitigation (e.g., TSS). Given the ongoing challenge of stormwater management costs, ensuring maximum efficiency at minimal expense is essential. Another crucial aspect is identifying cost-effective solutions. Consequently, the optimization process aims to determine the most beneficial WSUD configurations across different budget levels, enabling decision-makers to select solutions that align with their specific needs and financial constraints.

## 8.2. Green-Plan IT Tool

This study examined various planning and optimization tools, including UrbanBEATS and Optimatics, to evaluate their suitability for optimization in alignment with the research objectives. However, GreenPlan-IT was selected as the primary optimization tool due to its compatibility with PCSWMM and its alignment with the specific goals of this research.

Green-Plan IT is a planning programme developed by San Francisco Estuary Institute (SFEI) to help planners and engineers with the cost-effective selection and placement of WSUD techniques in urban catchments. GreenPlan-IT includes a green infrastructures site locator tool based on a geographic information system (GIS), a modelling tool, an optimisation tool, and an implementation tracking and reporting tool (Wu et al., 2019). For this research, only the optimisation tool has been used.

The modelling tool, which has been built based on the USEPA's SWMM5 software was used to establish baseline flow and pollutant loads as well as evaluate any reductions brought about by the implementation of WSUD in various catchment locations. The modelling tool in GreenPlan-IT is a subroutine of the optimisation tool. The optimisation tool instructs the modelling tool to evaluate

WSUD techniques performance and feed that information back to the optimisation tool during the optimisation process. The PCSWMM model was chosen because it is a widely used, dynamic rainfall-runoff simulation model that is well suited to simulating runoff quantity and quality in urban environments (Zhang and Guo 2014; Baek et al., 2015; Park et al., 2015).

More importantly, PCSWMM can simulate the hydrologic performance of seven different types of WSUD techniques (bioretention, rain garden, green roof, infiltration trench, permeable pavement, rainwater tank, and vegetative swale), allowing the optimisation approach to link the techniques performance to specific WSUD designs and locations. The optimisation tool employs the NSGA-II algorithm (Deb et al., 2002) to assess the benefits (runoff and pollutant load reductions) and costs of various WSUD deployment scenarios (techniques, location, and number) to determine the best cost-effective choices that meet user-defined stormwater management targets.

### 8.2.1. Optimisation Algorithm

The tool adopts NSGA-II approach for optimization. NSGA-II is a multi-objective optimisation method that belongs to the evolutionary optimisation technique family and is capable of creating optimal or near optimal trade-off solutions among competing objectives (Deb et al., 2002).

The nondomination sorting strategy used by NSGA-II makes it faster than any other multi-objective algorithm. In NSGA-II, solutions are ranked according to their degree of dominance within the population, with the highest ranking going to a solution that is not dominated by any other solution. Furthermore, the technique uses a crowded-comparison operator to conserve diversity along the Pareto-optimal front, allowing the programme to find the full Pareto-optimal region. This algorithm was mathematically described by Deb et al., (2002).

In recent years, NSGA-II has gained attention for outperforming other multi-objective evolutionary algorithms in handling complicated environmental optimisation issues. USEPA 2009; Maringanti et al., 2009, 2011; Rodriguez et al., 2011; Zare et al., 2012; Ahmadi et al., 2013) used NSGA-II to assist the selection and placement of optimal management techniques to decrease water quality degradation.

#### 8.2.1.1. Optimisation Process

The algorithm starts with a population of potential solutions that is generated at random. The population is defined as the concept of Pareto dominance (non-domination). When a solution performs no worse than the other solution in all objectives while outperforming the other solution in at least one, it is said to be non-dominant. Each solution is given a fitness (or rank) equal to its non-dominant level at the end of the sorting, with a lower rank indicating that the solution is dominated by fewer other solutions. In addition, a crowding distance is calculated for each individual solution as a measure of solution variety, defined as the size of the greatest cuboid surrounding a solution without including any other solution in the population. The population is more diverse when the average crowding distance is large. The parent populations are chosen from the population in the first step of the optimisation process using binary tournament selection based on rank and crowding distance. If the rank is lower than the other solutions or the crowding distance is greater than the other answers, an individual solution is chosen. Through the processes of crossover and mutation, the selected population produces an offspring population of the same size. Only the best N individuals are chosen to form a new parent population, where N is the population size, from the combined population of the present parent and offspring, which is sorted again according to non-domination. The use of both the parent and child populations in the sorting ensures elitism in this step. At each iteration, the current population is compared to previously found non-dominated solutions. The new parent population is then utilised to generate a new child population, and the cycle repeats until the stopping criteria are met. The optimisation is regarded to have converged at an optimal front at this stage, and the procedure can be terminated.

#### 8.2.1.2. Stop Criteria

Users can utilise user-defined stop criteria to stop the NSGA-II. Maximum number of iterations, no change in the new parent population for two consecutive loops, and no discernible increase in the fitness function after a given number of iterations are some of the most widely employed criteria. In this research the maximum number of iterations was used as a stop criterion. The number of possible variables, model simulation time, and model complexity (number of sub-catchments and drainage network) all affect how many iterative runs the optimization process needs to complete before reaching the best solutions. After multiple tests runs with various numbers of iterations, 262 iterations were found to be sufficient for the current setup.

### 8.2.2. Structure of GreenPlan-IT

The optimisation tool is structured around three functionalities: optimizer, WSUD performance evaluator, and cost calculator, each supported by a set of subroutines. The optimizer is a search engine that generates WSUD situations and uses the NSGA-II algorithm to drive the search process. The WSUD performance evaluator creates input files for the modelling tool to incorporate the generated scenarios, runs the model with updated input files, and returns the WSUD performance statistics generated by the model to the optimizer. The cost calculator is used to calculate the cost of each WSUD scenario and provide that data into the optimizer.

The three function groups collaborate in an iterative and evolutionary process to find the best WSUD solutions that fit the user's unique requirements and goals. During the search process, the optimizer generates new WSUD scenarios inside the search space defined by the universe of suitable locations discovered by the GIS Site Locator tool, the WSUD performance of each scenario is evaluated, and costs are estimated for each. The optimizer then evaluates the cost and performance statistics and adjusts the search path to yield a new set of possible WSUD alternatives, repeating the process until the iteration's stated requirements are met. The tool's conceptual overview is shown in Figure 8-1.

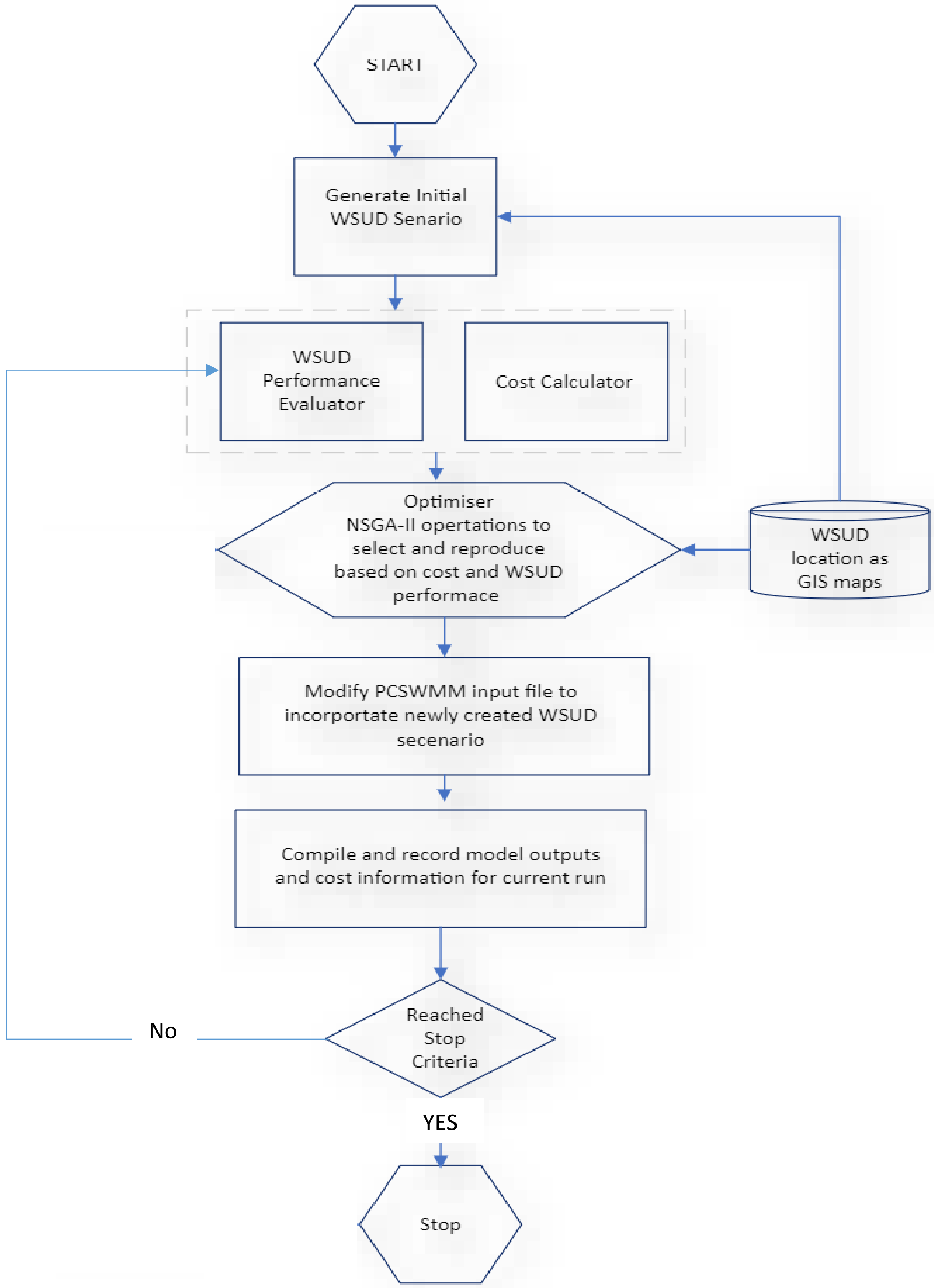



*Figure 8-1- GreenPlan-IT Tool Overview*

### 8.2.3. Optimisation Formulation

#### 8.2.3.1. Optimisation Objectives

The optimisation problem's aims were to (1) reduce the total relative cost of WSUD techniques implementation and (2) decrease the load of stormwater volume and quality pollutant (i.e., TSS load) at the catchment's outlet. The number of fixed-size units of the distributed WSUD techniques were used as decision factor. The choice variable for each suitable WSUD techniques ranged from zero to the maximum number of probable locations found using PCSWMM model GIS layer. The optimisation problem can be stated mathematically as Equation 8-1:

$$\text{Minimize } \sum_{i=1}^{n} \text{cost(WSUDi)} \qquad (8\text{-}1)$$

Maximize stormwater quantity and quality pollutant load reduction.

Subject to $n \leq N_{max}$

where WSUDi is the set of WSUD configuration decision variable associated with location I and Nmax is the maximum number of possible sites. The cost obtained from the above equation was used for the results comparison for each WSUD combination.

The NSGA-II operational parameters, such as population size, generation number, and crossover and mutation rates, determine the search algorithm and have a significant impact on the optimisation results. The parameters were chosen considering the values used by Deb et al., (2002), as well as the difficulty of the optimisation issue and the model run time. To find the best parameter values, several combinations of different population sizes and generations were considered including 20,80, and 100. In the end, the essential NSGA-II parameters were set to 200 as generations, 100 for population size, 0.9 for crossover probability, and 0.1 mutation probability, which was mostly consistent with Deb et al (2002) recommendations and adopted by Minh 2020 and Wu et al. (2019).

#### 8.2.3.2. WSUD Types and Design Specification

The Optimization Tool can incorporate four types of WSUD techniques: bioretention, permeable pavement, tree well (proprietary material), and tree pits, but it can also include up to six user-defined techniques. Chosen WSUD techniques and their specifications for sub-catchment scale optimisation task has been explained in chapter 7, section 7.10. For optimisation task and in sub-catchment scale in this research only bioretention, permeable pavement and infiltration systems were used. This was due to the systems performance at allotment scale to reduce the stormwater peak significantly. The technique's specification in this research was adopted based on a review of the literature and the common size of the techniques in existing urban development conditions (Section 7.10). The techniques specification had remained unchanged during the optimisation process.

#### 8.2.3.3. Decision Variables

The decision variables in the optimisation process were the number of units of each of the WSUD types in each of the sub-catchments within the study region, because the techniques specification had remained constant. The decision variable values for each applicable WSUD type range from zero to a maximum number of potential sites determined by using GIS map in the study area and available spaces divided by the area of the adopted techniques.

#### 8.2.3.4. WSUD Locations

The GIS interface is included in the PCSWMM model. Potential locations for WSUD technique adoption were identified using GIS layers and exploration in the research study region, taking into account open green spaces and connections to the stormwater network. The number of selected WSUD techniques was calculated by dividing the available spaces for techniques adopted by the system's surface area. To assess the performance of the chosen techniques at different scales on the catchment storm water runoff volume reduction, catchment C1 was divided into 650 sub-catchments while catchment C2 was divided into 46 sub-catchments. Figure 8-2.

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*Figure 8-2- Catchment 1 and 2 Sub-catchment Boundaries and Green Spaces*

#### 8.2.3.5. Stop Criteria Used

The tool's stop criteria were set to the maximum number of iterations (i.e., population size which is 100). Using specific user-defined stop criteria, users can terminate the NSGA-II. A maximum number of iterations, no change in the new parent population for two consecutive loops, and no noticeable rise in the fitness function after a predetermined number of iterations are among the often-employed criteria. The number of possible variables, model simulation time, and model complexity (number of sub-catchments and drainage network) all affect how many iterative runs the optimization process needs to complete before reaching the best solutions. After multiple tests runs with various numbers of iterations, 262 iterations were found to be sufficient for the current setup.

## 8.3. Stormwater Quantity Scenario Development at Sub-catchment Scale

The various scenarios developed for the optimization-simulation model are described in this section. The developed scenarios utilized enabled comparison of the chosen WSUD techniques performance under various conditions. By analysing the scenarios, a better understanding was achieved of how different implementation approach affected WSUD performance. Four scenarios were created for optimization and evaluation. For current and future conditions in the research area, two potential WSUD implementation scenarios were investigated during four storm events. Two storm events were selected from the study area's collected data and rainfall information and adopted for future conditions, presuming the study area will get dense as the impermeable area and rainfall will be increased in the future. Table 8-1 explains the scenarios that have been developed.

For choosing the single storm event, the maximum daily rainfall in the research area from the nearest rainfall station (section 5-10) during the year when stormwater collection data was obtained was taken into consideration.

Table 8-1- Developed Scenarios for Optimization Process _ Quantity

| Scenario Number | Implementation approach | Catchment | Current Storm Events | Future Storm Events | Current Fraction Impervious (%) | Future Fraction Impervious (%) |
|---|---|---|---|---|---|---|
| S1-1 | Allotment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C1 | continuous one-year (594.6 mm rainfall)** | continuous one-year (7.3% increase in rainfall intensity)* | 57 | 77 |
| S1-2 | Sub-catchment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C2 | continuous one-year (594.6 mm rainfall)** | continuous one-year (7.3% increase in rainfall intensity)* | 64 | 84 |
| S2-1 | Allotment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C1 | Single storm event (24.6 mm rainfall)*** | One event (7.3% increase in rainfall intensity)* | 57 | 77 |
| S2-2 | Sub-catchment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C2 | Single storm event (24.6 mm rainfall)*** | One event (7.3% increase in rainfall intensity)* | 64 | 84 |

*Table 2-1 (DELWP Guideline)

** Long term average annual rainfall for the site as explained in section 5-10. Rainfall data was collected for the study area during the time that stormwater data collection was underway in order to produce accurate optimization results and more precise model calibration and validation.

*** One-day storm event: the event was chosen from the collected rainfall data for the study area (section 5.10) to assess the chosen assets performance under a single heavy storm event as can happen in the area.

### 8.3.1. WSUD Costs

Implementing WSUD at the landscape scale incurs many costs ranging from permitting, traffic control, and construction to maintenance and the techniques operation. Costs related to construction, design, and engineering, as well as maintenance and operation over a 30-year life cycle, were taken into account for this study. The limited data available for the techniques suggest a wide range in costs depending on site-specific parameters, design configurations, and other factors and constraints, like socioeconomics in different local areas.

The total relative cost related to each WSUD scenario was calculated using a unit cost technique. Based on the overall cost and designed surface area of each feature, cost per unit surface area was determined for each WSUD type. Any WSUD scenario's overall relative cost was determined as

Total cost =

$$\sum (\text{number of WSUD technique} \times \text{unit cost} \times \text{surface area of each WSUD technique}) \qquad (8\text{-}2)$$

#### 8.3.1.1. WSUD Cost for Sub-catchment Techniques

In section 6.2.8 of this thesis, the life cycle cost of WSUD techniques has been explained in detail. This section explains the life cycle cost of sub-catchment scale techniques as was obtained from the literature review, Melbourne Water life cycle cost information sheet (Appendix C) and the local councils experience in Melbourne based on the techniques size.

There are different costs associated with implementing WSUD techniques from design, construction to maintenance and operation. Costs associated with design, construction, maintenance, and operation during a 30-year life cycle were taken into account for this study. Table 8- 2 shows the obtained and adopted costs for the chosen techniques.

Table 8-2- Sub-catchment Techniques Life Cycle Cost

| Technique | Construction ($/m$^2$) | Design (%) | Operation and maintenance ($/m$^2$/y) | Total cost ($/m$^2$) |
|---|---|---|---|---|
| Raingardens | 1,000 | 20 | 5 | 1,100-1,250 |
| Permeable Pavement | 200 | 60 | 5-10 | 250-350 |
| Infiltration Trench | 250 | 20 | 5-10 | 900-1,000 |

### 8.3.2. Constraints on WSUD locations

The maximum number of feasible sites, which were determined by GIS exploration based on available open green spaces and connections to the stormwater network, were used to limit the number of potential locations for each WSUD technique. The number of selected WSUD techniques was calculated by dividing the available spaces for techniques adopted by the system's surface area. The total area that was treated by WSUD within every sub-catchment set implicit restrictions on the number of WSUD implementation that were feasible within the sub-catchment depending on the sizing criteria. During the optimisation process, the number of WSUD units were decreased when the combined treatment areas of the WSUD technique units exceeded the area that could be used for treatment within each sub-catchment.

After reviewing the WSUD design guidelines, manuals and consulting with local stormwater experts, a sizing factor (defined as the ratio between technique surface area and its drainage area) was specified for each WSUD technique: 7 and 8% for bioretention and infiltration trench, and 50% for permeable pavement.

### 8.3.3. WSUD Sizing Criteria Sensitivity

Another important premise that affected how WSUD practices performed in the model was the percentage of catchment land that was drained to WSUD techniques. For the purposes of this study, and as was suggested by the tool guideline, permeable pavements, bioretention and infiltration trenches were each given a size factor of 50% 4% of the drainage area, respectively. The techniques for choosing and sizing WSUD systems to satisfy permit requirements, however, vary in complexity and can provide a wide range of designs under site-specific circumstances. Given this fact and by reviewing local councils design guidelines size factors for bioretention and infiltration trench were raised to 7 and 8 percent, respectively, while permeable pavement had remained at 50 percent, in order to test the sensitivity of the WSUD techniques sizing criteria.

### 8.4. Stormwater Quality Optimization Scenario Development at Sub-catchment Scale

Using the Green-Plan IT tool, WSUD techniques (i.e., permeable pavement, bioretention, and infiltration trench) were analysed in this study to determine the best alternative techniques within the study region to maximize the reduction of TSS load while minimizing the total cost of techniques implementation.

As covered in chapter five conclusion stormwater data collected for the research indicates that the research area has a significant generation of total suspended solids (TSS) load when compared to comparable urban catchments documented in the literature. The scenarios chosen for the research area’s TSS elimination were compiled in Table 8-3. For choosing the single storm event, the maximum daily rainfall in the research area from the nearest rainfall station (section 5-10) during the year when stormwater collection data was obtained was taken into consideration.

Table 8-3- Developed Scenarios for Optimization Process _ Quality

| Scenario Number | Implementation approach | Catchment | Current Storm Events | Future Storm Events | Current Fraction Impervious (%) | Future Fraction Impervious (%) |
|---|---|---|---|---|---|---|
| S4-1 | Allotment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C2 | One event (24.6mm rainfall) | One event (7.3% increase in rainfall intensity) | 64 | 84 |
| S4-2 | Sub-catchment scale WSUD techniques (Bioretention, Permeable Pavement and Infiltration Trench) | C2 | continuous one-year (594.6mm) | continuous one-year (7.3% increase in rainfall intensity) | 64 | 84 |

*Table 2-1 (DELWP Guideline)

** Long term average annual rainfall for the site as explained in section 5-10. Rainfall data was collected for the study area during the time that stormwater data collection was underway in order to produce accurate optimization results and more precise model calibration and validation.

*** One-day storm event: the event was chosen from the collected rainfall data for the study area (section 5.10) to assess the chosen assets performance under a single heavy storm event as can happen in the area.

## 8.5. Stormwater Quantity and Quality Management Scenarios Results and Discussion

### 8.5.1. Allotment versus sub-catchment Scale Stormwater Volume Reduction Cost Effective Curves - Continuous Events

The best trade-off between WSUD implementation costs and stormwater volume runoff reduction at the study area outlets is shown in Figures 8-3 and 8-4. The results relate to Sub-catchment C1 and C2 and scenarios S1-1 and S1-2, respectively, for the continuous simulation. Each point along the cost–effectiveness curve represents a solution of the number of bioretention, permeable pavement an infiltration trench in the study area. The optimal solutions are found on the left and upward edges of the search space when all solutions were simultaneously plotted. Number of systems in combination were changing to achieve the best curve.

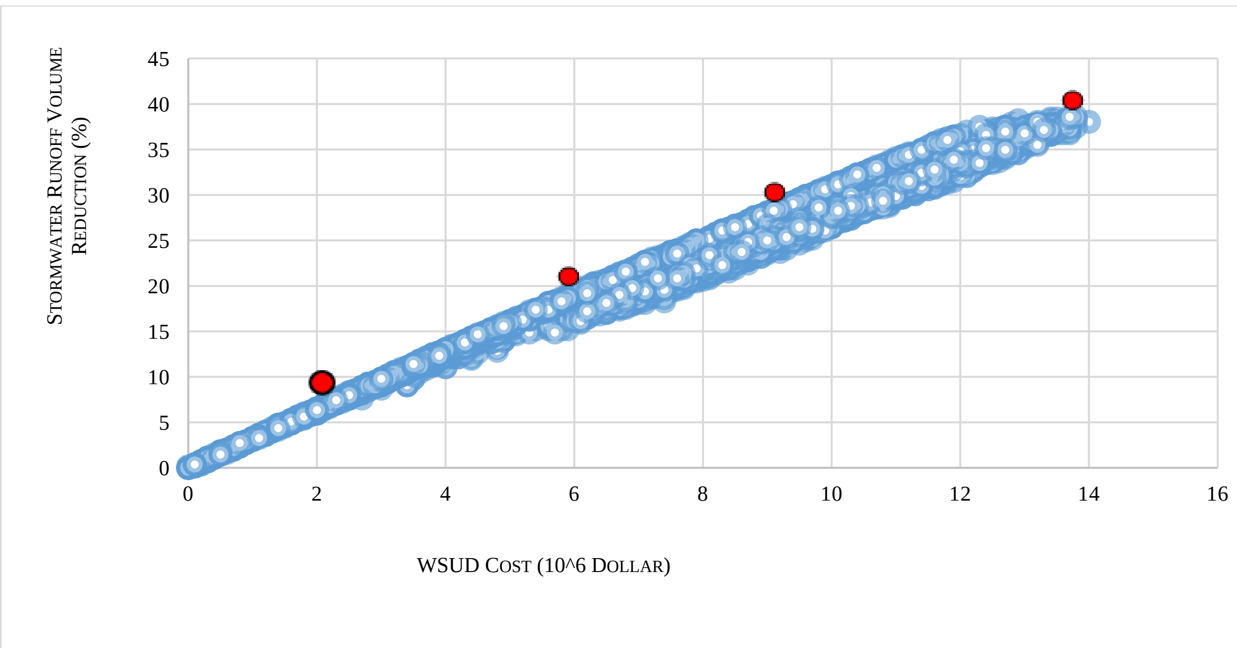


*Figure 8-3- Cost effective curves for Scenario S1-1*

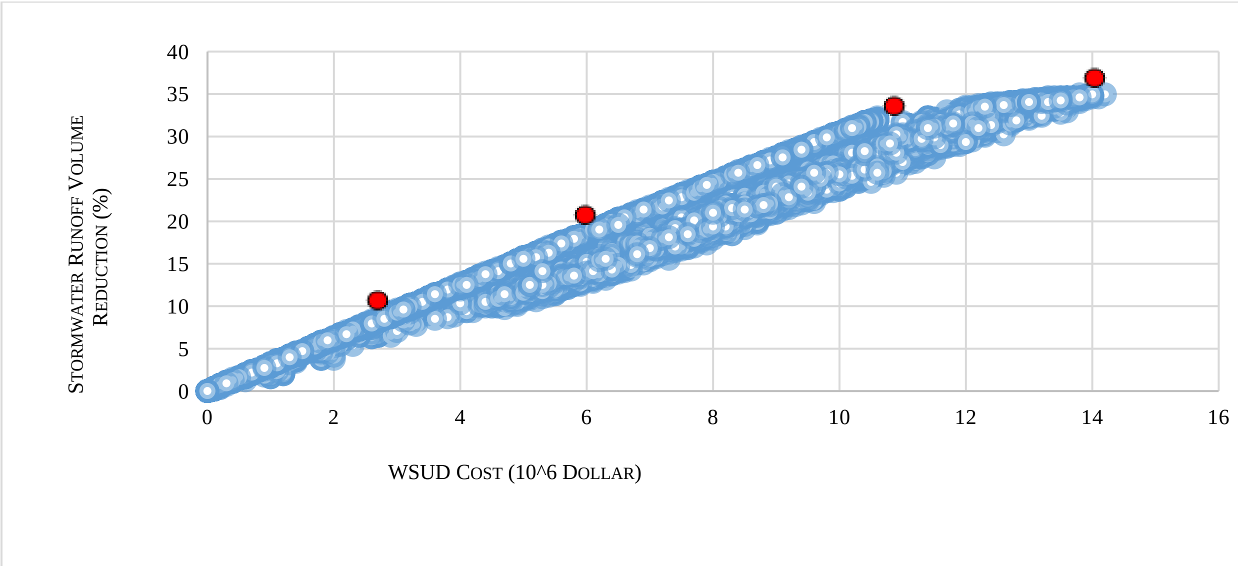


*Figure 8-4- Cost effective curves for Scenario S1-2*

The stormwater volume reduction efficiency of the optimal solutions was in the range of 0-39% and 0-35% for scenario S1-1 and S1-2 respectively. The greatest reduction efficiency was 35% to 39%, which translates to a $13.8 to $14.3 million dollars implementation costs. This indicates that the optimisation approach could help decision-makers find the most economical strategy that produces the maximum reduction possible within a specified budget. While implementing WSUD techniques at allotment and sub-catchment scales resulted in a maximum 39% stormwater quantity reduction for the current condition, the reduction was only achieved at around 30% for the future catchments condition with higher rainfall intensity and impervious fraction, as shown in Figure 8-5 and 8-6.

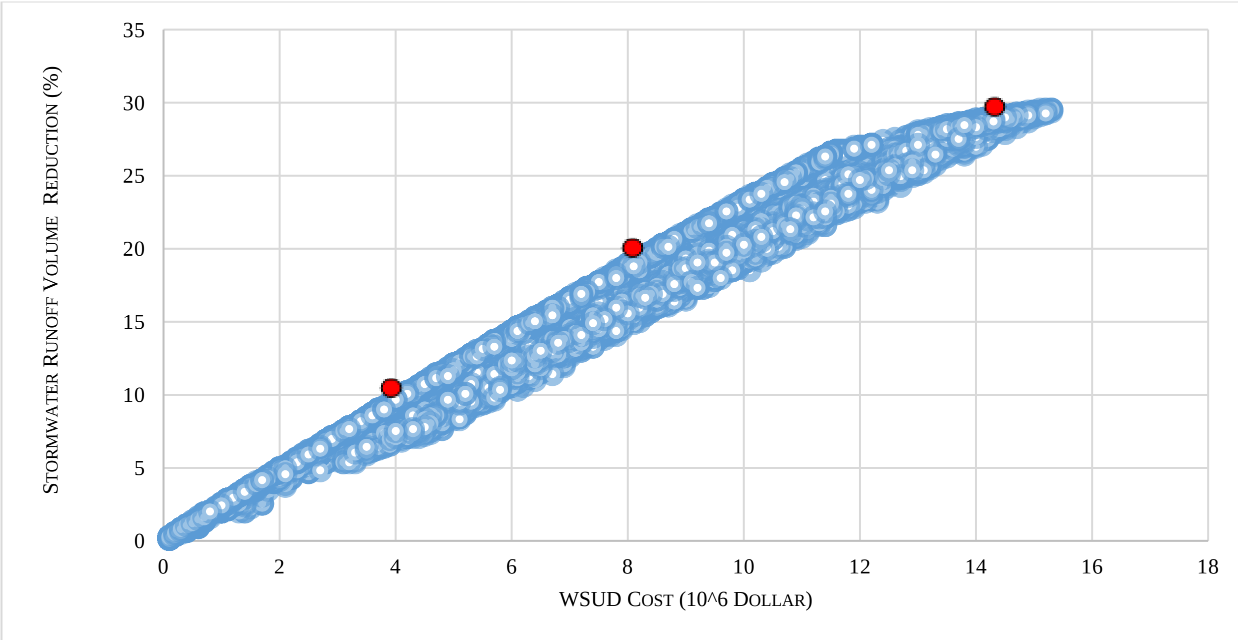


*Figure 8-5- Cost effective curves for Scenario S1-1 and for future scenarios.*

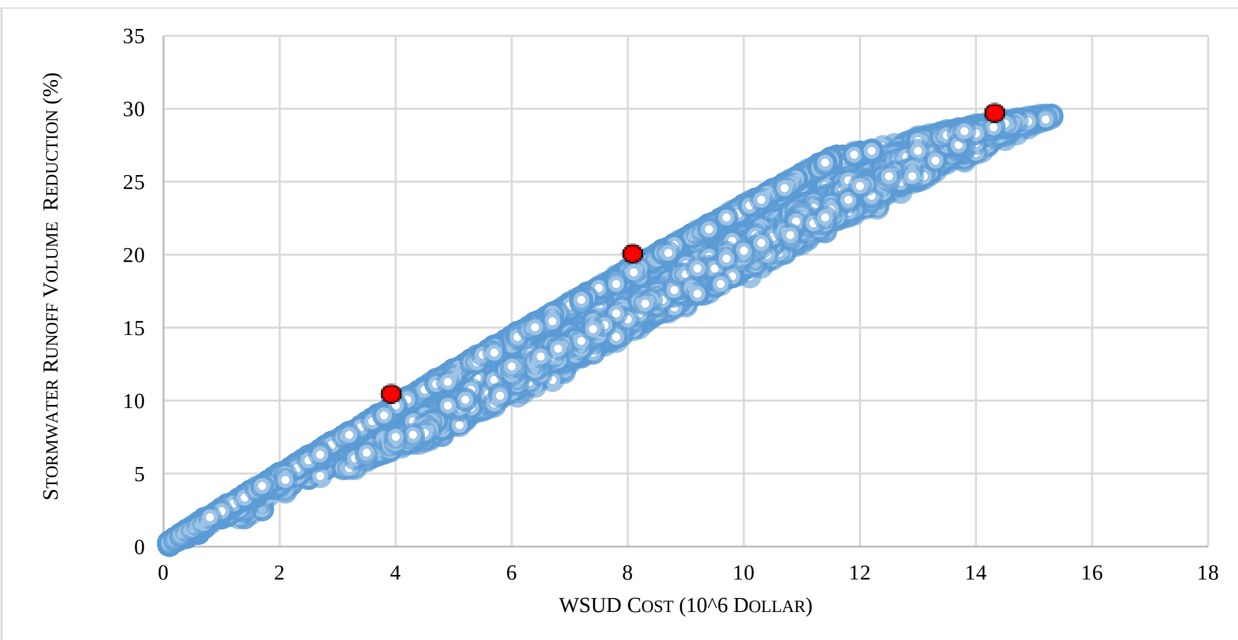


*Figure 8-6- Cost effective curves for Scenario S1-2 and for future scenarios*

### 8.5.2. Allotment versus sub-catchment Scale Stormwater Volume Reduction Cost Effective Curves - Single Storm Events

Figure 8-7 and 8-8 shows the optimal trade-off between WSUD implementation cost and stormwater quantity runoff reduction at the research study area outlets. The results are for a single storm event for Sub-catchment C1 and C2 and scenarios S2-1 and S2-2 respectively. Each point along the cost–effectiveness curve represents a solution of the number of bioretention, permeable pavement an infiltration trench in the study area. The optimal solutions are found on the left and upward edges of the search space when all solutions were simultaneously plotted. Number of systems in combination were changing to achieve the best curve.

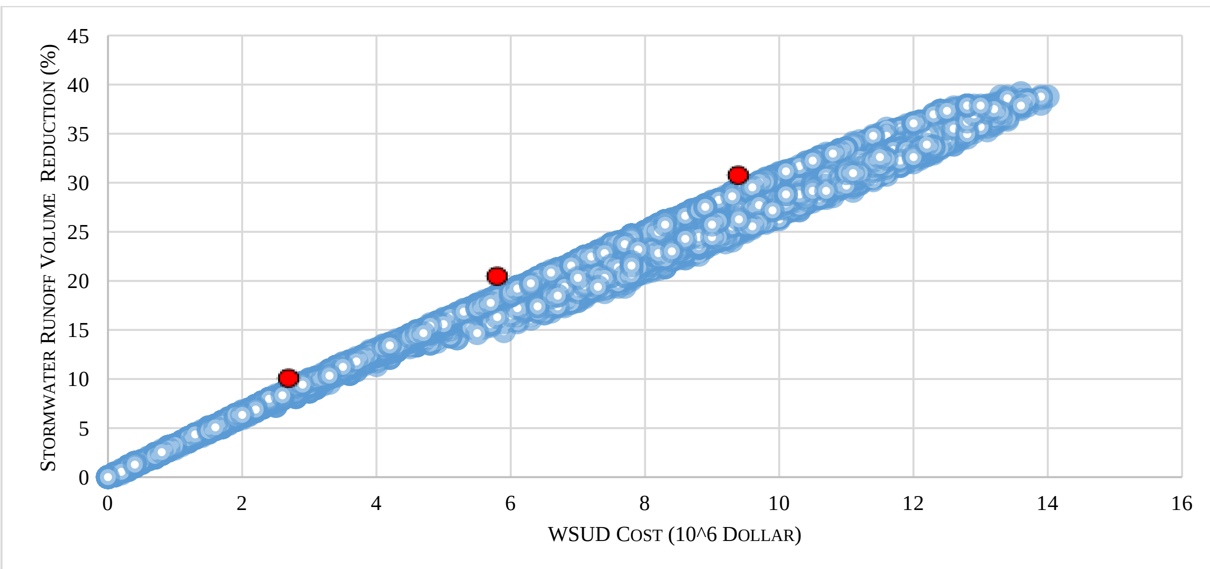


*Figure 8-7- Cost effective curves for Scenario S2-1 for the chosen event*

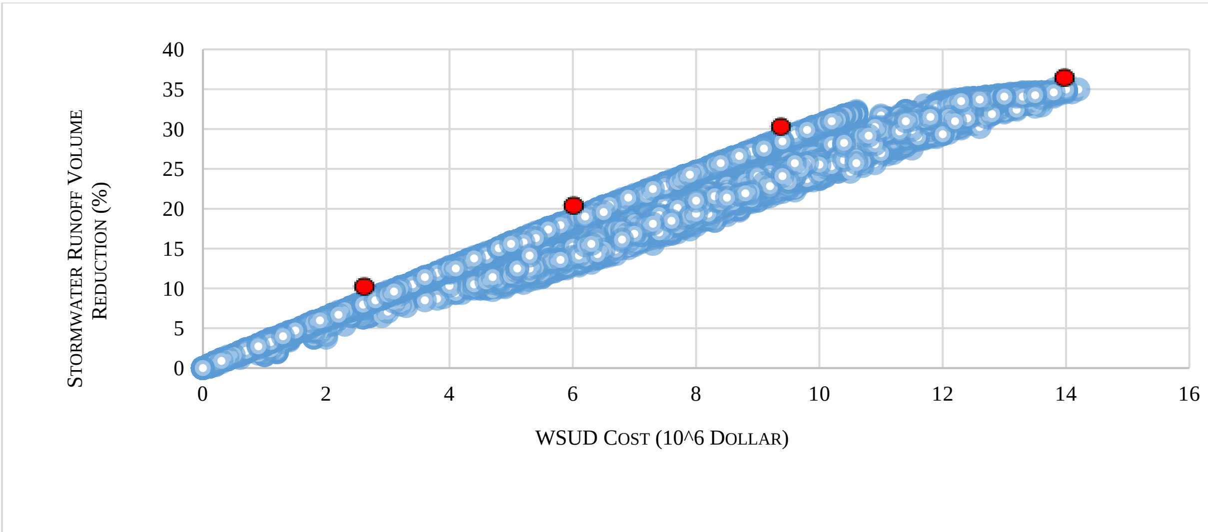


*Figure 8-8- Cost effective curves for Scenario S2-2 for the chosen event*

The stormwater reduction efficiency of the optimal solutions was in the range of 0–40% and 0-35% for scenario S1-1 and S1-2 respectively. The greatest reduction efficiency was 35% to 40%, which translates to a $14 to $14.3 million dollars implementation costs. This indicates that the optimisation approach could help decision-makers find the most economical strategy that produces the maximum reduction possible within a specified budget. While implementing WSUD techniques at allotment and sub-catchment scales resulted in a maximum 40% stormwater quantity reduction for the current condition under a single storm event, the reduction was only achieved at around 35% for implemented systems only in sub-catchments. For the future catchments condition with a 7.3% rise in rainfall intensity and a greater impervious ratio, both scenarios demonstrated efficiency of 33%.

### 8.5.3. Stormwater Quality Management Scenarios Results - Single and Continuous Events

The best solutions for reducing TSS in catchment C2 were graphed along a cost-effectiveness curve by the optimisation process. The curve links various combinations of chosen WSUD techniques used throughout the catchment and their related costs to TSS removal efficiency. The optimum balance trade-off between implementation costs and TSS load reduction is shown in Figure 8-9 and 8-10 for continuous rainfall and a single storm event as explained in Table 8-3.

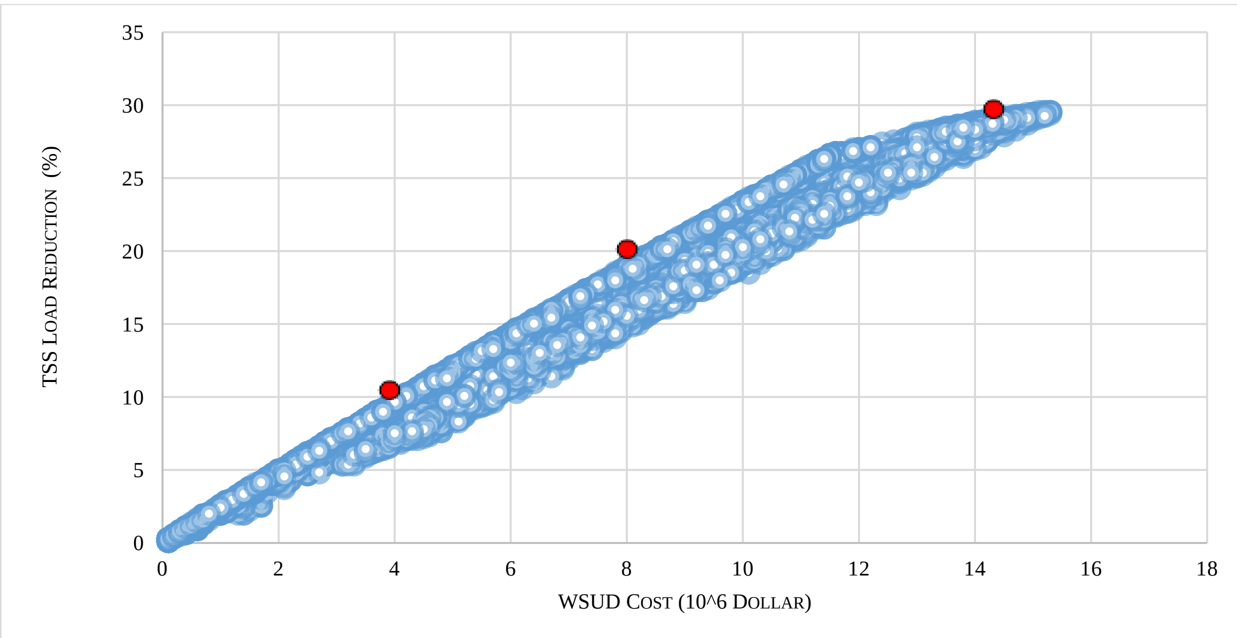


*Figure 8-9- Cost effective curves for Scenario S4-1*

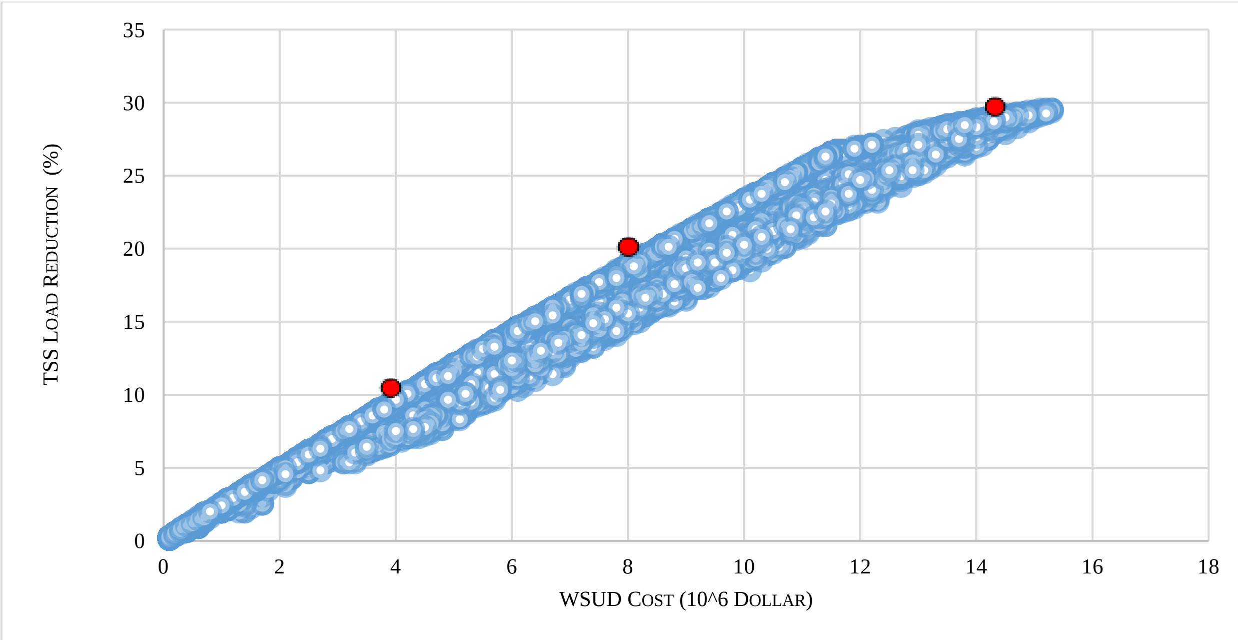


*Figure 8-10- Cost effective curves for Scenario S4-2*

The optimum solutions shown as the upper and leftmost borders of the optimisation domain are shown alongside each individual solution. The number of bioretention units, infiltration trenches, and permeable pavements present at each point on the cost-effectiveness curve indicate a combination, and each point can be investigated in terms of the magnitude of build-out within each sub-catchment.

An enormous variety of WSUD options for TSS load reductions are shown in Figure 7-8. The amount of pollution removed at the same level of cost could vary by up to 30%, while the difference in total relative cost between an optimal and a nonoptimal solution could be well below $20 million dollars. This demonstrates how applying an optimisation technique can assist stormwater managers in finding the most affordable option for attaining flow and water quality improvement aims while working with a limited budget. The optimal frontier's slope in Figure 7-8 depicts the marginal new WSUD installations, and the frontier's declining slope denotes decreased marginal returns from adding more WSUD installations, which are mirrored in rising costs. For instance, according to the curves, a 10% TSS removal can be accomplished with roughly $4 million dollars, but only 20% more TSS removal can be anticipated for the following $14 million dollars expenditure. The TSS reduction efficiency of the optimal solutions was in the range of 0–30% and 0-33% for scenario S4-1 and S4-2 respectively. For the future catchments condition with a 7.3% rise in rainfall intensity and a greater impervious ratio, both scenarios showed the same range of TSS reduction.

In conclusion, the results of this study aligned with expectations, as the tool parameters were carefully collected, calibrated and adopted based on a comprehensive understanding of the study area under existing conditions. The trade-off curves followed anticipated trends, confirming the reliability of the optimization process. The findings contribute to a broader understanding of stormwater management challenges both in Australia and internationally by refining methodologies for optimizing WSUD techniques.

By improving the representation of WSUD measures within PCSWMM and integrating a cost-effectiveness analysis tool (i.e., GreenPlan-IT), this study enhances the reliability of urban stormwater modelling. The proposed methodology can be replicated in similar research, providing a robust framework for assessing stormwater management strategies in developed urban settings. These insights have global relevance, particularly for rapidly urbanizing regions where sustainable drainage solutions are crucial. Additionally, the optimization framework developed in this study is adaptable to diverse climatic and hydrologic conditions, equipping policymakers and engineers with a valuable tool for designing resilient and sustainable urban water management systems.

#### 8.5.3.1. WSUD techniques effectiveness to Remove Total Suspended Solids (TSS)

Urban surface runoff has been severely impacted in recent decades by an increase in impervious area brought on by the process of urbanisation. Peak flow is enhanced, and concentration time is decreased as a result of the increasing impervious surface. Pollutants as sediments, nutrients, heavy metals, trace elements, and pathogens are deposited on the catchments during the dry weather period. Storms cause surface runoff to sweep away a sizable portion of the pollutants on impervious surfaces, degrading the receiving water body at both the local and regional levels (Minh 2020). The majority of the pollution load from cities is transported from occasions with a return duration of under two years (Guo and Urbonas 1996). Total Suspended Solids (TSS), one of the storm flow pollutants, are a significant contributor to the worsening of water quality. This causes aesthetic problems, greater water treatment costs, a loss in fishing resources, and severe ecological degradation of aquatic habitats (Bilotta and Brazier 2008). The efficiency of WSUD implementation for reducing the TSS of the storm flow has been the subject of numerous research utilising the experimental method. Hsieh and Davis presented two media profiles for bioretention design to achieve TSS removal effectiveness of more than 96% through conducting an experiment with various media scenarios (2006). Hunt et al.'s (2008) field study revealed that bioretention could only reduce TSS by 60%, which is less effective than the findings of earlier experimental studies (2008). In two separate climates, Hatt et al., (2009) investigation into the effectiveness of three field-scale biofiltration devices for the removal of TSS found that TSS was successfully removed with load reductions typically exceedingly more than 90%. According to Braswell et al., (2018), a series made up of permeable interlocking concrete pavement and a specialised box filter had a 96% TSS removal efficiency.

According to Abdollahian and Kazemi (2018), permeable pavement has a TSS elimination efficiency of between 47 and 69%. All types of extensive green roofs had a range in the event mean concentration reduction rate of suspended solids (SS) of 64.3% to 73.1% (Gong et al., 2019). Planted green roofs reduced TSS by up to 83% throughout the course of the 16-month trial, which included nine storm occurrences MacAvoy et al., (2016).

The findings of this study demonstrate a variation in TSS load reduction at the catchment outlet from 30% to 33% for a continuous storm event and a single event under current catchment conditions and future climate and land use change. The percentage was obtained from a combination of raingarden, infiltration trench and permeable pavement techniques. While the obtained percentage does not reach the established guideline standards for pollution removal from an urban catchment (as defined in section 2.8 of this document), it should be noted that this figure is for an existing developed catchment that will get denser in the future as rainfall intensity increases.

## 8.6. Performance of WSUD techniques for Stormwater Quality and Quantity Reduction

The WSUD techniques associated to each point on the cost effectiveness curve provide insight on the reason and priority of chosen techniques. From the cost-effectiveness curves, three solutions with reductions of 10%, 20%, and 30% (Figure 8-3 to 8-10, three red dots) were chosen for a more detailed analysis. The cost-effectiveness curve has points at each of which a different WSUD combination is shown.

For the chosen solution, the WSUD selection can be (1) assessed in terms of stormwater quantity and quality reduction size and % implementation, and (2) spatially analysed in terms of

WSUD options across each sub-catchment. For each chosen solution, the percentage of each of the three WSUD techniques is quantified (Figure 8-11). Bioretention identified as the most efficient technique at all stormwater quantity reduction percentages, making up 80% of all WSUD chosen, followed by infiltration trench (15%), and permeable pavement (3%). Infiltration trench found more cost effective while its reduction percentage was raised from 15% at the 10% reduction scenario to 22% at the 30% reduction. Because of its low treatment ratio and the limited number of suitable sites connected to its huge surface area, permeable pavement was the least cost-effective option for all three reduction scenarios, with utilisation remaining below 10%. The selection of each WSUD technique for the sub-catchment study in this research was largely driven by the technique's respective representation within the optimization process explained in chapter 6 and unit cost. Changing any of these will change how each WSUD system will be utilized for any given solutions.

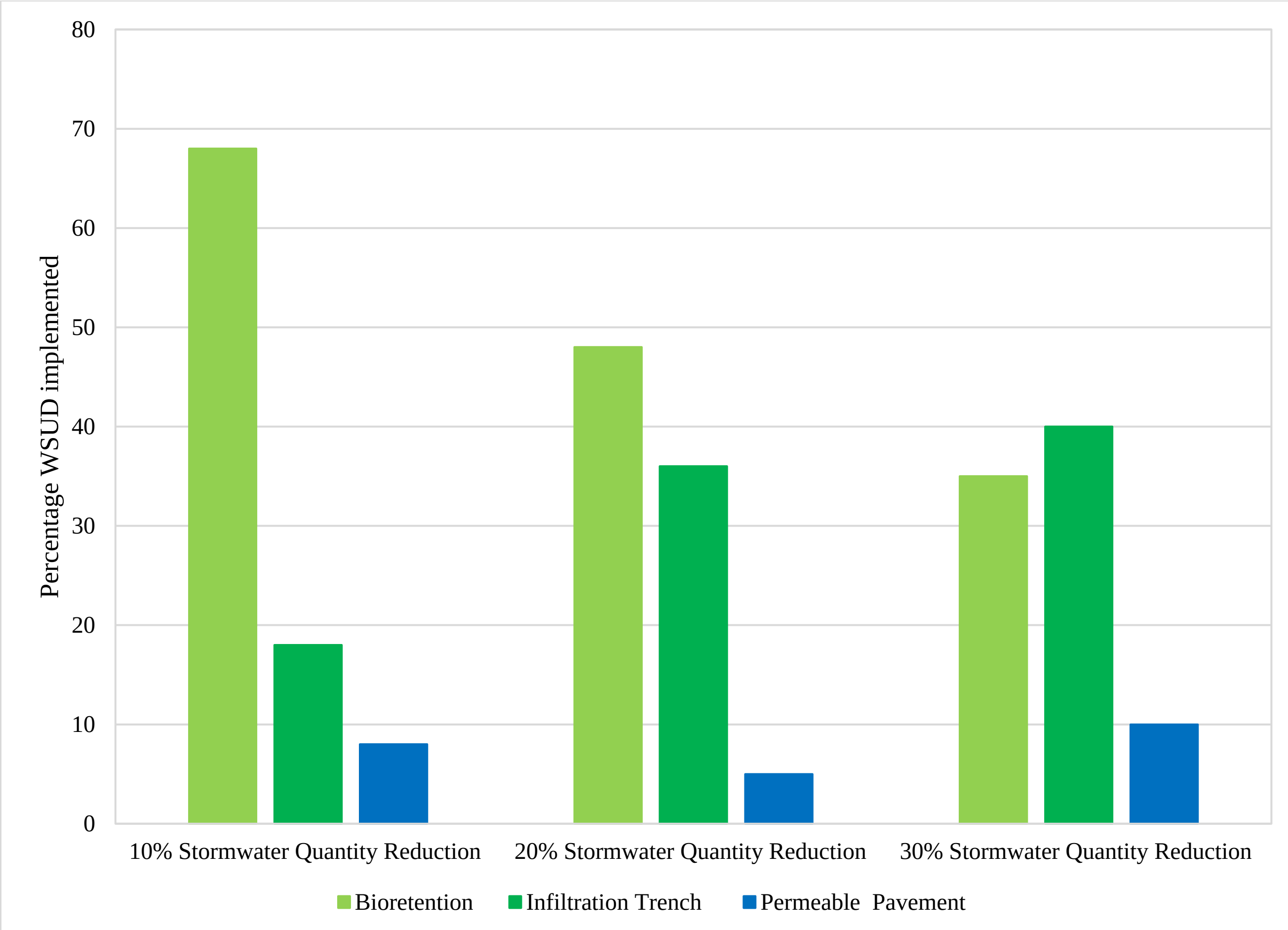


*Figure 8-11- Percentage of each WSUD techniques adopted for the three best solutions.*

The chosen WSUD treatment performance against the implemented area was also assessed (Figure 8-12). Looking at a 30% reduction nearly 62% of the sub-catchments were treated with bioretention, 18% with infiltration trenches, and 8% with permeable pavement. While reduction percentage was increased, the percent of area treated by infiltration trench increased and bioretention percentage decreased.

The most cost-effective WSUD among the three chosen techniques was bioretention, which was treating more than 50% of the areas that are available in all three solutions. The least cost-effective WSUD technique, in contrast, appears to be permeable pavement because it only treats up to 10% of areas.

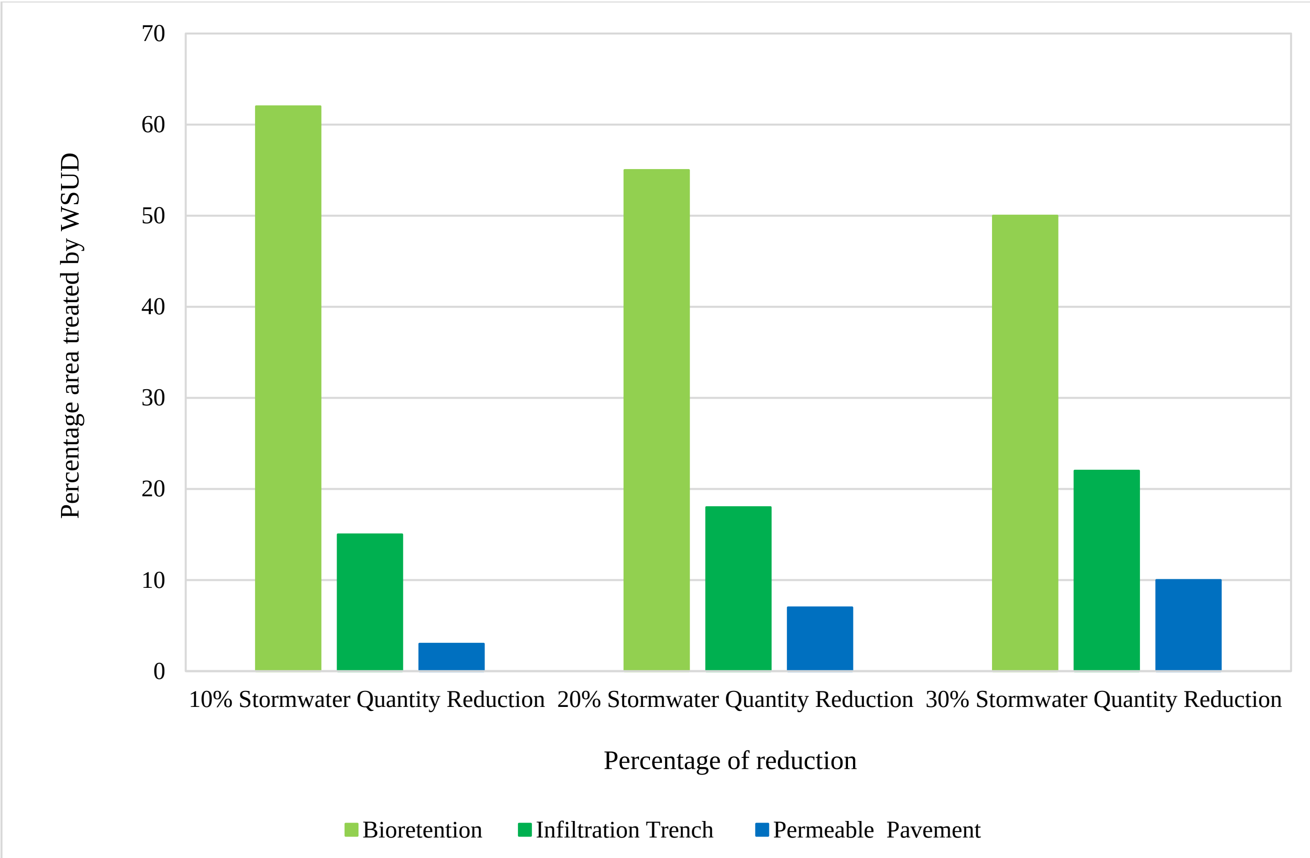


*Figure 8-12- Percentage of area treated By WSUD techniques treated for the three best solutions.*

Future scenarios and the outcomes of the stormwater quality optimisation scenarios revealed almost the same performance for the chosen techniques.

The results of WSUD utilisation were mapped by sub-catchments to learn more about the best location for chosen techniques given the defined objectives and constraints. The number of bio-retentions, infiltration trenches, and permeable pavement techniques found in each sub-catchment for the 40% stormwater runoff volume reduction scenario is shown in Figure 8-13 to 8-15.

The optimisation procedure found that the sub-catchments with high reduction yield (darker areas), where the WSUD techniques may be most effective and required more techniques. The overall number of WSUD sites determined depends on the unit size utilised for each chosen technique. In a different design for any WSUD was employed, the optimal solutions were differed in WSUD numbers and compositions. The figures show the maximum runoff volume reduction adopting combinations of chosen techniques. The assumptions, constraints, and optimisation objectives particular to this case study must be taken into consideration while interpreting the optimisation outcomes. If one or more assumptions varied, the optimisation may have produced entirely different sets of alternatives in terms of WSUD selection, distribution, and cost. If the optimisation aims, for instance, had been to reduce overall runoff volume rather than pollutant load, the results may have been entirely different in terms of WSUD selection, distribution, and cost.

The total relative cost associated with different reduction targets calculated from the unit cost does not necessarily represent the true cost of an optimum solution for the sub-catchments evaluated and is not transferrable to other sub-catchments due to the large variation and uncertainty associated with unit WSUD cost information. Instead, these costs have to be seen as a standard framework for assessing and contrasting the relative effectiveness of various WSUD situations. Implementation costs will likely be significantly lower than the modelling would predict as WSUD techniques can be implemented as a part of current or improved capital implementation strategies and transportation projects, through batch design and construction where large areas of the urban landscape are retrofitted at once, as a component of new development, and possibly through public-private partnerships. Figures 8-16 to 8-18 show the WSUD combinations for maximum pollution load reduction in C2.

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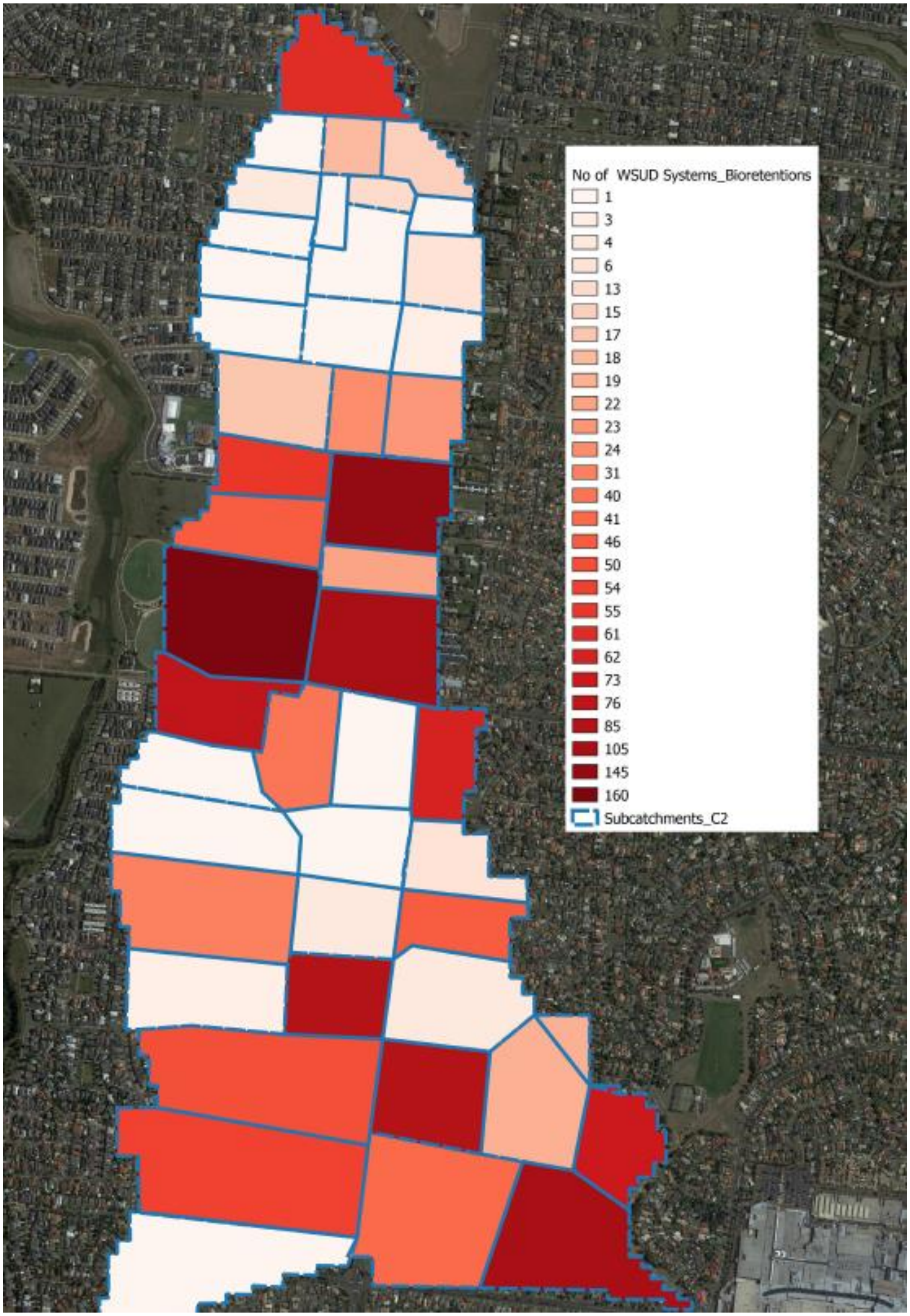


*Figure 8-6- Number of Bioretention in each sub-catchment for the maximum runoff volume reduction in C2 Outlet*

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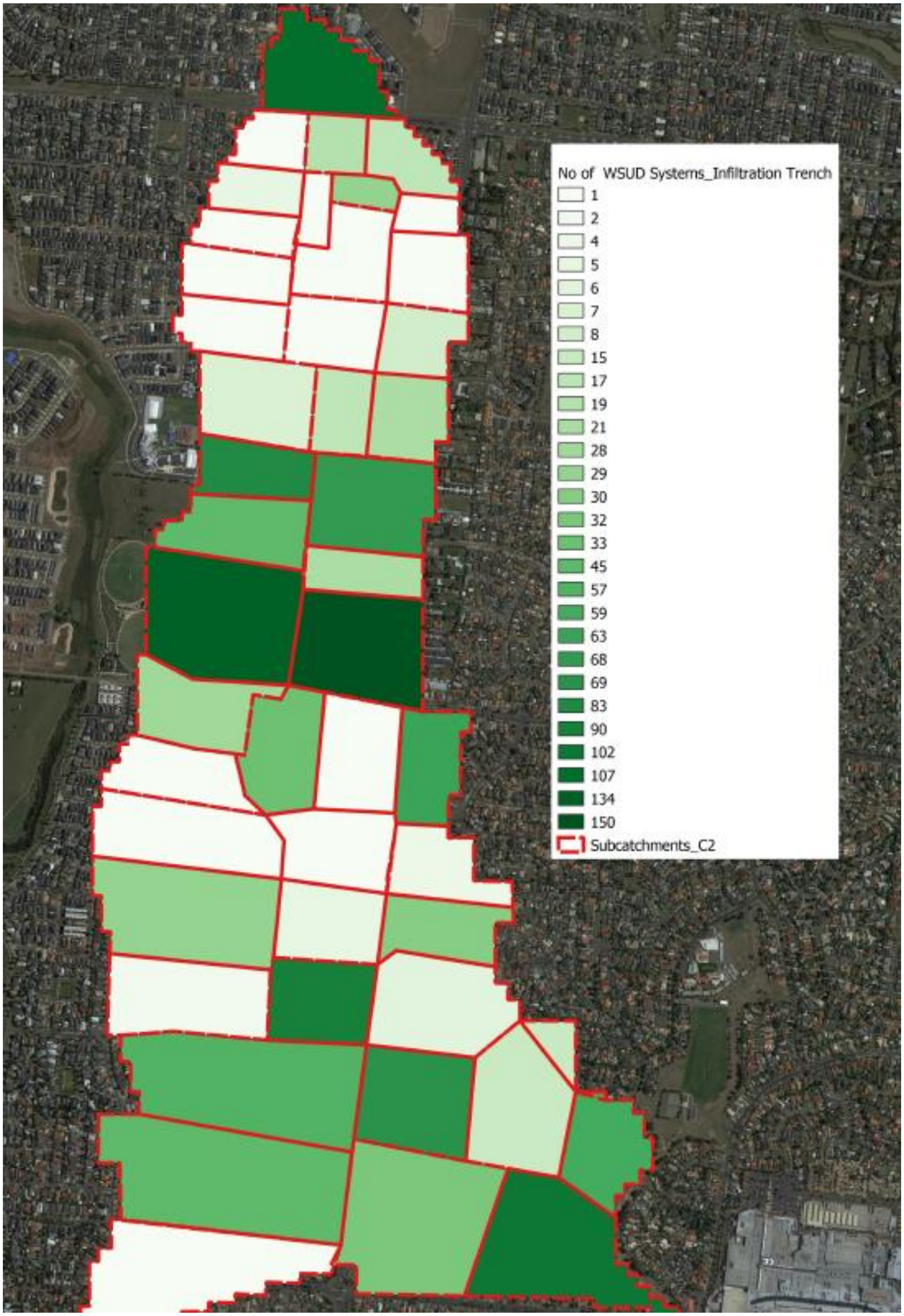



*Figure 8-14- Number of Infiltration Trenches in each sub-catchment for the maximum runoff volume reduction in C2 Outlet*

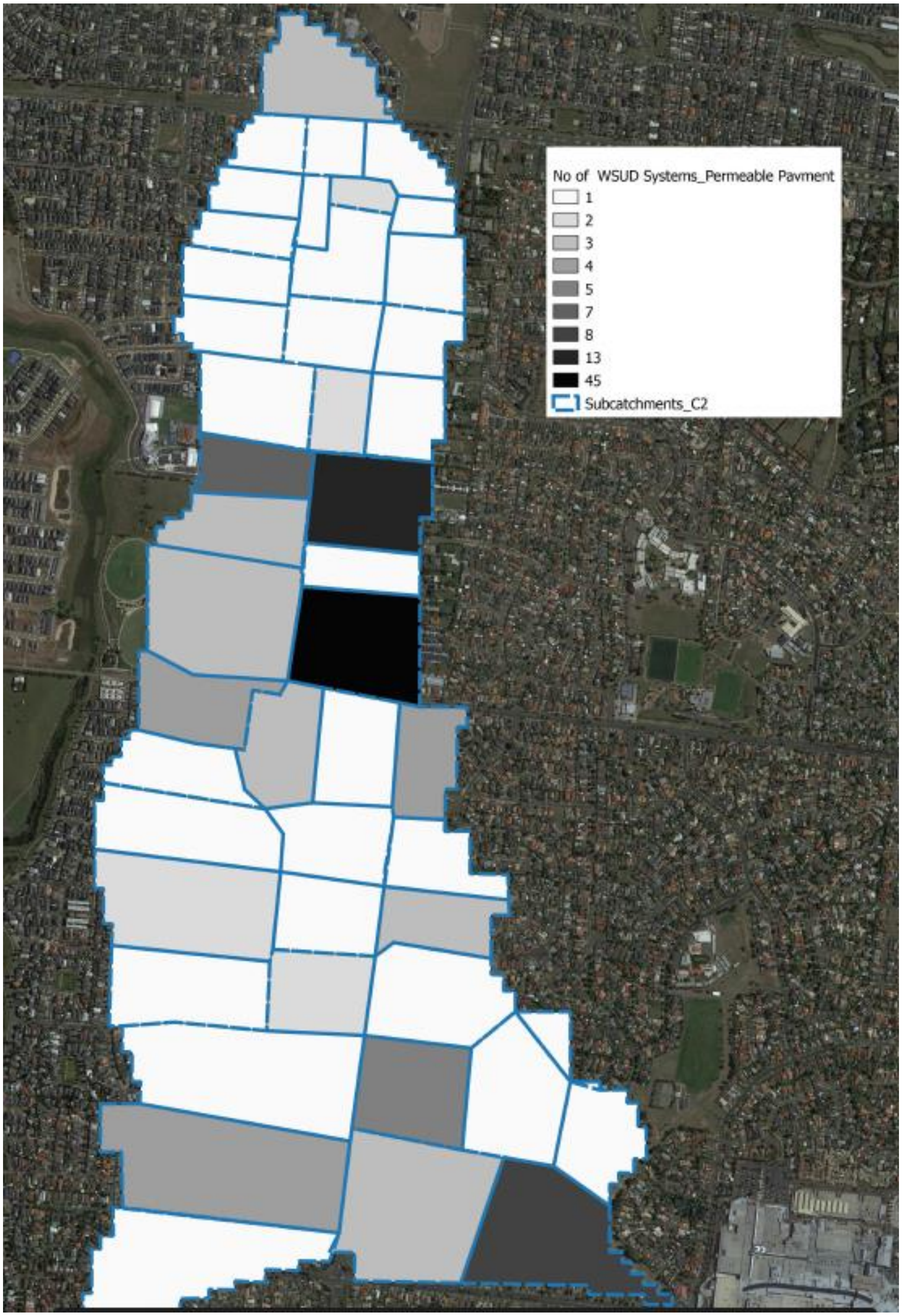
3771 


*Figure 8-75- Number of Permeable Pavements in each sub-catchment for the maximum runoff volume reduction in C2 Outlet*

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*Figure 8-8- Number of Bioretention in each sub-catchment for the maximum TSS load reduction in C2 Outlet*

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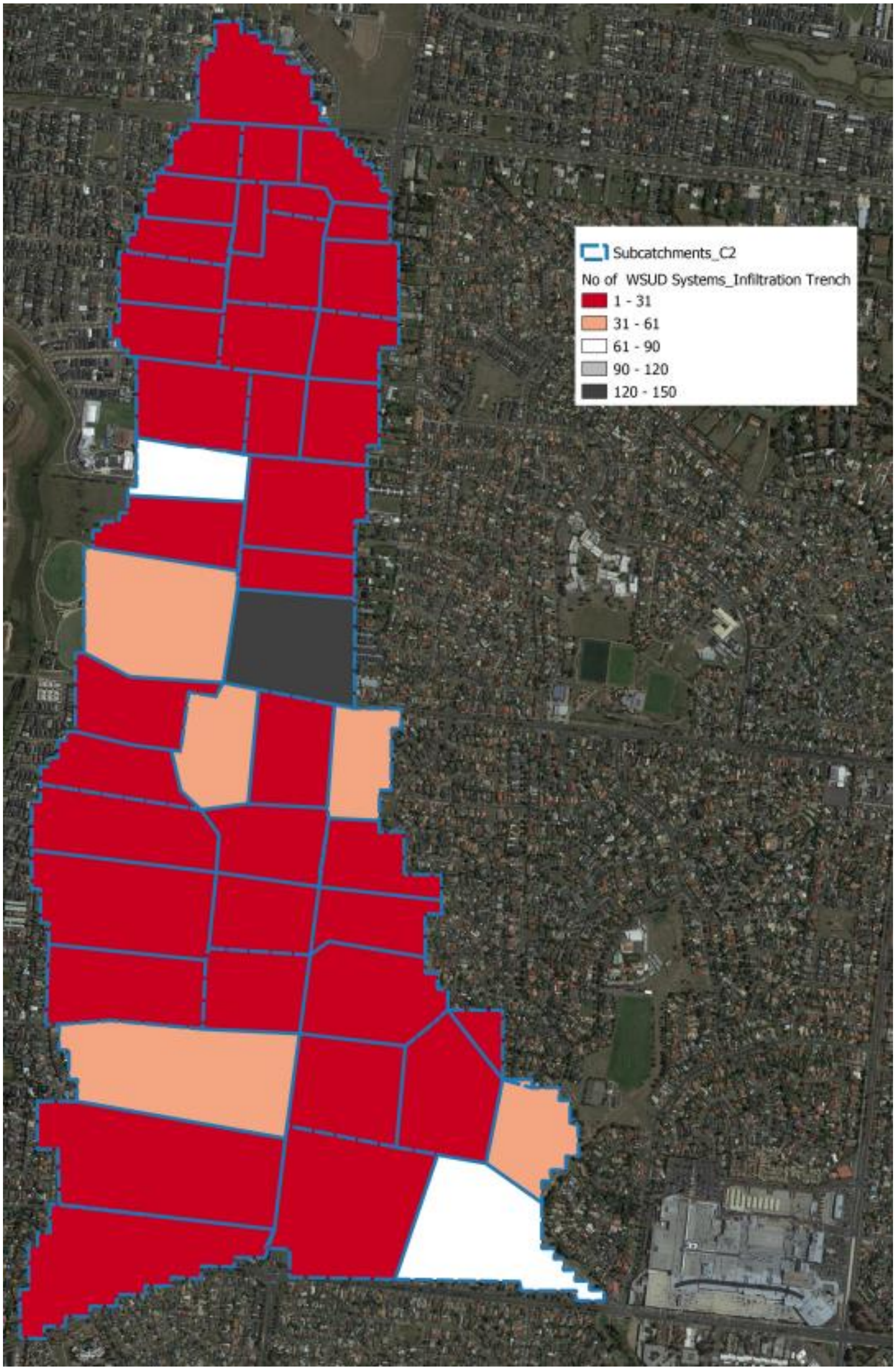


*Figure 8-917- Number of Infiltration Trenches in each sub-catchment for the maximum TSS load reduction in C2 Outlet*

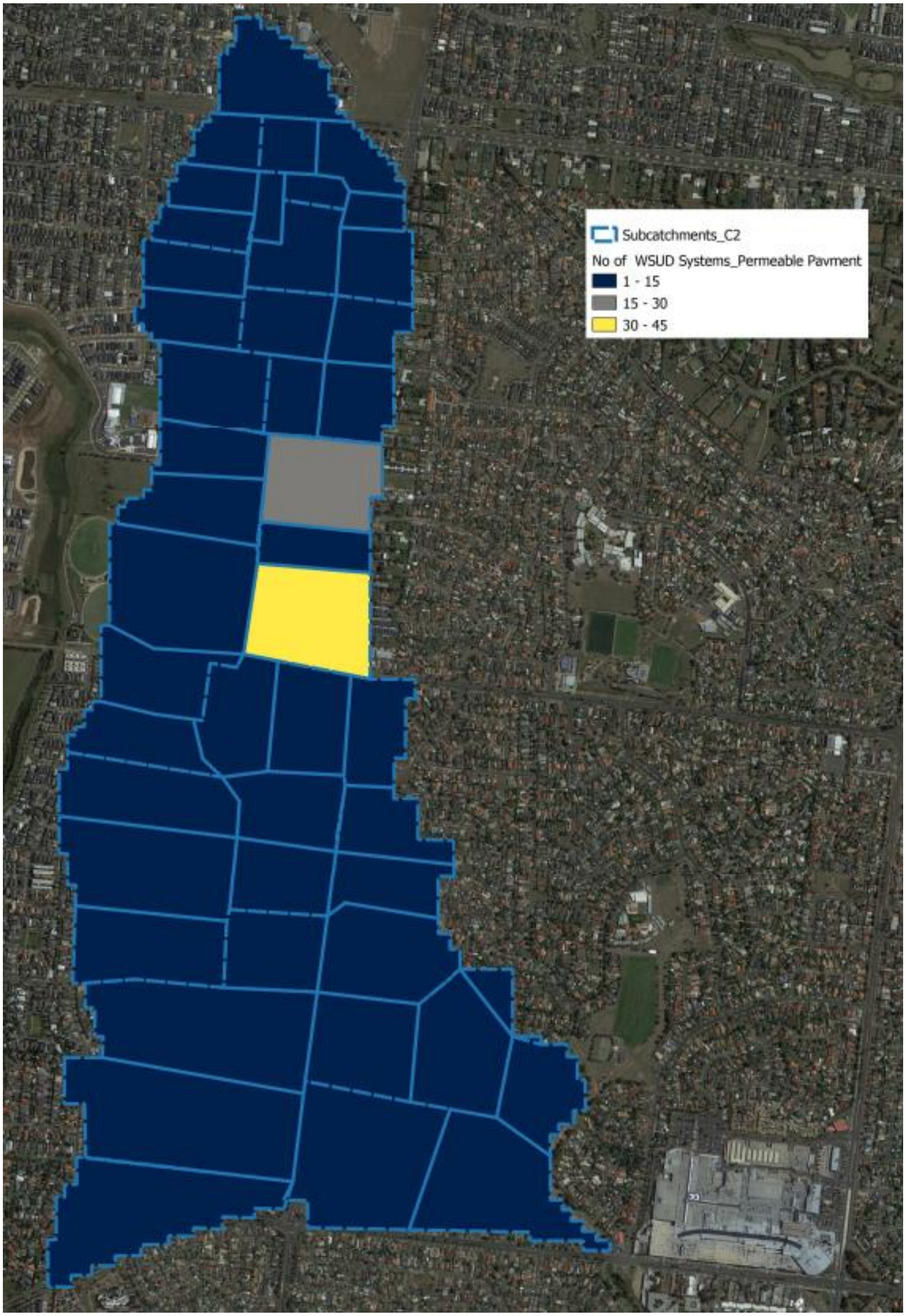


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*Figure 8-18- Number of Permeable Pavements in each sub-catchment for the maximum TSS load reduction in C2 Outlet*

Wu et al. (2019) explored almost optimal combinations of green infrastructure (WSUD techniques) that minimized overall relative cost and optimized pollution load reduction at the catchment scale in USA using the GreenPlan-IT tool. The location and selection of three locally preferred WSUD approaches (bioretention, infiltration trench, and permeable pavement) were evaluated based on their cost and efficacy. The results suggest that the best and nonoptimal choices may differ in load reduction efficiency by 30% and in overall relative cost by below $20 million dollars. Permeable pavements, tree boxes, and green roofs with unit costs were taken into consideration in different research conducted in Vietnam By Hai 2020 to determine the optimal design solutions for the Cau Bay river basin. The optimization method generated a cost-effectiveness curve that connects the TSS reduction efficiencies to the cost of installing WSUD techniques. The advantage of the optimization strategy became clear when the TSS reduction efficiencies of the perfect and non-optimal solutions were notably different at a fixed cost of LID implementation.

## 8.7. Conclusion

To determine the optimal WSUD strategies implementation for stormwater quantity and quality management at an urban catchment level, an optimisation tool created by San Francisco Estuary Institute was chosen that combines an optimisation methodology (NSGA-II) with a stormwater model (SWMM5). To evaluate possible stormwater quantity and pollutant load reduction solutions related with various combinations of WSUD technique deployment, three WSUD approaches including bioretention, infiltration trenches and permeable pavements were chosen, and their performance was examined at an allotment scale.

The final result of the optimisation processes was a cost-effectiveness curve that represented the best management costs for various levels of reductions. The widespread between the optimal frontier and intermediate solutions produced during the optimisation process as well as the diminishing marginal returns connected with an increasing number of WSUD installations serve to highlight the benefit of using an optimisation approach to help identify the most cost-effective solution for achieving a specific reduction goal or within a constrained budget.

The research analysis can provide stormwater managers with a range of nearly ideal retrofit and buildout scenarios that consider the benefits to the environment and the costs of various WSUD options. The possibilities presented here could serve as a roadmap for future investments in storm water management in any urban area which is growing too fast.

Given the optimisation formulation and the sensitivity of the WSUD costs and sizing criteria, assumptions about those parameters had an impact on the results. Therefore, to enable a successful and meaningful application of the approach wherever it is utilised, site-specific design and trustworthy local cost information should be employed, and sensitivity analysis and evaluation of cost-control strategies or economies of scale are advised.

The established integrative methodology offers decision-makers important data about trade-offs between multiple objectives. The catchment approach is especially beneficial since it facilitates in creating deeper WSUD implementation strategies that consider the physical interaction and dynamic processes happening within an urban catchment.

The results of this study indicate that three WSUD techniques including bioretention, infiltration trenches and permeable pavements are more effective than others at managing the quantity and quality of catchment outlet stormwater, but they also indicate that there are restrictions on reduction due to the limitations of the techniques' implementation and green spaces. Under existing condition in an urban area, it is challenging to find a suitable location in an urban area for the systems to operate. We should also be cautious because although the number of systems can significantly contribute to an urban catchment outlet stormwater management, but the life cycle costs are very difficult for local councils to cover. It was also recognised that in a developed urban catchment, the system's performance during heavy rainfall with an increase in intensity is not significantly different. This was observed through the similar reduction percentage for stormwater volume and TSS load at the study area outlet under existing and future condition. This translates to WSUD systems performing better during small storm events and less impervious area in urban catchments.

. Finally, it is critical to interpret the optimisation results in light of the problem description, model assumptions, and sensitivity of WSUD parameters in each individual application. A representative baseline condition must be established with a high level of confidence in order to ensure that the optimisation results are meaningful. This is especially true when cost-benefit optimisation of future management objectives is the focus of the modelling effort because the baseline model serves as the foundation for comparative assessment of various WSUD implementation scenarios.

# 9. Chapter 9: Summary, Conclusions, and Recommendations

The goal of this study was to evaluate whether or at what scale were WSUD approaches may be beneficial in a developed urban catchment to manage the catchment's outlet stormwater quality and quantity. There were three research questions for this study.

1- What are the constrains on WSUD implementation in an existing urban area?

Benefits of WSUD techniques adoption in greenfield and new development areas have been covered in several research. However, this study discovered several limitations on the use of WSUD strategies in existing urban catchment areas. A list of limitations was developed and provided to stormwater engineers and specialists from Melbourne, Australia's local councils to get their opinions on the difficulties in implementing the techniques under existing condition and to better understand the appropriate WSUD techniques that may be used at the allotment and street scales. The first research question was properly addressed by the task's findings, which are covered in chapter four of this thesis. The findings highlighted a significant challenge in implementing Water-Sensitive Urban Design (WSUD) techniques within high-density urban environments. Limited space poses a critical constraint for street-scale solutions such as sedimentation basins, raingardens, constructed wetlands, ponds, and lakes. While smaller bio-retention systems and swales can be integrated into the urban landscape, their effectiveness in managing large storm events remains a concern. Similarly, lot-scale solutions like dry wells and rainwater tanks also face spatial limitations, making widespread adoption difficult.

From an urban planning perspective, local governments actively promote lot-scale WSUD systems to enhance stormwater management. However, key barriers to adoption include residents' willingness to implement these systems, maintenance costs, and a lack of awareness regarding their functionality. Consequently, urban planners and policymakers must navigate multiple challenges, including selecting the most effective WSUD strategies, ensuring system efficiency, minimizing costs, and addressing public concerns. Achieving a balance between technical feasibility, economic viability, and community acceptance is crucial for sustainable urban water management and long-term resilience. The constraints and challenges associated with WSUD implementation at various scales, as identified in this research, will offer valuable guidance for local council planners. By providing insights into the effectiveness and feasibility of different techniques at both allotment and catchment scales, this study supports informed decision-making in urban planning. It emphasizes the need to balance technical constraints with community needs while enhancing the resilience of future urban developments. The findings will assist planners in selecting the most suitable WSUD strategies that align with long-term sustainability goals, infrastructure limitations, and public acceptance.

2- What are the WSUD approaches which can be implemented optimally considering their locations, size and type to manage the stormwater quality and quantity?

To answer this question stormwater modelling scenario running at different scales (i.e., allotment, sub-catchment, and the whole catchment) was undertaken plus an optimisation task for the whole study area. Following sections summarised the findings of each task:

***<u>Allotment scale</u>***

The results of the survey conducted, which are detailed in chapter four, were taken into consideration to understand the WSUD techniques that may be applied ideally under the current and future conditions at urban catchments at different scales including an allotment and sub-catchment. According to the survey's findings, the most popular methods for hydraulic modelling and scenario running were picked as bio-retention (raingardens), vegetated swales, infiltration trenches, permeable pavements, and rainwater tanks.

For a typical allotment with a single dwelling and WSUD assets retrofitted, Chapter six of this thesis has examined the extent to which various WSUD approaches can reduce stormwater runoff volume and peak flow size compared with two dwellings in the future condition with more impervious area.

Due to the townhouses' replacement of the single dwelling with two or more of them in the condition with redevelopment with two dwellings, the allotment's impervious ratio was increased for the future condition. The residential lot was chosen from a West Melbourne urban catchment, the research study area. For scenario modelling, five chosen WSUD assets—rainwater tanks, infiltration trenches, rain gardens, and vegetated swales—and combinations of these were examined.

The study's findings demonstrated that, when compared to the other systems, vegetated swales and rainwater tanks without reuse were less effective at reducing the mean annual runoff volume and peak size at the allotment under both current and future development conditions with a higher impervious fraction (i.e., 80 versus 60%)

Peak flow rates for storm events with an ARI of 1 in 5 years corresponding to the typical design standard for the piped drainage system were reduced by 90% for the future redeveloped condition. It was also observed that smaller events up to a 1 in 1-year ARI were usually eliminated, reducing runoff discharges' frequency, and causing reduced disturbance to the waterways. A combination of three assets including a tank, a rain garden and an infiltration trench-maintained peaks discharge for the 1.5-year ARI close to the household's predevelopment levels floe under future developed conditions. Besides, there were significant reductions in peak flows throughout the storm events with an ARI of 1 in 10 up to 86%. Total mean annual runoff volume reductions were observed up to 90% with three WSUD assets for the future developed condition. While they had the highest costs, the combination of a rainwater tank, rain garden and infiltration trench assets provided the most reduction in stormwater runoff volume and peak flow size undertaking MUSIC and PCSWMM modelling tools. The reduction percentage of peak size and mean annual runoff volume were similar for the selected WSUD assets in modelling tools MUSIC and PCSWMM. An LCC analysis was undertaken using the MUSIC model to rank the considered scenarios. The LCC analysis ranked the scenarios for implementation at the selected allotment from the least to the costliest one.

Effectiveness of the chosen WSUD assets was confirmed at a residential allotment scale in this study to manage the future urbanization impact, urban floods. High reduction percent of peak flow and runoff volume will be enough to maintain the allotments' pre-development condition under future development condition. However, the systems LCC and the maintenance process should be considered essential criteria for the system implementation at a residential allotment scale.

### ***Sub-catchment and catchment scales***

The effectiveness of rainwater tanks and raingardens were evaluated on stormwater runoff volume and peak flow size reduction for small sub-catchments in a residential allotment scale and up to 2 ha sub-catchments and the whole study area using MUSIC and PCSWMM models. Results are summarised in chapter six.

Various climate scenarios were simulated using both continuous and individual event modelling approaches. Continuous simulation was conducted for a wet period spanning from December 1, 2010, to March 31, 2011, based on rainfall data collected from the Laverton rainfall station, situated in proximity to the study area. For individual storm events, short-term patterns were obtained from the BoM Hub data website. Additionally, maximum rainfall data for 10% and 50% Annual Exceedance Probability (AEP) events were extracted from the rainfall Intensity-Frequency-Duration (IFD) data system on the BoM website for the study area. Ten different storm patterns were considered for each exceedance probability to define the single storm events.

Implementing rainwater tanks and raingardens in only 28% of the total area within small sub-catchments resulted in a reduction of runoff volume by up to 30% over the designated period. Furthermore, the installation of 3 kL and 5 kL tanks led to a decrease in peak flow magnitude by up to 15% at the catchment outlet during the specified period. Additionally, when combined with raingardens, 3 kL and 5 kL tanks reduced peak flows in the area by up to 18% for 10% Annual Exceedance Probability (AEP) events and up to 14% for 50% AEP events.

Using MUSIC model runoff data for the catchment, the partial flow analysis method was employed to evaluate the effectiveness of rainwater tanks and raingardens in reducing flow peak sizes across three different scales: individual allotments, individual sub-catchments, and sub-catchments combined within the area. Possible flow scenarios were plotted for these specific areas to assess catchment behaviour under varying rainfall intensities. Probability fits were also plotted and compared for defined scenarios. The results indicated that implementing rainwater tanks at the allotment scale brought about significant reductions in runoff peak sizes compared to the selected sub-catchment and sub-catchments combined. However, raingardens were found to be more effective in reducing runoff peak sizes across all too. At the catchment scale, there was no substantial disparity in reduction of stormwater runoff by implementing 3 kL or 5 kL tanks. Initial findings from MUSIC modelling indicated that raingardens could offer greater benefits in reducing peak flows across both allotment and catchment scales, spanning various Annual Recurrence Intervals (ARIs). However, linking additional roofs to rainwater tanks may enhance the likelihood of reducing peak flows at the catchment scale. Raingardens showed 3 to 5% more effectiveness for peak flow reductions for big events (ARI >5) at a catchment scale compared to rainwater tanks. Both rainwater tanks and raingardens showed 30 to 35% runoff volume reduction at a catchment scale.

## ***Optimisation***

Permeable pavement, infiltration trenches, and bioretention techniques were examined at the next level in sub-catchment scale in this study They were selected with consideration for how they affected peak flow size and stormwater runoff volume reduction at various scales, as well as following a literature review and common WSUD techniques in Australia's urban catchments. Chapter seven covered the details as the techniques were presented in PCSWMM model.

To determine the optimal WSUD techniques implementation scale for stormwater quantity and quality management at the catchment outlet, an optimisation tool created by San Francisco Estuary Institute was chosen that combines an optimisation methodology (NSGA-II) with a stormwater model (PCSWMM).

The result of the optimisation processes was a cost-effectiveness curve that represented the best management costs for various levels of reductions. The widespread between the optimal frontier and intermediate solutions produced during the optimisation process as well as the diminishing marginal returns connected with an increasing number of WSUD installations serve to highlight the benefit of using an optimisation approach to help identify the most cost-effective solution for achieving a specific reduction goal or within a constrained budget.

The research analysis can provide stormwater managers with a range of nearly ideal retrofit and buildout scenarios that consider the benefits to the environment and the costs of various WSUD options. The possibilities presented here could serve as a roadmap for future investments in storm water management in any urban area which is growing too fast.

Given the optimisation formulation and the sensitivity of the WSUD costs and sizing criteria, assumptions about those parameters had an impact on the results. Therefore, to enable a successful and meaningful application of the approach wherever it is utilised, site-specific design and trustworthy local cost information should be employed, and sensitivity analysis and evaluation of cost-control strategies or economies of scale are advised.

The results of this study indicate that three WSUD techniques including bioretention, infiltration trenches and permeable pavements are more effective than others at managing the quantity and quality of catchment outlet stormwater, but they also indicate that there are restrictions on reduction due to the limitations of the techniques' implementation and green spaces.

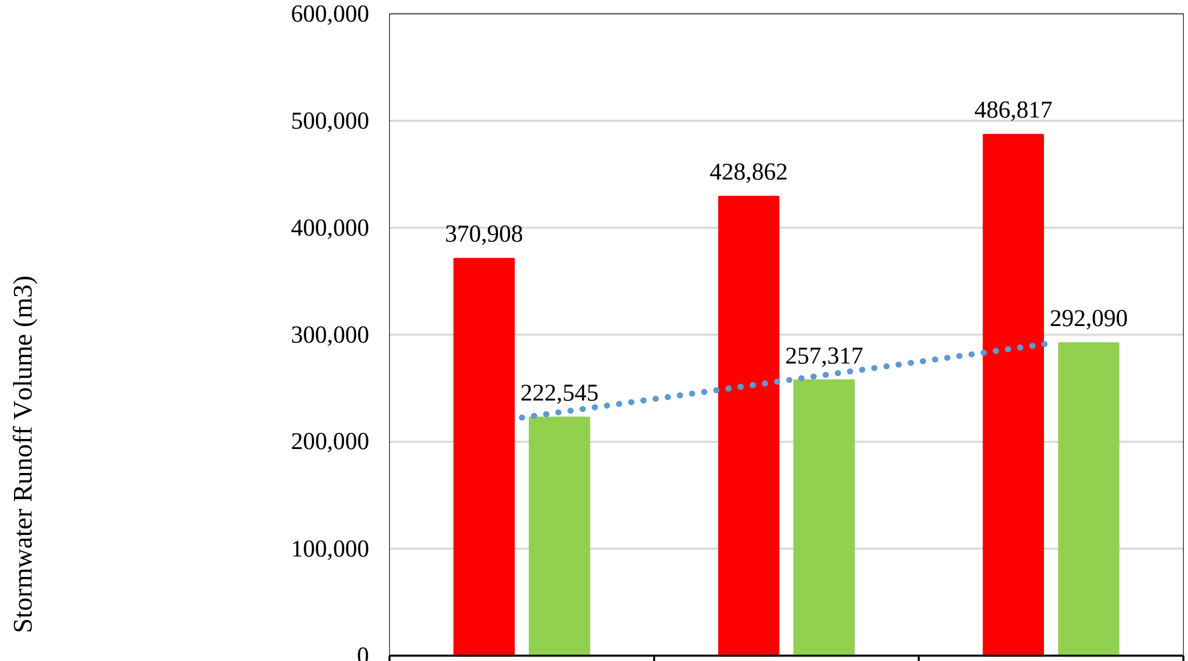
Under existing condition in an urban area, it is pretty challenging to find a suitable location in an urban area for the techniques to install.

It should be noted that while the number of systems may be significantly helpful in managing stormwater runoff quality and quantity from urban catchment outlets, local councils find it extremely difficult to afford the associated operating and capital costs. It was also acknowledged that there is no noticeable difference in the system's functioning during periods of intense rainfall in a developed urban catchment.

In summarising the research results to respond to question two, it can be said that WSUD techniques are effective at managing both the volume and quality of urban stormwater at both allotment and sub-catchment scales depending on available space, their combinations of techniques, and storm event severity. They were shown to have more efficacy for minor storm events; however, the cost of implementation and maintenance should be viewed as a substantial barrier for their widespread adoption. The findings indicate that the difference in total relative cost between optimal and nonoptimal solutions can reach well below $20 million dollars, and the effectiveness of load reduction for both quality and quantity at the study catchment could vary as much as 33% to 40% adopting a combination of raingardens, infiltration trenches and permeable pavements. A key distinction between this study and the extensive body of existing research lies in the methodological approach, particularly in the use of real-collected data for model development and calibration. Unlike many previous studies that rely on theoretical assumptions or generalized datasets, this research is grounded in actual observed data, ensuring that the modelling process accurately reflects real hydrological and environmental conditions. By incorporating site-specific data and rigorously calibrating the model, the findings offer a higher degree of reliability and applicability to practical urban stormwater management scenarios at different scales. This enhances confidence in the results, allowing for more precise predictions of WSUD performance and enabling local planners and policymakers to make well-informed decisions tailored to specific urban contexts. Ultimately, this approach contributes to bridging the gap between theoretical modelling and real-world implementation, advancing the field of urban water management through evidence-based, data-driven solutions.

3- How long the augmentation of the stormwater system can be deferred in the study area by WSUD approaches implementation considering climate variability and urbanisation?

A number of factors, including the unique characteristics of an urban catchment, climatic variability, the rate of urbanisation, the percentage of the WSUD measures put in place, and the network's age, capacity, maintenance, and upgrade, can greatly affect how long it takes to augment the stormwater system. There is no standard response to this question because it necessitates a thorough examination of each study area individually. However, given the analysis done for this study, it is possible to anticipate when the current stormwater system will need to be expanded by looking at Figure 9-1:




| Scenarios | Current Catchment Condition | 10 Years Later | 20 Years later |
|---|---|---|---|
| Stormwater Volume (m3) | 370,908 | 428,862 | 486,817 |
| Stormwater Volume Future Reduced (m3) | 222,545 | 257,317 | 292,090 |

*Figure 9-1- Stormwater volume reduction comparison under existing and future scenarios*

The stormwater runoff volume for Catchment 1 is shown in Figure 8-1, with only 10% adoption of WSUD under the existing and future impervious fractions, as chosen for the present study. The adopted WSUD strategies (bioretention, infiltration trench, and permeable pavements) demonstrated a maximum reduction of stormwater volume up to 33 to 40 percent in both the current and future scenarios at a cost of $14 to $16 million dollars. This demonstrates that, given the level of urbanisation and rainfall intensity in the study area, the techniques' performance has a limit. The local councils will be able to delay their existing stormwater network augmentation, with the combination of the WSUD techniques. This cost is though highly relevant to their implementation and maintenance budget for the techniques, as well as their budget for minor drainage upgrades to mitigate flood circumstances across the municipality. Limited space in light of the municipality's growing densities and community opposition to the deployment of WSUD techniques in their green, open spaces should also be taken into account.

It was concluded that, in comparison to the current urban catchments conditions, the performance of the WSUD techniques would not shift as a result of climate change because the overall runoff reduction % stayed constant as storm intensities increased for the future condition.

It was also acknowledged that, to improve stormwater runoff reduction, investments in larger WSUD may be required as storm strength increases; however, given space constraints, this aspect may not be highly successful. Overall, the open spaces in the urban catchment were crucial in restricting the effectiveness of the WSUD techniques; that being said, the techniques' installation might still be viewed as advantageous.

This modelling study provides information that can help determine whether implementing WSUD controls is a feasible option and, if so, what the best implementation strategies are. This methodology offers WSUD solutions that are high-benefit, low-cost, and most likely to be adopted and maintained under existing urban catchment and when it’s developed to the maximum impervious fraction.

## 9.1. Recommendations

This study examined the deployment of WSUD techniques in allotment and sub-catchments and the existing green spaces across the study area to graph the cost-effective curves of the most preferred adopted WSUD techniques in reducing stormwater volume and TSS load at the catchment outlet. It aimed to illustrate the cost-benefit curves associated with stormwater volume and pollutant load (i.e., TSS) reduction. This was done following an assessment of the effectiveness of the techniques on their own and in combination across various scales including residential lot, street sub-catchments and the whole catchment. While the techniques demonstrated effective impacts on improving stormwater quantity at different scales, either individually or in combination, the cost-effective curves revealed a maximum reduction of 40% in stormwater volume at the catchment outlet and even less reduction up to 33% for the future condition under climate change and urbanization. However, the percentage for pollutant load reduction at the catchment outlet was only up to 33%, which falls short of meeting the target set by the established guidelines. Analysis of water quality samples collected in the study area revealed that total nitrogen (N) and phosphorus (P) concentrations were consistently low — below thresholds that would typically justify costly nutrient reduction interventions. While a specific percentage of reduction in runoff volumes and stormwater pollutant (i.e., nutrients and phosphorus) is an important target in many areas, this research findings suggested that WSUD implementation for pollutants removal may not yield significant returns in some specific context.

This confirms the need for additional techniques, despite the potential cost implications not being effective. It was understood that the techniques' effectiveness decreases in areas with high rainfall and extensive land use. Therefore, it is advised that WSUD be taken into account as a potential solution need accurate budget planning and justification to lower the study area's stormwater volume at the catchment outlet. This study at least showed that the storm water system's load can be decreased by strategically putting WSUD techniques within the allotment or the available green open spaces throughout the urban catchment. Available space was ranked as the most important barrier for sedimentation basins, raingardens, constructed wetlands, ponds and lakes implementation in urban areas as they are. While the performance of a solution won't be adversely affected by the addition of more WSUD techniques, community consultation and strategies to encourage community need more focus and investigation.

As the effects of climate change continue to raise the risk, these solutions could also be examined in conjunction with other stormwater management techniques like underground harvesting tanks and catchment scale rainwater tanks (if they are preferred option to be implemented by an authority) to see which one is the most economical in reducing the peak size and volume of urban stormwater runoff. These solutions and the methods created could be used as a starting point for examining various stormwater management strategies and their efficacy.

The framework employed here makes it simple to extend the work. Engineers and decision makers can find the optimisation framework useful for evaluating a variety of options and compiling a list of options to take into account for the final design. To maximise the benefits of the WSUD techniques, other objectives could be added to the optimisation model. For example, to evaluate the efficacy of them in removing contaminants from stormwater, water quality targets could be included. Doing a survey in the region to learn more about how people really feel about the idea of the techniques on their allotments might be one method to expand this study and increase its efficacy. Lastly, research needs to be done to evaluate the degree of uncertainty in the data. Comprehending the uncertainty of data is crucial when working with extensive datasets like the ones obtained in this investigation.

Local governments can enhance urban drainage and WSUD assessment frameworks by developing standardized optimization modules that integrate seamlessly with existing planning and management systems. Additionally, offering specialized training programs for government planners and consultants on the application of optimization tools, such as GreenPlan-IT, can strengthen the capacity for data-driven stormwater management.

To promote a holistic approach, fostering cross-disciplinary collaboration among engineers, urban planners, economists, and environmental scientists is essential for embedding optimization into comprehensive urban planning strategies. Policymakers should also introduce incentives that mandate or reward the application of optimization in WSUD and stormwater infrastructure projects, ensuring its widespread adoption. Establishing industry-wide guidelines and best-practice documents will further standardize optimization methodologies, facilitating consistency in implementation. Strengthening partnerships between academia and industry can accelerate the translation of research-driven optimization techniques into practical, real-world applications.

Furthermore, integrating real-time data and model recalibration processes will enhance the accuracy and reliability of optimization outcomes. The use of digital twins and smart water management systems should be prioritized to bridge the gap between theoretical models and real-world performance. Lastly, encouraging active stakeholder engagement is crucial to ensuring that optimization strategies align with community needs and contribute to long-term urban resilience and sustainability.

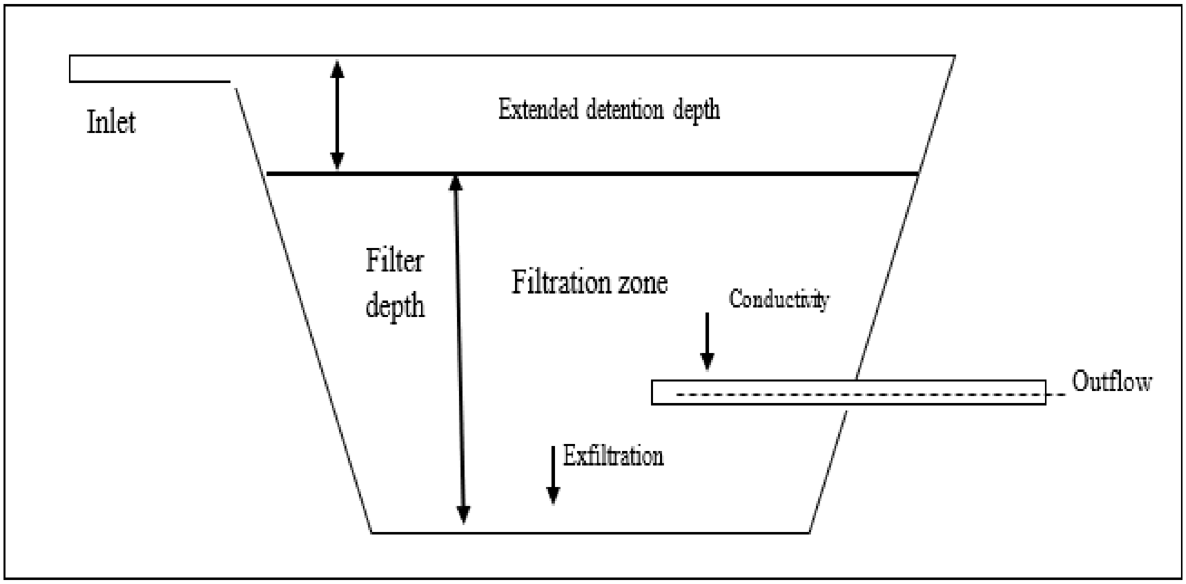
In conclusion, it is important to highlight that the assumptions made in this study were intentionally aligned with existing conditions and the prevailing development patterns of urban areas, as reflected in the current planning context and available data. These assumptions were necessary to establish a baseline model that is both manageable and representative of the urban form at the time the study was conducted.

That said, the research acknowledges that urban planning and suburban development are inherently non-linear and often exhibit a mosaic-like progression, shaped by a range of social, political, and economic factors. Such complexities can lead to spatial and temporal variability in infrastructure deployment, land use patterns, and hydrological responses—all of which may influence the performance and effectiveness of WSUD strategies.

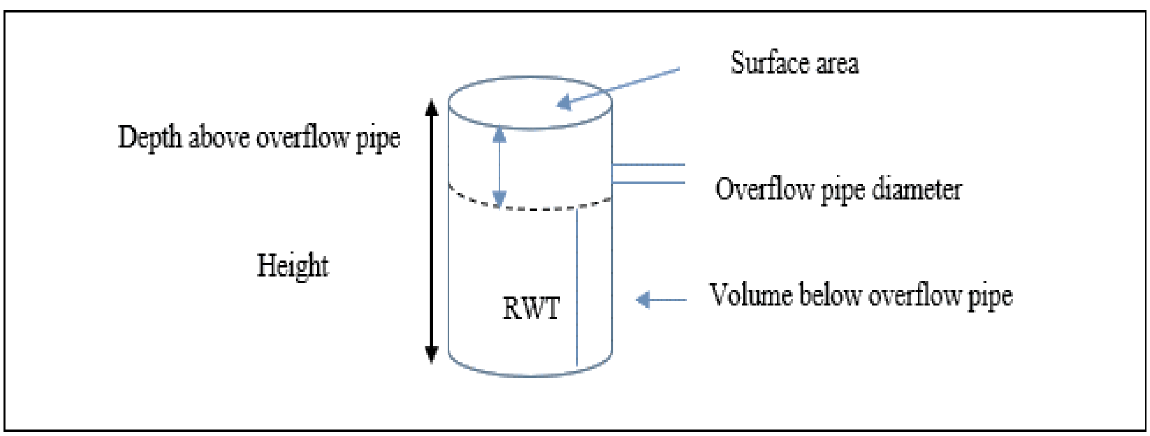
To account for these factors, the modelling framework developed in this study was designed with a high degree of flexibility. Although the model scenarios are grounded in current conditions, the framework allows for input parameters to be readily adjusted as planning contexts shift or as new data emerge. This adaptability enables the model to support ongoing, responsive planning efforts, allowing decision-makers to test and simulate alternative development trajectories in the face of evolving urban dynamics.

Moreover, the thesis explicitly addresses the limitations associated with assuming linear patterns of urban development. It underscores the need for future research to apply and test the framework under more fragmented, non-linear growth scenarios. Through this approach, the study not only retains its relevance under existing conditions but also establishes a foundation for more nuanced and context-sensitive modelling in increasingly complex urban environments.

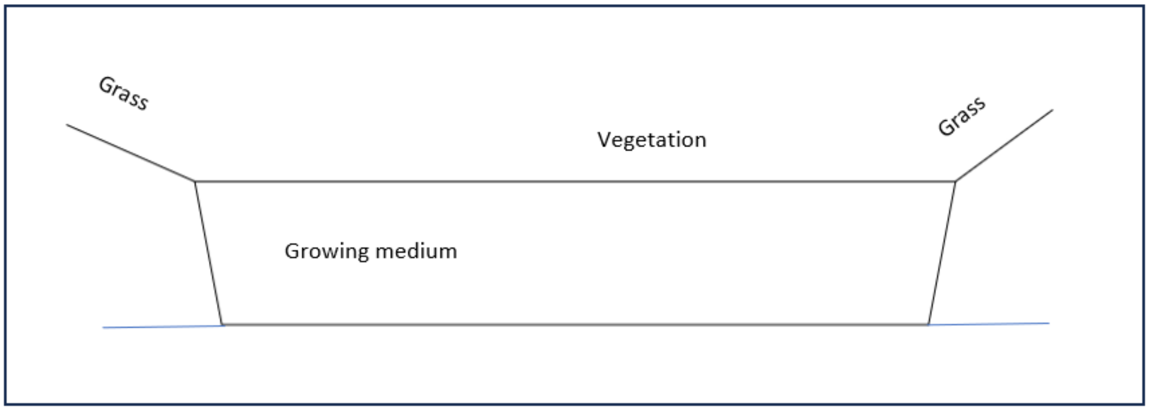
# Appendix A



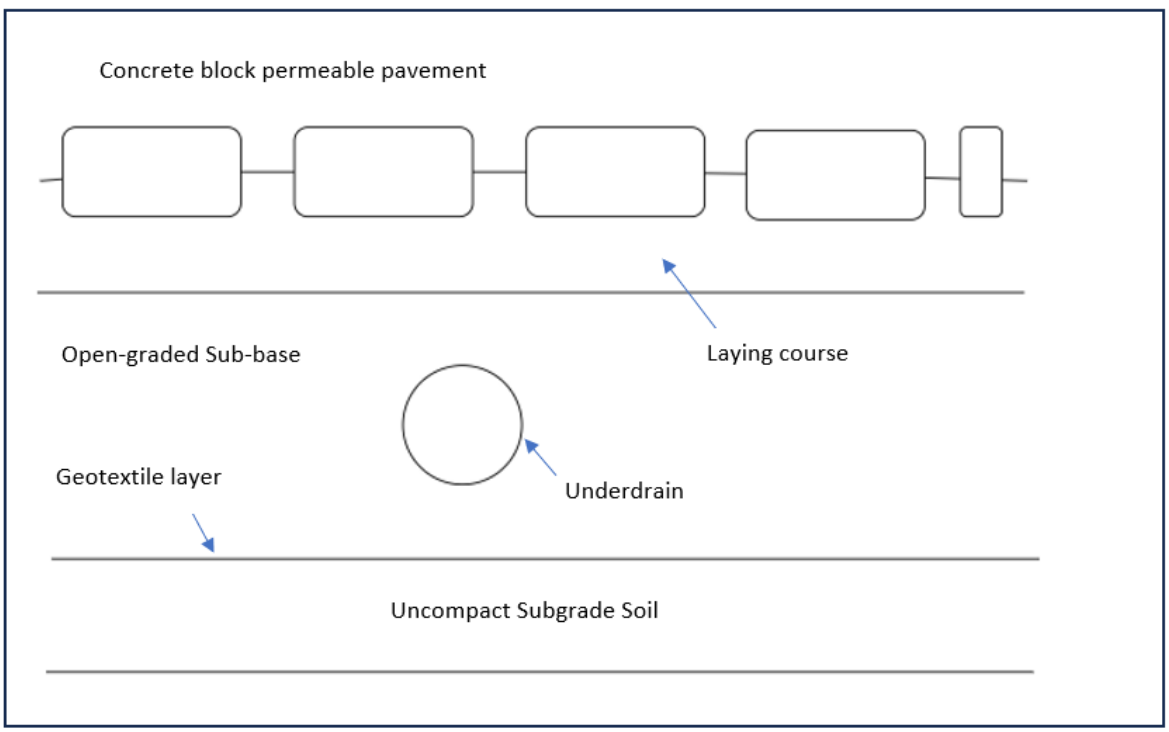

Schematic diagram of a raingarden



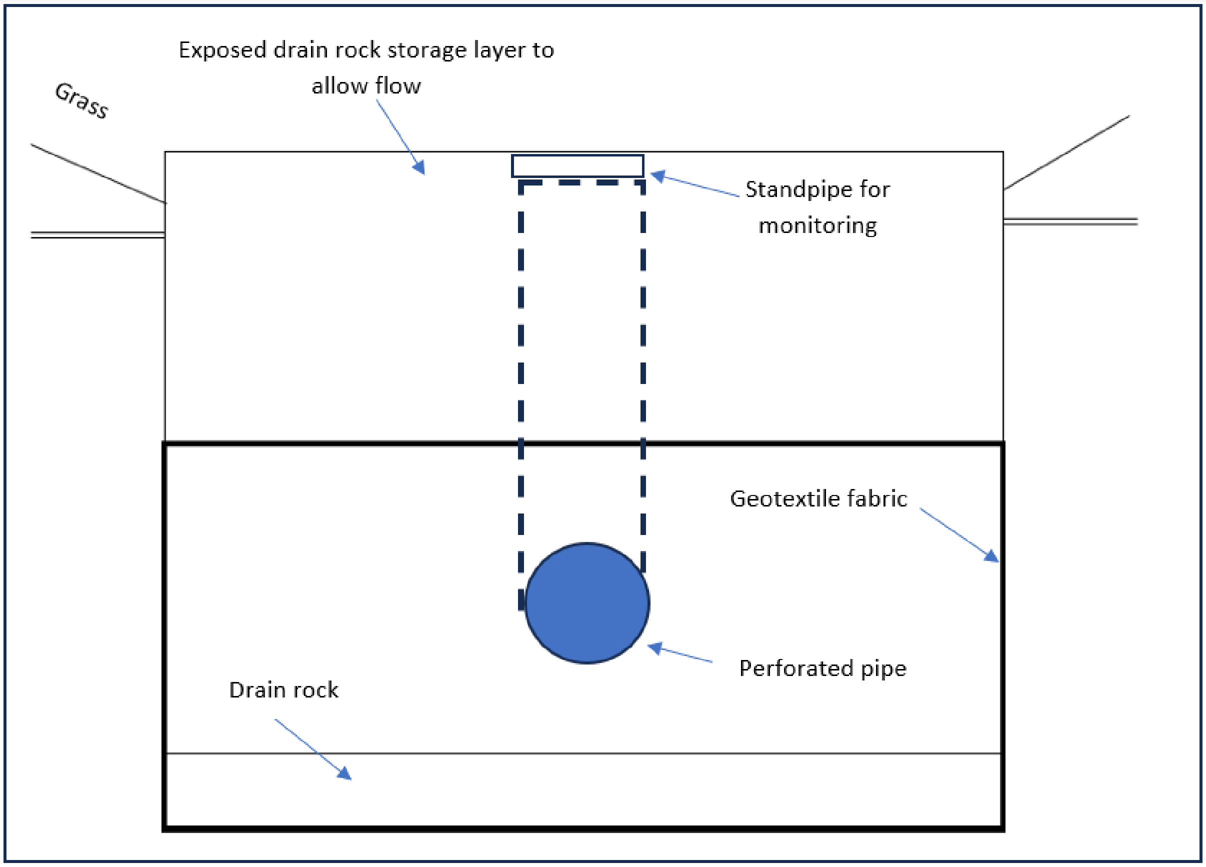

Schematic diagram of a rainwater tank




Schematic diagram of a swale




Schematic diagram of a permeable pavement




Schematic diagram of an infiltration trench

# Appendix B

## ***Average allotment size in the study area***

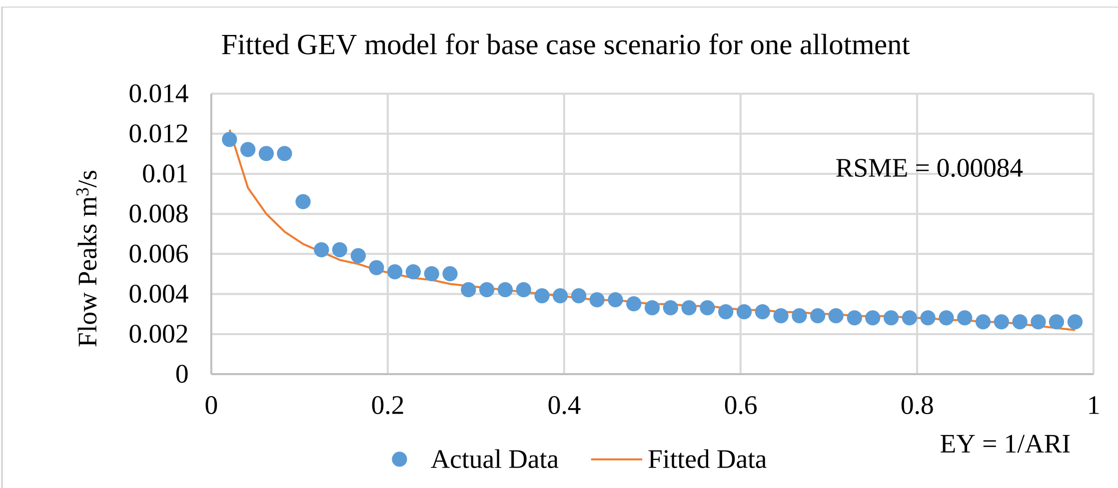
36 allotments selected randomly in the study area from each sub-catchment and their areas exported from GIS layer. Total allotment area, roof area, paving and garden areas are presented in Table A4-1.

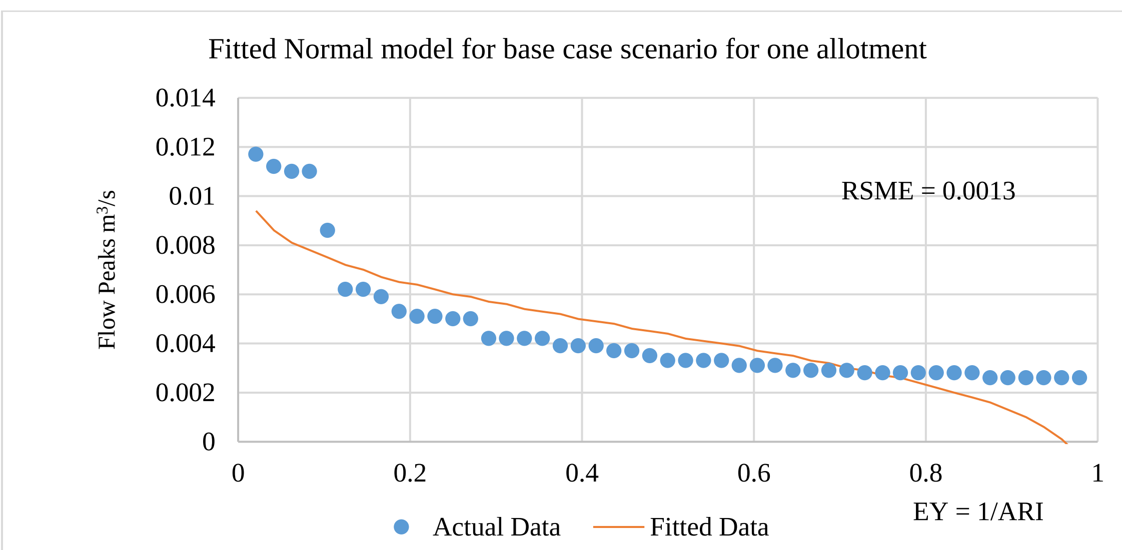
| Number | Total area ($m^2$) | Roof area ($m^2$) | Paving and driveway ($m^2$) | Gardens and lawn ($m^2$) |
|---|---|---|---|---|
| 1 | 550 | 300 | 90 | 160 |
| 2 | 500 | 265 | 155 | 80 |
| 3 | 530 | 240 | 70 | 220 |
| 4 | 410 | 190 | 80 | 140 |
| 5 | 610 | 330 | 80 | 200 |
| 6 | 360 | 160 | 100 | 100 |
| 7 | 470 | 220 | 110 | 140 |
| 8 | 380 | 160 | 100 | 120 |
| 9 | 495 | 265 | 70 | 160 |
| 10 | 460 | 180 | 120 | 160 |
| 11 | 400 | 160 | 80 | 160 |
| 12 | 550 | 220 | 80 | 250 |
| 13 | 510 | 210 | 80 | 220 |
| 14 | 550 | 200 | 130 | 220 |
| 15 | 500 | 260 | 90 | 150 |
| 16 | 531 | 380 | 65 | 86 |
| 17 | 657 | 382 | 60 | 215 |
| 18 | 620 | 290 | 100 | 230 |
| 19 | 465 | 290 | 85 | 90 |
| 20 | 470 | 330 | 75 | 65 |
| 21 | 385 | 240 | 55 | 90 |
| 22 | 450 | 230 | 50 | 170 |
| 23 | 430 | 240 | 60 | 130 |
| 24 | 460 | 270 | 60 | 130 |
| 25 | 520 | 255 | 65 | 200 |
| 26 | 550 | 230 | 50 | 270 |
| 27 | 535 | 260 | 55 | 220 |
| 28 | 505 | 235 | 50 | 220 |
| 29 | 470 | 185 | 65 | 220 |
| 30 | 600 | 250 | 70 | 280 |
| 31 | 430 | 170 | 60 | 200 |
| 32 | 470 | 180 | 70 | 220 |
| 33 | 465 | 170 | 65 | 230 |
| 34 | 425 | 180 | 65 | 180 |
| 35 | 445 | 220 | 75 | 150 |
| 36 | 510 | 210 | 90 | 210 |
| Average | **491** | **238** | **78** | **175** |

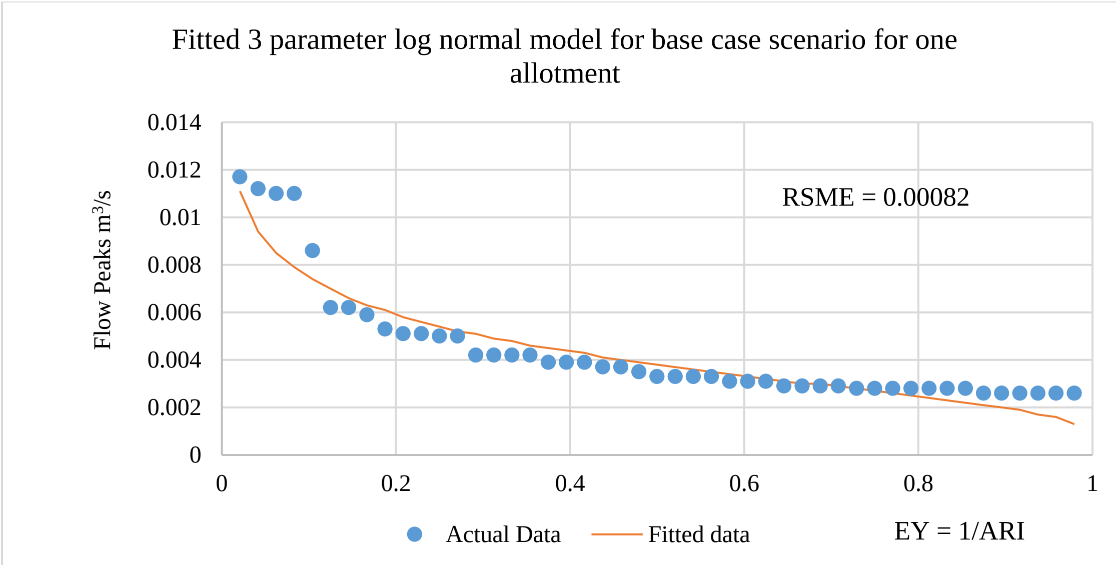
## ***Selection of suitable probability fitting model (Statistical distribution)***

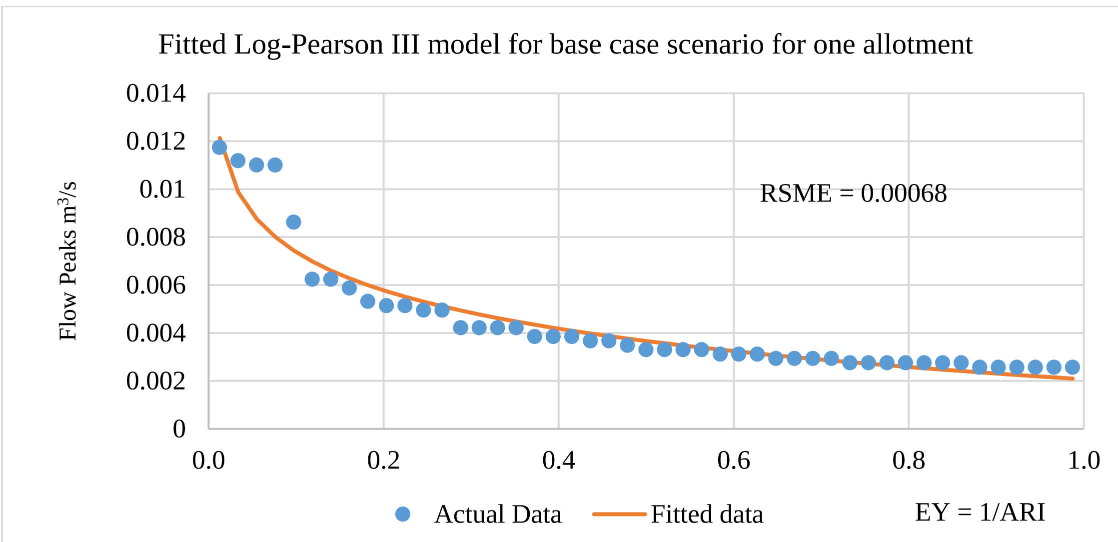
Five different probability models tested for the plotted data from MUSIC model for each scenario. according to the graphs presented in this appendix and Rout Square Mean Error (RSME) calculation, the best fit was selected as Log-Pearson III. Graphs A5-1 to A5-5 present the fitted models and Table A5-1 shows the RSME amounts for each model.

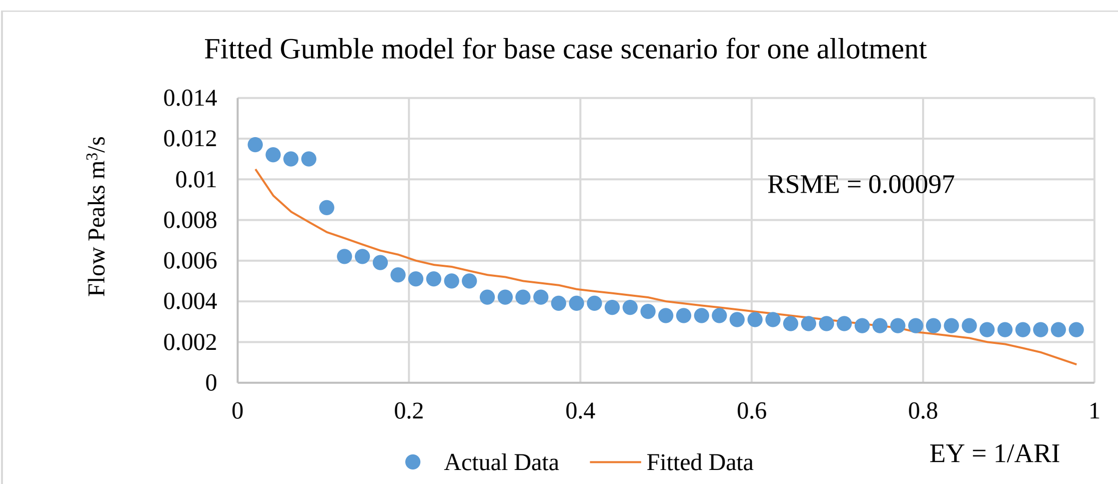
| Fitted model | RSME |
|---|---|
| Generalized Extreme Values (GEV) | 0.00084 |
| Log Normal | 0.0013 |
| 3 parameters log normal | 0.00082 |
| Log-Pearson III | 0.00068 |
| Gumble | 0.00097 |




Fitted 3 parameter log normal model for base case scenario for one allotment
0.014
0.012
0.01
0.008
0.006
0.004
0.002
0
Flow Peaks m³/s
RSME = 0.00082
0
0.2
0.4
0.6
0.8
1
Actual Data
Fitted data
EY = 1/ARI


Fitted Log-Pearson III model for base case scenario for one allotment
0.014
0.012
0.01
0.008
0.006
0.004
0.002
0
Flow Peaks m³/s
RSME = 0.00068
0.0
0.2
0.4
0.6
0.8
1.0
Actual Data
Fitted data
EY = 1/ARI


Fitted Gumble model for base case scenario for one allotment
0.014
0.012
0.01
0.008
0.006
0.004
0.002
0
Flow Peaks m³/s
RSME = 0.00097
0
0.2
0.4
0.6
0.8
1
Actual Data
Fitted Data
EY = 1/ARI

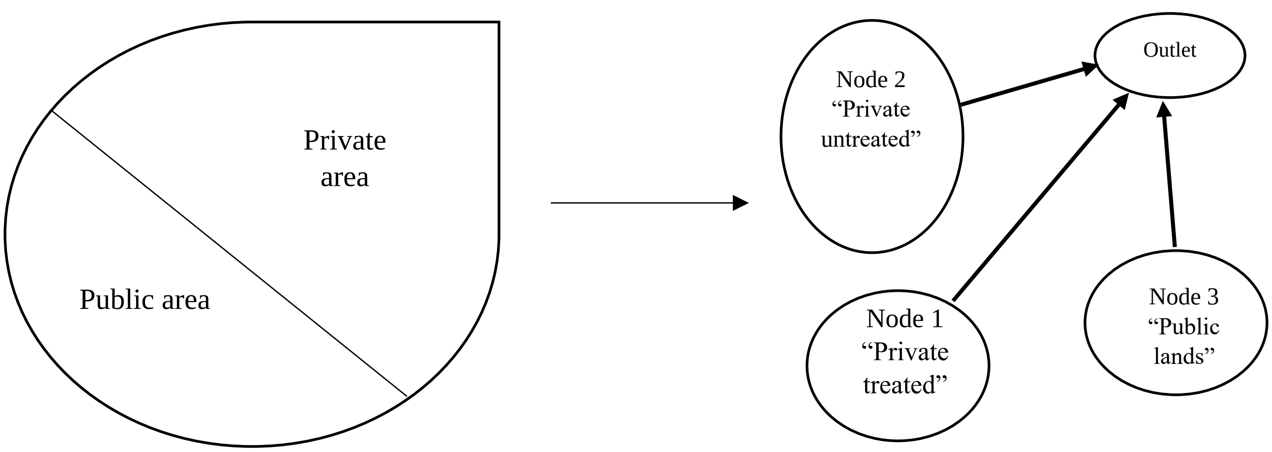
## ***Raingarden adoption in MUSIC model***

In this appendix adoption of raingardens in MUSIC model is explained. One sub-catchment was selected and steps are explained for raingardens adoption throughout the sub-catchment as an example.

1- Selected sub-catchment is C1-1. The total area of C1-1 is 27 ha. There are public areas such as parks and public roads and private areas such as property's roofs and gardens within the sub-catchment. The ratio for private and public areas was calculated as 75% and 25% respectively using following equations:
   - Private ratio %: total number of allotments * average area of allotments ha/total area ha (414*0.0495/27= 0.75)
   - Public ratio %: 1- private area ratio % (1- 0.75= 0.25)

To represent the sub-catchment, three nodes were defined as explained in the next step.

2- To represent the existing catchment condition three nods were considered for the area as following:
   - One node to represent the private area and were not connected to raingardens (Node 1)
   - One node to represent the private area and were connected to raingardens (Node 2)
   - One node to represent the public lands area (Node 3).




2-1- Node 1 was named "Private untreated" with no raingarden and the area was calculated for that according to the following equation:

Total area ha*0.75 (percentage for private area) * (1-0.28) = 14.58 ha

2-2- Node 2 was named "Private treated "with raingardens and the area was calculated for that according to the following equation:

Total area ha * 70% * 28% = 5.67 ha

The 28% was taken from ABS report 2010 and represent the percentage of households were connected to rainwater tanks in March 2010. So, the same amount was considered for adopting raingardens.

2-3- Node 3 was named “Public lands” and the area was calculated for that according to the following equation:

Total area ha *0.25 = 6.75 ha

Figure A6-1 shows the nods as defined in MUSIC model.

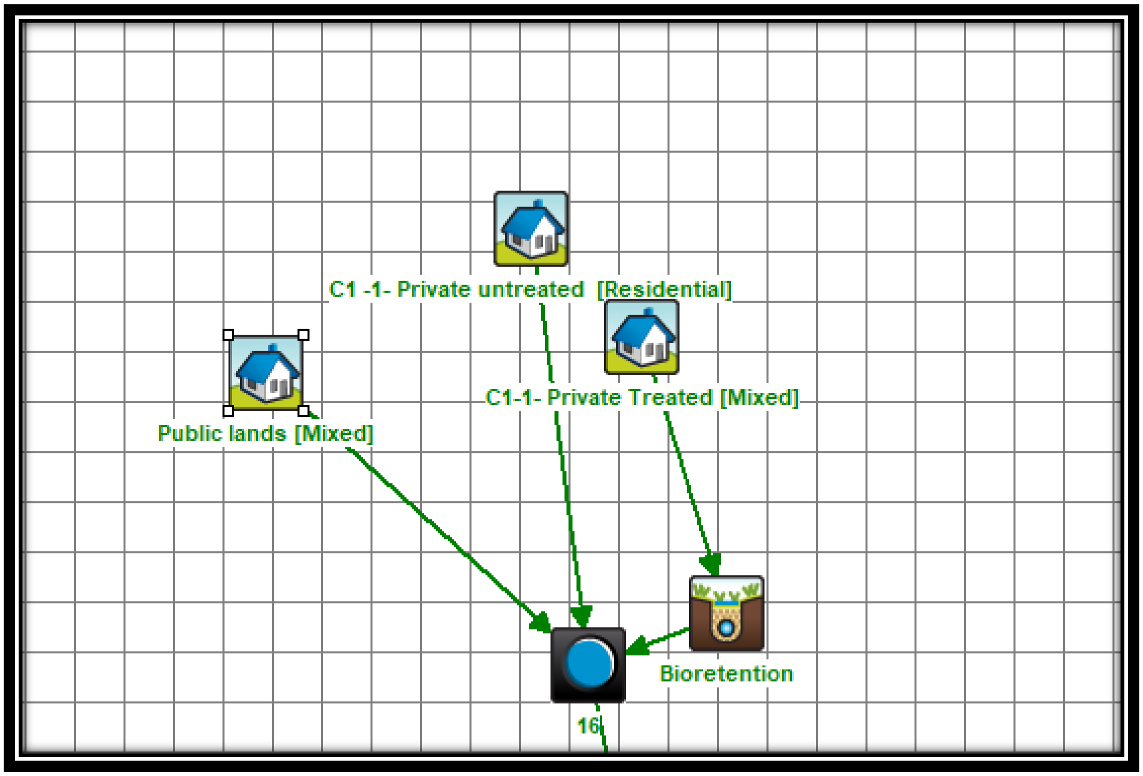



Figure A6-1. Urban nodes as defined in MUSIC model.

The Node 2 was connected to raingardens and the number of raingardens calculated using following equation:

Number of households in C1-1 * 0.28= 120

The size of raingardens was selected 5m$^2$ according to the Little Stringybark Creek stormwater treatment plant report and size of allotments in the area.

https://www.melbournewater.com.au/sites/default/files/2018-09/MWA2073A%20Little%20Stringybark%20Creek%20ESO%20Treatment%20Plan%20eForm%20Updated%2020170605_005.pdf

Following parameters were multiplied by number of raingardens for Node 3:

- Filter area
- Perimeter
- And surface area.

3- Impervious and pervious fractions within the nodes in sub-catchment

3-1- Node 1 has an impervious fraction of 65% according to the Table A6-1. The allotments are the most important components of private areas in sub-catchments and the average allotment size was calculated for the area as the table.

Table A6-1. Average allotment size and components in the study area

| **Allotment size ($m^2$)** | **Roof area ($m^2$)** | **Paving and driveway ($m^2$)** | **Garden and lawn($m^2$)** |
|---|---|---|---|
| 495 | 240 (0.49 of the total) | 80 (0.16 of the total) | 175 (0.35 of the total) |

3-2- Node 2 has an impervious fraction of 65% as Node 1.
3-3- Node 3 has an impervious fraction of 70% (MW music guideline)

https://www.melbournewater.com.au/planning-and-building/developer-guides-and-resources/guidelines-drawings-and-checklists/guidelines

4- To run the scenarios time of concentration also was calculated for C1-1 according to PCSWMM model and was assigned to drainage links.

# Appendix C

| ASSET | ASSET PARAMETERS | CONSTRUCTION[1] | MAINTENANCE | | RENEWAL |
|---|---|---|---|---|---|
| | | | ESTABLISHMENT (FIRST TWO YEARS) | ONGOING | |
| WETLANDS[2] | < 500 m²<br>500 to 10,000 m²<br>> 10,000 m² | \$150/m²<br>\$100/m²<br>\$75/m² | | \$10/m²/yr<br>\$2/m²/yr<br>\$0.5/m²/yr | No data |
| SEDIMENT BASINS[2] | < 250 m²<br>250 to 1000 m²<br>> 1000 m² | \$250/m²<br>\$200/m²<br>\$150/m² | | \$20/m²/yr<br>\$10/m²/yr<br>\$5/m²/yr | Remove and dispose of:<br>Dry waste = \$250/m³<br>Liquid waste = \$1,300/m³ |
| ON-STREET RAINGARDENS[3] | < 50 m²<br>50 to 250 m²<br>> 250 m² | \$2000/m²<br>\$1000/m²<br>\$500/m² | | \$30/m²/yr<br>\$15/m²/yr<br>\$10/m²/yr | Minor reset =<br>\$50 to \$100/m² |
| BIORETENTION BASINS[3] | < 100 m²<br>100 to 500 m²<br>> 500 m² | \$1000/m²<br>\$350/m²<br>\$250/m² | | \$5/m²/yr | No data |
| TREE PITS[3] | < 10 m² total<br>10 to 50 m² total<br>> 50 m² total | \$8000/m²<br>\$5000/m²<br>\$1000/m² | Two to five times ongoing maintenance cost | No access issues = \$150/asset/yr<br>Traffic issues or specialist equipment required = \$500/asset/yr | No data |
| GRASS SWALES AND BUFFER STRIPS[4] | Seeded – no subsoil drain<br>Seeded – subsoil drain<br>Turfed – no subsoil drain<br>Turfed – subsoil drain<br>Native grasses established | \$15/m²<br>\$25/m²<br>\$20/m²<br>\$35/m²<br>\$60/m² | | \$3/m²/yr | No data |
| VEGETATED SWALES AND BIORETENTION SWALES[4] | | 150/m² | | \$5/m²/yr | No data |
| IN-GROUND GPTS | < 300 L/s<br>300 to 2000 L/s<br>> 2000 L/s | \$50,000/asset<br>\$150,000/asset<br>\$250,000/asset | N/A | Inspection = \$100/visit<br>Cleanout = \$1000/visit | No data |

1 Includes planning and design
2 Area at normal water level
3 Area of filter media at bottom of extended detention
4 Total vegetated area

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