Using Rainwater Tanks as Stormwater Control Measures to Improve Runoff and Water Quality Management in Urban Areas

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Urbanisation and associated human activities impact rivers and streams that flow through urban areas. Impacts include receiving large volumes of stormwater runoff loaded with high concentrations of contaminants during rainfall events. Different decentralised stormwater control measures such as raingardens and rainwater harvesting tanks have been used to mitigate stormwater runoff in urban areas, and different natural materials such as mussel shell waste have been incorporated in these measures to remove contaminants in runoff near pollution sources.

Rainwater tanks have been recently recognised as stormwater control measures based on the tank's ability to detain roof runoff during rainfall events. Despite the increased attention on using rainwater tanks to mitigate stormwater runoff in urban areas, there is still a lack of knowledge regarding their effectiveness to mitigate stormwater runoff at different scales of urban areas such as industrial and residential lands. Furthermore, the use of filtration units in the rainwater tanks to improve the mitigation performance and to remove common contaminants in roof runoff such as zinc have not been investigated yet. The mussel shell wastes have been used to remove dissolved zinc from stormwater runoff in stormwater control measures. However, the removal efficiency of zinc during different filtration conditions such as varied flow rates and short contact times with water have not been fully investigated yet. Therefore, this thesis investigated the use of mussel shell waste as filtration media in rainwater tanks to remove zinc from roof runoff, and evaluated the use of rainwater tanks to mitigate stormwater runoff at residential and industrial scales in Christchurch, New Zealand.

The effectiveness of mussel shell to remove zinc was investigated for untreated mussel shell (UTMS), and heat-treated mussel shell (TMS). Two types of filtration units were designed to investigate the use of UTMS and TMS as filtration media in the rainwater tank. The first filtration units included 1.0 m depths of

the filtration media connected to a gravity-driven outlet, and the second filtration units included 0.8 m depths of the filtration media connected to a siphonic-driven outlet. An actual roof runoff was collected from galvanised roofing, and the removal performance of zinc was estimated during controlled saturated flow rates of 1, 3, 5, 10 L/min.

The collected roof runoff showed high concentrations of dissolved zinc with an average zinc concentration of 3347.2 µg/L, which is ca. 200 times higher than the recommended concentration of zinc in urban streams (i.e. 15 µg/L) to protect 90% of the freshwater organisms according to the ANZECC’s guidelines (ANZECC, 2000). Both the TMS and UTMS demonstrated high removal efficiencies of dissolved zinc. The heat treatment of the mussel shell generally improved the removal performance of dissolved zinc. The TMS media showed significantly higher ($p \leq 0.05$) removal performance of zinc compared to the UTMS media for the tested flow rates during 0.8 m depths of filtration media, while the TMS showed higher average removal efficiencies but with there was no significant difference ($p > 0.05$) for the tested flow rates during the 1.0 depths. For all flow rates, the overall average removal efficiencies of the TMS were estimated at 94% and 82% for 1.0 m and 0.8 m depths of filtration media respectively, while the overall average removal efficiencies of the UTMS were estimated at 92% and 72% for 1.0 m and 0.8 m depths of filtration media respectively. The removal performance of zinc decreased as water flow rates through the TMS and UTMS increased.

The EPA's Storm Water Management Model (SWMM) was used to evaluate the mitigation performance of rainwater tanks for two urban blocks that represented the residential and industrial land use in Christchurch, New Zealand. Three management scenarios were simulated using 5-min time steps throughout a 12-year period starting from 4 November 2007 until 4 November 2019. The first management scenario represented the Business As Usual (BAU) conditions, and used as the reference of actual stormwater peaks and total volumes of outflow in the selected blocks. The second management scenario represents using Rainwater Harvesting for Toilet Flushing (RWH-TF) uses which simulated the use of collected rainwater to supply water for toilet flushing uses in the existing buildings of the selected blocks. The third management scenario represents using Rainwater Harvesting for Stormwater Treatment (RWH-ST) uses which simulated the use of rainwater tanks and filtration units as stormwater detention units to collect, and slowly discharge treated roof runoff into the stormwater network.

The simulation results showed effective mitigation performance in both the residential and industrial blocks. The average reductions in peak runoff during the RWH-ST scenarios were estimated at 52.9% and 45% in the residential and industrial blocks respectively, while the average reductions in runoff volumes

were estimated for 37.3% and 19.5%. The RWH-TF scenarios showed lower peak reductions with averages estimated at 36.3% and 23.9% in the residential and industrial blocks respectively, while the average volume reductions estimated at 42.9% and 27.5% in the residential and industrial blocks respectively. During RWH-ST scenario, the integrated rainwater tanks with filtration units showed effective treatment performance throughout the simulation period with more than 99% and 58% treatment ratio of roof runoff in the simulated residential and industrial blocks respectively.

The results of this thesis exposed potential benefits for using integrated rainwater harvesting tanks (i.e. with filtration units) as stormwater control solutions to improve runoff and water quality management in urban areas. In particular, the use of mussel shell waste as filtration media in rainwater tanks provided cost-effective solutions to remove metals from runoff in order to protect the ecological conditions in urban waterways, and the use of rainwater tanks reduced stormwater runoff volumes and peak flows during rainfall events at both residential and industrial scales.

Keywords: Rainwater harvesting, Sotrmwater Management, Low Impact Development, Hydrologic Modelling.

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Terminology

Abbreviations and Acronyms

ANZECC	Australian and New Zealand Environmental and Conservation Council
ARI	Annual Recurrence Interval
$CaCO_3$	Calcium carbonate
EDS	Energy-dispersive x-ray spectroscopy
GIS	Geographic Information System
IBC	Intermediate Bulk Container
ICP-MS	Inductively coupled plasma mass spectrometry
LID	Low Impact Development
PAH	Polycyclic aromatic hydrocarbon
PCSWMM	Personal Computer Storm Water Management Model
PVC	Polymerizing Vinyl Chloride
RWH	Rainwater Harvesting
RWH-ST	Rainwater Harvesting for Stormwater Treatment
RWH-TF	Rainwater Harvesting for Toilet Flushing
SEM	Scanning electron microscope
SWMM	Storm Water Management Model
TMS	Treated Mussel Shell
UTMS	Untreated Mussel Shell
XRD	X-Ray Diffraction

Chapter 1: Introduction

1.1 Overview

Urbanisation and associated human activities impact urban streams and rivers. Impacts generally appear in forms of consistent poor water quality, and sudden changes in flow patterns with larger volumes and flashier peak flows during rainfall events (Junior et al., 2010). Such impacts occur primarily due to the change in natural land cover in urban catchments which affect the hydrological responses of associated waterways and create several pollution sources that leach contaminants into runoff during rainfall events (Walsh, 2000).

The degradation of water quality in urban waterways has been linked to several types of contaminants in stormwater runoff such as heavy metals that drain with runoff into urban waterway during rainfall events (Hvitved-Jacobsen et al., 2010). Heavy metals such as zinc, copper, and lead pose significant threats to the freshwater organisms due to their toxic effects on the freshwater organisms (Ayangbenro et al., 2017). Furthermore, such metals in their dissolved forms have the ability to bio-accumulate in the food chains causing detrimental health concerns and long-term impacts on the associated ecosystems (Singh et al., 2011). As a result of these threats, metals such as zinc, has been classified among the contaminants of most concern in rivers and streams that flow through urban areas in Christchurch, New Zealand (Margetts & Marshall, 2018) as its concentrations consistently exceeded the recommended guidelines for aquatic life protection as per the Australian and New Zealand Environment and Conservation Council standards (ANZECC, 2000).

Previous studies recognised stormwater runoff as a significant source of loading zinc into urban waterways during rainfall events (Charters et al., 2016). Before reaching stormwater runoff, zinc is initially generated from different impervious surfaces in urban areas such as roads, galvanised roofing and other miscellaneous construction sites. Among these sources, galvanised roofing has been recognised as a significant source of leaching zinc to stormwater runoff (Charters et al., 2016) where high concentrations of zinc can be generated from existing galvanised roofing and can be boosted by factors such as corrosion, wear and tear, and low pH levels of roof runoff (Wicke et al., 2014). Furthermore, zinc generated from galvanised roofing generally present in its dissolved form (i.e. bioavailable) which makes it more likely to impact the aquatic taxa and harder to remove by conventional stormwater control measures (Charters et al., 2017). Thus, there has been increased attention to the importance of developing decentralised stormwater control measures such as raingardens and other treatment measures to remove zinc in runoff near to the pollution sources before reaching the stormwater network.

Recent control measures have incorporated the use of mussel shell waste in different applications of stormwater treatment. Applications include removing zinc, copper, lead, cadmium, phosphate, and other contaminants in water (Currie et al., 2007). Mussel shell waste is readily available as a discarded by-product from marine-fish industries in New Zealand. Previous research conducted by Craggs et al. (2010) showed potential opportunities for using untreated crushed mussel shell to remove dissolved zinc from contaminated stormwater runoff; however, other study suggested further treatment methods such as heat treatment to prepare the mussel shell in order to improve the shell performance in different industrial applications (Mo et al., 2018). However, the performance of heat-treated mussel shell to remove dissolved zinc from water has not been explored in previous studies. Furthermore, the use of crushed mussel shell as a filtration material in stormwater control measures requires effective removal performance during different filtration conditions such as varied flow rates and contact times with water, which also has not been fully investigated yet.

On the other hand, impervious surfaces have also increased stormwater runoff volumes in urban areas which resulted in frequent excess flow and inundations in the stormwater network components with greater risk of flooding around urban streams and rivers (Walsh et al., 2012). Different stormwater control measures have been developed to mitigate stormwater runoff in urban areas. Measures range from conventional drainage networks to convey stormwater runoff with large scale detention basins, to more emerging low impact decentralised measures to mitigate stormwater runoff throughout urban areas (Council, 2009). Decentralised stormwater measures include rainwater harvesting tanks, raingardens, green roofing, and permeable pavements (Dietz, 2007). Such low impact decentralised measures mitigate stormwater runoff near to the source and provide several benefits such as water conservation and urban greening. Among these measures, rainwater harvesting (RWH) is the most ancient method in use to maintain water supply for human needs (Campisano et al., 2017) and rainwater tanks have been recently recognised as stormwater control measures to mitigate stormwater runoff volumes in urban areas (Palla et al., 2017). This is based on the tank performance to detain roof runoff, which alleviates the pressure on the existing stormwater network during rainfall events.

Despite the increased attention for using rainwater tanks to mitigate stormwater runoff in urban areas, there is still a lack of knowledge regarding the tank effectiveness to mitigate stormwater runoff volumes and peak flows at different scales of urban areas. This include the mitigation performance in residential and industrial areas and during different uses of the collected rainwater. The uses of collected rainwater have direct impacts on the mitigation performance of rainwater tanks, in which better mitigation performance can be achieved with higher usage and demand on the collected rainwater (Palla et al., 2017). All previous assessments for using rainwater tanks to mitigate stormwater runoff have limited the demand on rainwater for toilet flushing uses only, or combine it with occasional non-potable uses such as laundry demand (Petit-Boix et al., 2018). However, the mitigation performance of rainwater tanks has not been investigated for using the integrated rainwater tanks as stormwater detention units to collect, treat, and slowly discharge treated runoff into the stormwater network.

Therefore, the first phase of this thesis investigated the potential use of heat-treated crushed mussel shell and untreated crushed mussel shell as filtration media in the RWH tank to remove zinc in roof runoff during different filtration conditions. The second phase of this thesis evaluated the potential use of RWH tanks to mitigate stormwater runoff at residential and industrial scales in Christchurch, New Zealand. This include using GIS-SWMM modelling to evaluate the mitigation performance of rainwater tanks during different management scenario and uses of the collected roof runoff.

1.2 Research Objectives

The main two aims of this thesis were to investigate the potential use of TMS and UTMS as filtration media in the RWH tanks to remove zinc from roof runoff, and to evaluate the potential use of RWH tanks to mitigate stormwater runoff volumes and peak flows at residential and industrial urban scales.

With these aims, the thesis was carried out according to the following main objectives:

1. To design the filtration unit, and assess the effectiveness of using the following outlets to drain treated water in the filtration unit:
    - Gravity-driven outlet;
    - Siphonic-driven outlet.
2. To compare the effectiveness of using TMS vs UTMS to remove zinc from roof runoff in rainwater tanks. This included measuring the removal efficiency of zinc during several flow rates (1, 3, 5, 10 L/min) to represent different actual operation conditions by using:
    - (1.0 m depth of TMS) vs (1.0 m depth of UTMS) in a standard pipe with diameter of 100 mm that connected to a gravity-driven outlet;
    - (0.8 m depth of TMS) vs (0.8 m depth of UTMS) in a standard pipe with diameter of 100 mm that connected to a siphonic-driven outlet.
3. To evaluate the rainwater tank performance to mitigate stormwater runoff volumes and peak flows at residential and industrial scales in Christchurch. This included using GIS-SWMM modelling to simulate the performance during the following two scenarios:
    - The use of rainwater tanks to supply water for toilet flushing uses;
    - The use of rainwater tanks as stormwater detention units (collect, treat and slowly discharge roof runoff).

1.3 Thesis Structure

The thesis is structured in the following order:

- Chapter one presents an overall introduction regarding the need for this research along with the objectives and the structure of this thesis;
- Chapter two presents a literature review of previous work related to stormwater management, the recent uses of RWH tanks around the globe, and the commonly used filtration media in stormwater measures to provide a context for this research;
- Chapter three presents the laboratory investigation regarding the use of TMS and UTMS as filtration media in rainwater tanks. This include the methodology used to perform the experiments, the results, and discussion of the findings of these experiments;
- Chapter four presents the GIS-SWMM modelling regarding the use of RWH tanks to mitigate stormwater runoff at residential and industrial scales in Christchurch, New Zealand. This include the modelling methodology, the results, and discussion of the findings of these models;
- Chapter five presents conclusions and recommendations for future research areas derived from the findings and results of this thesis.

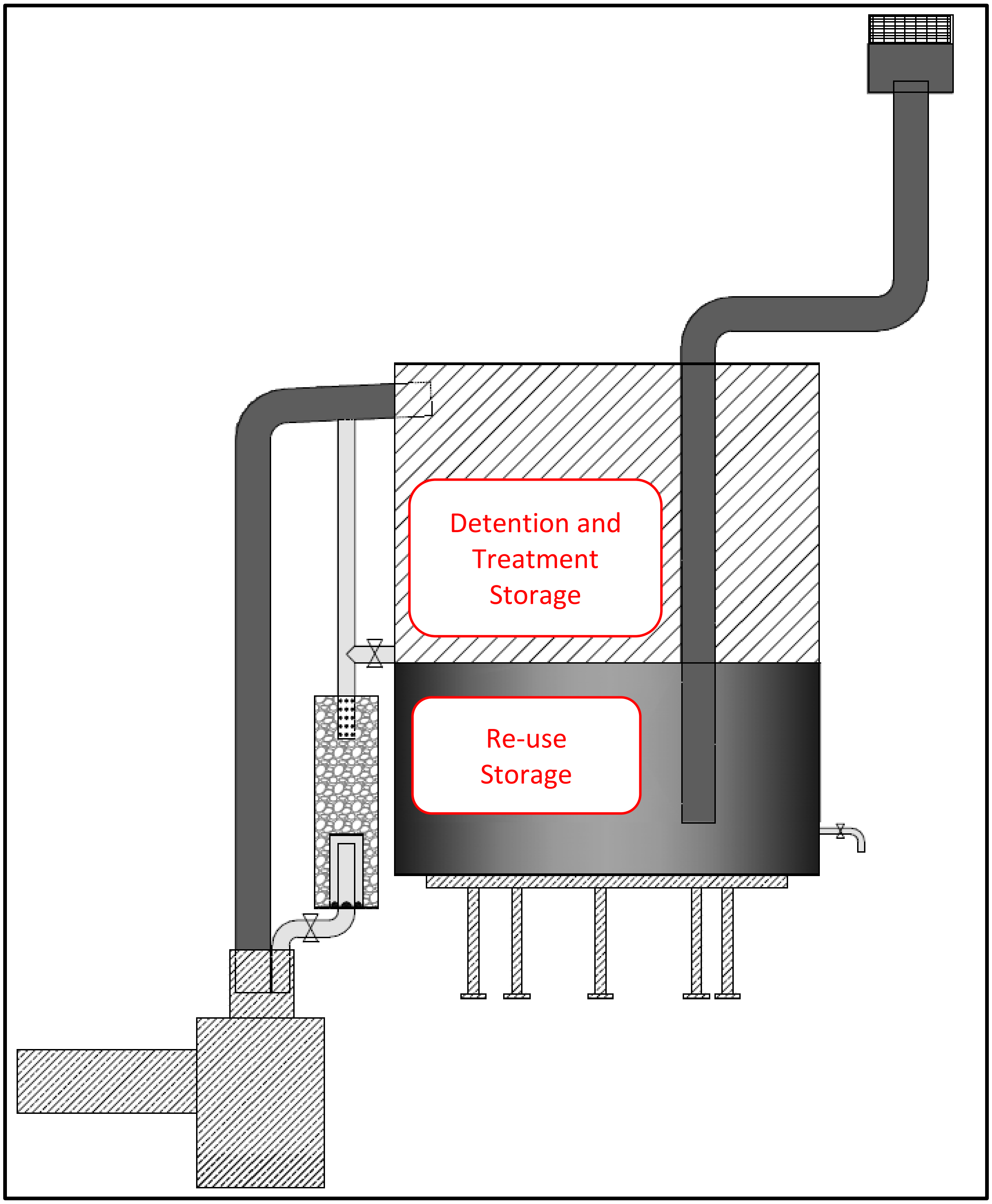


Figure 1.1: The proposed integrated rainwater tank in this study which support the benefits of water supply, roof runoff treatment, and stormwater runoff mitigation.

Chapter 2: Literature Review

This chapter presents a review of previous studies related to stormwater management and recent uses of RWH tanks around the world. This includes previous studies that evaluated the performance of rainwater tanks to mitigate stormwater runoff, and studies related to the quality perspective of the harvested rainwater. Furthermore, this chapter includes a review of the commonly used materials to remove contaminants from stormwater runoff, along with the configurations and parameters required to design effective filtration units.

2.1 Urbanisation and Stormwater Management

The continuous acceleration in urbanisation and associated human activities in urban catchments impact rivers and streams that flow through urban areas. Impacts include changes in flow patterns with larger runoff volumes and flashier peak flows during rainfall events (Palla et al., 2017). Furthermore, the elevated concentration of contaminants in runoff resulted in consistent poor water quality in urban waterways (Zhang et al., 2017).

Stormwater runoff has been recognised as a significant source of loading contaminants to urban streams and rivers (Davis et al., 2001). Different stormwater control measures have been historically used to manage stormwater runoff in urban cities, were generally classified under two main categories; conventional and decentralised control measures (Zhang et al., 2017). Conventional stormwater measures have been primarily developed to support the main objective of preventing urban flooding (Zhang et al., 2017). Conventional measures generally consist of distributed drainage infrastructure and pipeline networks to convey surface runoff generated from impervious surfaces into the surrounded waterways (Debo et al., 2002). Besides conventional pipeline, drain, and tunnel networks, conventional measures may also include large-scale detention basins and urban wetlands that work to reduce the stormwater discharge volumes and improve the quality of stormwater runoff.

On the other hand, decentralised concepts of stormwater management with environmentally friendly control measures have been introduced to support sustainable development in urban areas (Winz et al., 2011). For instance, Low Impact Development (LID) is one of the decentralised concepts for managing stormwater in urban areas. Different LID measures have been used in

North America and New Zealand since the early 1970s to alleviate pressure on conventional stormwater measures, and to support various benefits such as water conservation, urban greening, improving the quality of stormwater runoff (Fletcher et al., 2015). LID measures include rainwater harvesting tanks, permeable pavements, green roofs, raingardens and vegetative swales (Council, 2009). The main aim of such measures is to minimise the impacts of urbanisation on urban waterways (Zhang et al., 2017). The associated benefits include urban greening and water conservation. The small-scale applications of such measures have accelerated the implementation process into existing buildings in urban areas (Zhang et al., 2017).

2.2 Rainwater Harvesting

Collecting rainwater is the most ancient way in use to maintain the human needs of water supply (Campisano et al., 2017). Driven by climate change, and supported by new filtration technologies, rainwater harvesting (RWH) tanks are being used to secure an independent source of water supply in several places around the globe. At the same time, a recent assessment showed that using RWH tanks in urban areas could mitigate stormwater runoff and alleviate pressure on the existing stormwater network during rainfall events (Petit-Boix et al., 2018).

2.2.1 Modern Areas of Use of Rainwater Harvesting

The installation of RWH tanks has been driven by different purposes and objectives around the world. For instance, in Japan, the Great Eastern Earthquake (magnitude of 9.1, March 2011) caused sudden rises in the number of houses with installed RWH tanks to collect rainwater primarily for emergency uses. Before the earthquake, RWH tanks were mainly installed in large-scale commercial buildings and schools primarily to minimise urban flood risks, and to fill shortage gaps in the water supply (Campisano et al. 2017).

In the United Kingdom, RWH tanks in residential areas have been typically installed to maintain non-potable water uses such as toilet flushing, laundry, and house gardening uses. Larger RWH tanks can be found in public buildings, supermarkets and schools to support potable water supply due to the associated economic viability (Ward et al., 2012).

Australia is one of the most active countries in the installation of RWH tanks at residential scales (Eroksuz et al., 2010). By the end of 2013, one in three Australian households had installed RWH tanks mainly to support potable water supply in rural areas (Campisano et al., 2017).

Germany is also a frontrunner in the installation of RWH tanks at the residential scale with 33% installation ratio for newly constructed houses. This is mainly due to funds provided by the local authorities to support the growth of green infrastructure in Germany (Schuetze, 2013). Most of the RWH tanks have been used primarily to support non-potable uses such as toilet flushing, laundry uses, gardening, and car washing, due to serious air pollution in Germany (Campisano et al., 2017).

In the United States, Texas has the highest installation ratio of RWH tanks. Generally, RWH tanks have been used to fill the gap between water supply and demand, along with other non-potable uses such as firefighting, toilet flushing and gardening (Thomas et al., 2014).

In New Zealand, rainwater has spiritual values to the Maori (i.e. indigenous Polynesian people of New Zealand) and is considered as pure water to drink (Abbott et al., 2007). More than 10% of New Zealanders are using RWH tanks to secure their non-potable water demands, particularly in rural areas without municipal water supply (Abbott et al., 2007). Some councils in New Zealand have regulated the installation of RWH tanks in new private residential developments as stormwater control measures to mitigate runoff and to reduce the environmental impacts of these developments (Gabe et al., 2012).

2.3 Previous Assessment for Rainwater Tanks Performance

Several previous studies have attempted to evaluate the performance of RWH tanks to mitigate stormwater runoff volumes and peak flows (Campisano, et al., 2017). These studies have formed somewhat contradictory conclusions regarding the actual mitigation performance, and highlighted different design parameters to improve the tank performance. According to Burns et al. (2015), the mitigation performance depends mainly on the storage capacity of the tanks and the demand on the collected rainwater. Similar results have been revealed by Petit-Boix et al. (2018), which indicated that the balance between water demand, storage capacity, and rainwater availability is essential to achieve effective mitigation performance. However, the use of collected rainwater varies according to different financial, social and environmental aspects (Campisano, et al., 2017). Previous studies that assessed the mitigation performance have mainly restricted the use of collected rainwater to supply water for toilet flushing uses (Palla et al., 2017) or combined it with occasional non-potable uses such as laundry demand (Petit-Boix et al., 2018). Furthermore, these assessments have investigated the mitigation performance based on site-related conditions such as land uses and rainfall intensity climates.

For instance, a study by Petrucci et al. (2012) evaluated the tank mitigation performance for a residential block located in France, and found that effective reductions in runoff volumes and peak flows were only noticeable for light rainfall events. A similar study conducted by Burns et al. (2015) for a pre-urban block located in the south-west of Australia showed effective reductions in runoff volumes during light rainfall events for the existing outdoor water demand only. Such results were obtained based on the specific site-related conditions such as water demand and the existing storage capacities.

Similarly, previous models have assessed tank mitigation performance and produced results that represent the assumed water demands and the site-related conditions in the selected locations (Palla et al., 2017). All previous assessments have used the EPA's Storm Water Management Model (SWMM) to assess the mitigation performance of tanks, as SWMM is commonly used for the design, analysis and planning of stormwater runoff around the world.

Several previous SWMM models were carried out by a mean of simplified assumptions (Palla et al., 2017). For instance, a study conducted by Petrucci et al. (2012) used SWMM without including the RWH tanks in the model; instead, equivalent initial losses were assigned to replace the storage capacity of the tanks before the start of the outflow in the model. Other models introduced by Huang et al. (2015), and Walsh et al. (2014) simulated the use of RWH tanks by using the Rain Barrel node provided in SWMM 5.0 to estimate the water volumes and outflow rates in the tanks; however, the Rain Barrel nodes have been recently recognised as an inaccurate tool to assess the actual mitigation performance of RWH tanks (Campisano, et al., 2017). This is due to the fact that Rain Barrel nodes do not support separate calculations for the yield water volume in the tank, and the overflow rate out of the tank, which is required to obtain accurate calculations of the mitigation performance.

Another SWMM model introduced by Palla et al. (2017) assessed the reductions in stormwater runoff volumes in a residential urban block located in Italy with an assumed constant daily water demand on the collected rainwater per inhabitants for toilet flushing uses only. A recent study introduced by Petit-Boix et al. (2018) simulated the use of RWH tanks for two residential neighbourhoods in the US and Spain by using PCSWMM 5.1007 with an assumption of constant demand on the collected water within all neighbourhood buildings. Results revealed that the RWH tanks had the potential to reduce the total stormwater runoff by 35 to 47% percent considering several factors such as rooftop areas, and the assumed demand on the collected water. Thus, further investigation is required to evaluate the mitigation performance in different urban land uses and rainfall intensity climates.

In regard of the required time steps to analyse the tank performance to mitigate runoff volumes and peak flows in SWMM, a dimensionless model was introduced by Campisano et al. (2012) indicated that at least hourly time steps were required to accurately measure the reduction in runoff volumes, and sub-hourly time steps (i.e. minutes) were mandatory for accurate peak reduction calculations.

2.4 Zinc in Roof Runoff

In general, roof runoff has better water quality comparing to other runoff from surfaces such as parking lots and sidewalks (Hamdan, 2009). However, recent studies revealed that roof runoff carries substantial loads of contaminants such as heavy metals into urban streams and rivers (Meera et al., 2006). Heavy metals such as zinc cause detrimental environmental impacts on the receiving waterways due to its toxic effects on freshwater organisms even at low concentrations (Sabin et al., 2005). Thus, due to its consistently high concentration in urban streams, zinc has been classified amongst the water contaminants of most concern in Christchurch (Margetts & Marshall, 2018).

Galvanised roofing exposed to rainwater can leach high concentrations of dissolved zinc to urban waterways (Charters et al., 2017). According to Milne et al. (2008), roof runoff was the main source of zinc loading to the waterways in Wellington, New Zealand, and Moores et al. (2009) revealed similar results for the urban streams in Christchurch. Charters et al. (2016) revealed that zinc has been found mainly in its dissolved form in the roof runoff which makes it harder to remove by conventional treatment measures, and more likely to have potential impacts on aquatic taxa. Thus, due to the high concentrations of zinc in roof runoff, integrating filtration units with RWH tanks to remove dissolved zinc provides a potential opportunity to treat runoff near pollution sources. Pollution sources such as large industrial and commercial buildings with galvanised roofing leach substantially high concentrations of dissolved zinc to stormwater runoff during rainfall events.

2.5 Filtration Materials to Remove Zinc.

Different natural materials have been used to remove zinc and other metals from runoff (Hipp et al., 2006). Sand and gravel have been commonly used to treat stormwater from particulate contaminants such as debris and other suspended solids (Paul et al., 2015). However, due to their limited adsorption capacity and surface areas, their effectiveness to remove dissolved metals found to be relatively low (Genç-Fuhrman et al., 2007). Other natural materials, such as crushed oyster shells, red mud, zeolite, and limestone have been previously used to remove dissolved metals from stormwater runoff (Shin et al., 2014). Calcium carbonate ($CaCO_3$) is the main constituent of different filtration materials that have been used previously such as limestone, oyster shells, and mussel shells. Previous studies revealed that $CaCO_3$ has the ability to remove dissolved zinc from water by adsorption and precipitation (Clark et al., 2004; Sdiri et al., 2012) and previous laboratory evaluation showed high removal performance of zinc by using crushed mussel shell (Craggs et al., 2010). Mussel shell waste is readily available in New Zealand as industrial by-products for the marine-fish industries, and using such waste material as filtration media in the rainwater tanks would provide cost-effective solutions to remove zinc from roof runoff near to the pollution sources.

2.5.1 Crushed Mussel Shell

Crushed mussel shells have been already incorporated in several applications of water treatment. Applications include removing phosphate, lead, cadmium, copper and zinc (Currie et al., 2007).

Recent research suggested a number of methods to prepare the mussel shell before using them in different applications. These methods include washing, drying, heating, and crushing the shell into different practice sizes (Mo et al., 2018). These treatment methods have been introduced to improve the shell's performance in different industrial applications. For instance, a study by Currie et al. (2007) revealed that the heat treatment of the shell improved the removal efficiency of phosphorus from contaminated water.

Heat treatment is primarily performed to remove the attached organic and protein matters in the shell (Ballester et al., 2007). Several heat-treatment approaches have been introduced to treat the shell for different industrial applications. Djobo et al. (2016) showed that heating the shell at a temperature of 500°C for 2 hours can remove all organic contents. Other study showed that heating the shell at temperatures higher than 850°C will result in a reduction in $CaCo_3$ content and increase the calcium oxide (CaO) content (Chiou et al., 2014). A standard treatment method has been suggested by (Barros et al., 2009) which include drying the shell at 190°C for 18 min and then heating for 15 min at 500°C. Although these treatment procedures have been suggested to treat mussel shell for different industrial applications other than water treatment, similar heat treatment methods could improve the removal efficiency of zinc in water treatment applications.

2.5.2 Removal Performance of Mussel Shell

A previous study showed effective removal performance of dissolved zinc from stormwater runoff by using mussel shell waster (Craggs et al., 2010). Nevertheless, several factors affect the removal performance of zinc, including pH level, contact time between water and the shell, and the particle sizes of shells. The study demonstrated the use of mussel shell to remove zinc from runoff by using different particle sizes of shells with different contact times with water. The use of mussel shell as filtration media in the RWH tank would have relatively short contact times with water during different flow rates, which has not been fully investigated yet. In actual filtration units, the influence of contact times with filtration media can be investigated by measuring the removal performance of zinc during different flow rates of water with fixed depths of filtration media, or by measuring the removal performance of zinc for fixed flow rates of water with different depths of filtration media.

2.6 Design Parameters of Filtration Units

The hydraulic conductivity and the contact times with water are essential parameters to design and achieve effective removal performance of zinc in the filtration units. The hydraulic conductivity is defined as the rate at which water passes through the filtration media (Sanford et al., 1995). The hydraulic conductivity depends mainly on the particle sizes of the filtration media. It is essential to use the optimum particle sizes as this affects both the contact time and the removal performance of zinc. Larger particle sizes provide higher flow rates with smaller surface areas which consequently results in lower removal performance while, on the other hand, finer particles provide larger surface areas with lower flow rates. Finer particles would reduce the flow rate through the filtration media, which increases the risk of clogging. Clogging along with lower flow rate can minimise the removal performance over time along with other adverse aesthetic and drainage consequences (Le Coustumer et al., 2012). A study conducted by Craggs et al. (2010) showed high potential to remove zinc in stormwater water by using different particle sizes, mainly fine particles with ranges of 0.5-2.0 mm; however, the removal performance for using wider particle size ranges and different contact times with water has not been fully investigated yet.

2.6.1 Drainage in Filtration Units

In general, conventional filtration units operate under gravity force, where gravity acts as the main driving force of water to drain through filtration media. A gravity drainage system includes an outlet placed beneath filtration media and designed to discharge water under atmospheric pressure through at the lowest point of the unit (Arthur et al., 2001). In such gravity drainage systems, a continuous air-core flow is also drawn into the system during water discharge. The air-core flow substantially reduces the drainage capacity, and makes it less effective at discharging large volumes of water during heavy flow conditions. The air volume in a typical gravity drainage system can fill up to 70% of drainage volume which limits water from entering the system (Lucke et al., 2015). Thus, larger pipe diameters and additional outlets may be required to handle the associated airflow in such gravity drainage systems. Furthermore, gravity flow systems are subjected to more frequent clogging issues near the outlets and through the system components. Even partially blocked pathways can reduce the flow capacity, and cause more frequent maintenance needs along with frequent overflows, and leaks problems.

Emerging concepts for siphonic drainage systems have been introduced to eliminate typical drainage challenges. The basic theory of creating siphonic drainage is to minimise the airflow and to operate the system with full-bore of water under a pressure lower than atmospheric pressure (Arthur et al., 2001). Negative pressure in a full-bore water system can regulate flow patterns and produce higher water discharge rates (Lucke et al., 2015). Similar recent siphonic roof drainage concepts have been introduced to solve such drainage challenges. A roof siphonic drainage system generally consists of an outlet that is designed to cut off air vortex flow from entering the connected system. The lack of air inflow combined with the gravity force in a full-bore water system creates negative pressure that activates siphonic action, which results in regulated flow through the system components (Wright et al., 2006). Such siphonic flow systems showed effective self-cleaning actions due to the continuous change in flow rates and pressures, which remove suspended particulates throughout the system components particularly near the outlets (Arthur et al., 2001). Thus, the use of a similar concept of siphonic outlet could regulate water flow near the outlet and reduces air bubbles formation inside filtration units. The available literature regarding the use of siphonic flow in water treatment and the use of integrating filtration units in the rainwater tank is very scarce. Therefore, this research investigated the potential use of similar siphonic outlets in the filtration units, and to evaluate the removal performance and the associated operational conditions of the filtration unit.

2.7 Summary of Literature Review

The ability of rainwater tanks to mitigate stormwater runoff volumes and peak flows and to alleviate pressure on stormwater network has been demonstrated through various modelling studies (Campisano, et al., 2017; Petit-Boix et al., 2018; Palla et al., 2017). However, there is still a gap in knowledge regarding the tank performance at different scales of urban areas with different rainfall intensity climates. In particular, the tank performance to delay stormwater runoff volumes and peak flows during rainfall events at residential and industrial scales in low rainfall intensity climate such as in Christchurch have not been investigated yet. The use of collected rainwater plays an essential role in improving the mitigation performance of runoff, and previous studies have only limited the use of collected roof runoff to supplying water for non-potable demands only. However, using different scenarios to manage the collected roof runoff could improve the mitigation performance of the RWH tanks. Management scenarios include using rainwater tanks as stormwater detention units to collect, and slowly discharge the collected runoff through restricted flow conditions (i.e. filtration units, valves, etc.) would result in effective mitigation of stormwater runoff discharge during rainfall events. Such knowledge would support future design and planning of stormwater management in urban areas. Furthermore, the feasibility of integrating filtration units in rainwater tanks to improve the quality of runoff by removing common contaminants from roof runoff near pollution sources has not been fully explored yet.

Previous studies demonstrated the ability of mussel shells to remove high concentrations of dissolved metals from stormwater runoff (Craggs et al., 2010 ; Gregorie N., 2018; Bremner C. et al., 2020). However, the use of mussel shells as filtration media the rainwater harvesting tank to remove metals and improve the quality of runoff near pollution sources have not been investigated yet. Furthermore, the removal performance of metal during different flow rates representative of actual operating conditions have not been fully explored yet. Additional treatment methods to prepare mussel shells could improve the removal performance of metals from water. Treatment methods include heating, crushing, sieving mussel shell could result in effective removal performance of dissolved metals.

Chapter 3: Laboratory Experiments and Evaluation

3.1 Introduction

This chapter presents the laboratory investigation regarding the use of crushed mussel shell as filtration media in the rainwater tanks to remove dissolved zinc and improve the quality of runoff in order to reduce ecological impacts to receiving waterways. This includes the methodology used to perform laboratory experiments, the associated results, and discussion of the findings in this chapter.

Two main stages of laboratory experiments were performed to assess the removal performance of zinc by using the TMS and UTMS as filtration media in the rainwater tank. The first stage experiments included using 1.0 m depths of TMS and UTMS in standard PVC pipes of 100 mm diameter connected to a gravity-driven outlet at the bottom of each filtration unit. The second stage experiments included using 0.8 m depths of TMS and UTMS in standard PVC pipes of 100 mm diameter that connected to a siphonic-driven outlet at the bottom of each filtration unit. All filtration units were filled with similar densities of filtration media.

Actual roof runoff was collected from a galvanised roof at the Soil and Water Engineering Laboratory at Lincoln University. The removal performance of zinc for the TMS and UTMS were assessed by using similar controlled saturated flow rates of 1, 3, 5, 10 L/min. The saturated hydraulic conductivity of the TMS and UTMD for the selected particle sizes were calculated.

3.2 Methodology

3.2.1 Preparation of Mussel Shell

Partially crushed mussel shell waste were purchased from a local retailer for landscape supplies (Pearson LTD). The purchased mussel shell were crushed then sieved to the selected particle size of >1 mm and < 12.7 mm to form the UTMS (Figure 3.1). The UTMS media was then filled in the filtration units and connected to the water tank. An additional step of heat treatment was performed to prepare the TMS. The mussel shell were heated at 500 C° for 15 min and left to cool at room temperature and then re-sieved to remove the ashes and fine particles that resulted from heat treatment (Figure 3.2). The TMS media was then filled in the filtration units and connected to the water tank.

Figure 3.1: The used particle size of for the TMS and UTMS (>1.0 mm and <12.7mm).

Figure 3.2: The heat treatment of crushed mussel shell: Before - In the oven - After.

3.2.2 Roof Runoff Collection

Actual roof runoff was collected from the galvanised roofing extension at the Soil and Water Engineering Laboratory at Lincoln University (Figure 3.3). The site was selected to collect roof runoff in order to represent the use of galvanised roofing in urban areas. Thus, a downpipe was diverted into a 1.0 m^3 tank to collect roof runoff from several rainfall events between July and October 2019. A forklift was used to transfer the filled tank inside the lab, and a peristaltic pump was used to pump the collected runoff into a higher level transparent 1.0 m^3 tank (i.e. Intermediate Bulk Container "IBC") in order to run the experiments.

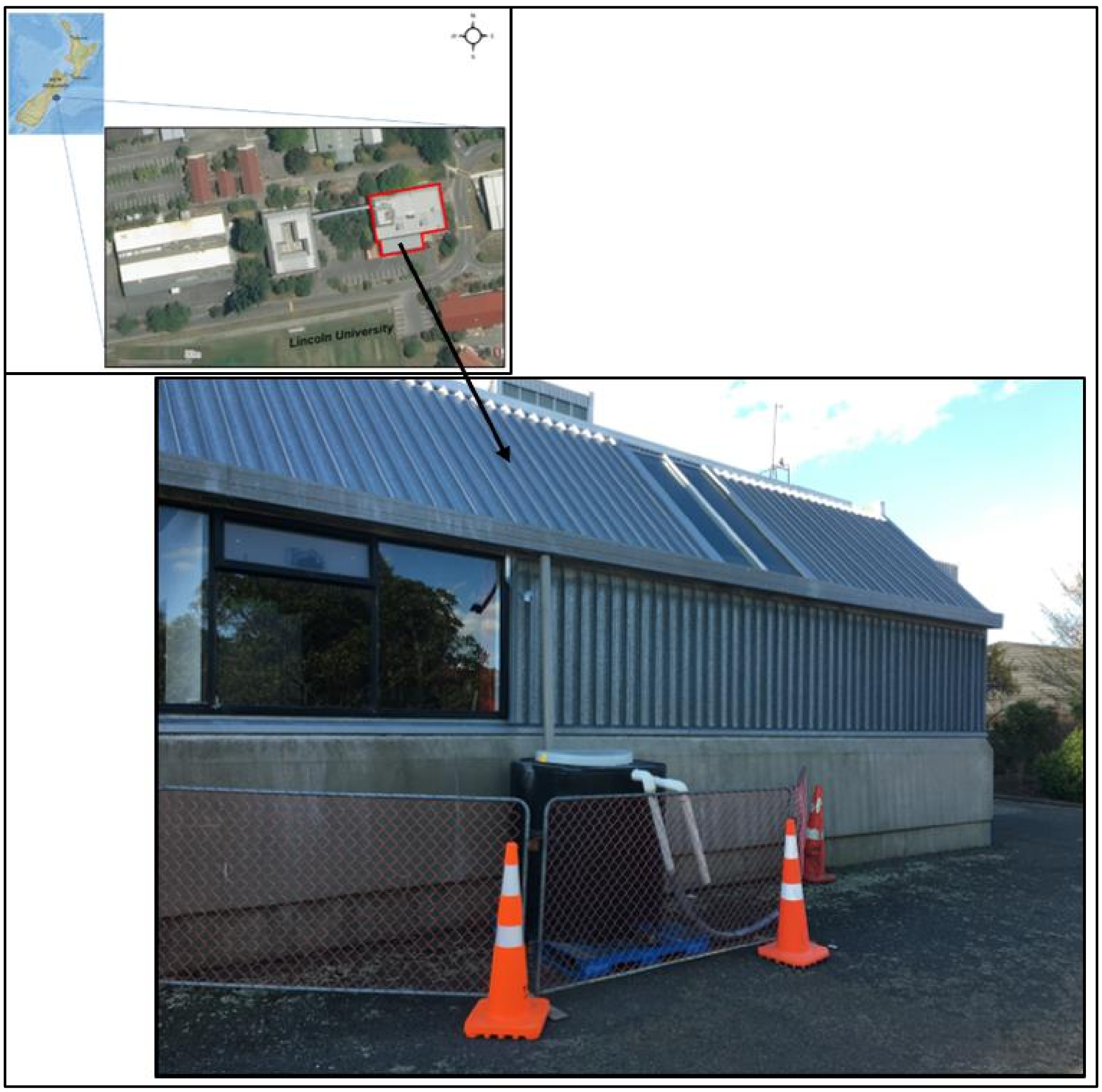


Figure 3.3: the 1.0 m^3 PVC tank placed to collect roof runoff at the Soil and Water Engineering Laboratory, Lincoln University.

3.2.2.1 The Design of Filtration Units and Testing Configurations

The filtration units were specifically designed to evaluate the potential use of TMS and UTMS as filtration media in the integrated rainwater tanks. Two types of outlets were used inside the filtration units to discharge treated water. The first type of filtration units included 1.0 m depths of each filtration media connected to a gravity-driven outlet, where gravity acts as the main driving force of water. The second type of filtration units included a siphonic-driven outlet, where negative pressure created inside the outlet was assumed to help to regulate water flow through the filtration media during unrestricted flow conditions. A transparent filtration unit was designed to evaluate the flow patterns through the filtration media in each outlet.

The aim of using two types of outlets was to assess the associated flow patterns and operational conditions of each outlet during unrestricted flow conditions. However, in both stages of experiments, all tests were performed under control saturated flow rates by using two PVC valves to eliminate the effects of outlets on the removal performance of zinc.

The two PVC valves were used to control the flow rates through the filtration unit in order to measure the removal efficiency of zinc during different filtration rates. All filtration units were formed by using 1.0 m lengths of 100 mm diameter PVC pipes fitted with PVC fittings including cover plates, elbows, bulkheads and reducers to connect the filtration unit to the IBC tank (Figure 3.4). The upper valve was used to control flow between the IBC tank and the filtration unit, while the lower valve was used to control outflow rates of the filtration rate. Controlled flow rates of 1, 3, 5, 10 L/min were used to represent actual operation conditions and to evaluate the associated removal performance of zinc in all experiments. A transparent 25 mm diameter breathing pipe was added between the upper valve and the filtration unit. The main aim of the breathing pipe was to release trapped air in the filtration media and to ensure full saturation conditions during the experiments.

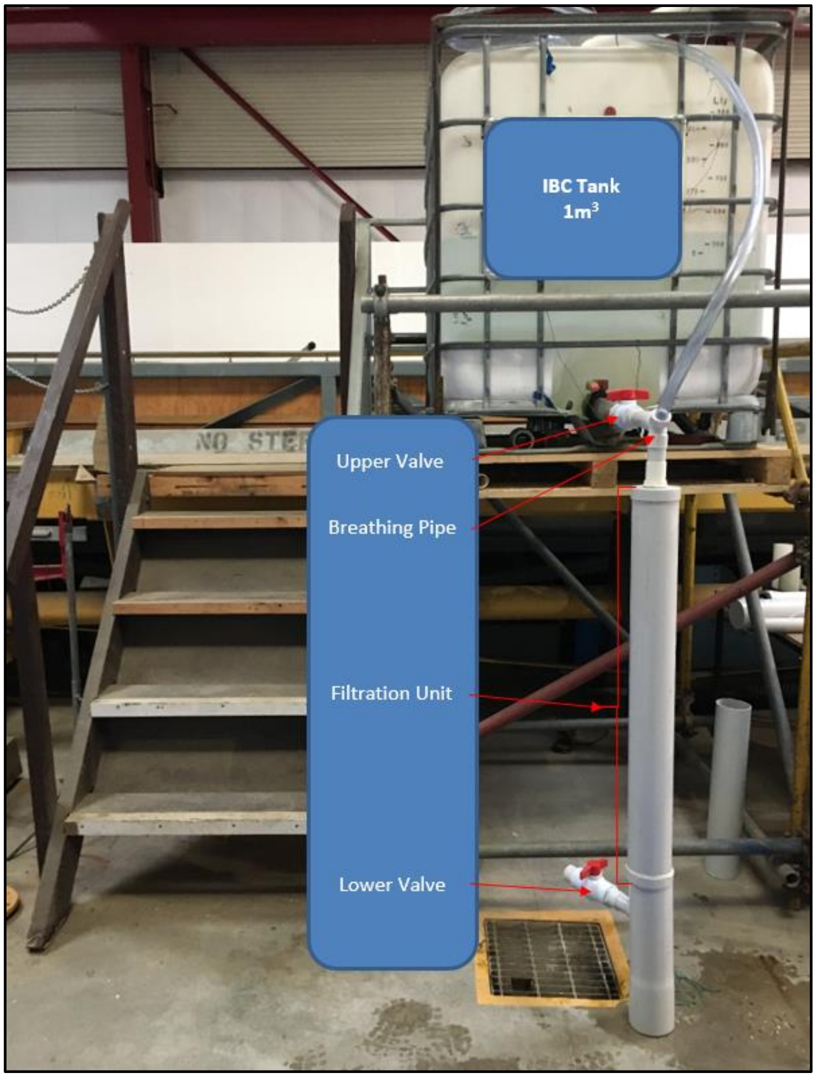


Figure 3.4: The configurations used to test the removal performance of the filtration units.

3.2.2.2 First Stage: 1 m depths of Filtration Media (Gravity Driven Outlet)

In this stage, 1 m depths of the TMS and UTMS were used as filtration media respectively, and a perforated pipe was used as an outlet in each filtration unit. The perforated pipe was formed by drilling 30 vertical holes of 8 mm into a 40 mm diameter PVC pipe (Figure 3.5). A PVC mesh with opening sizes of 1.0 mm was used to wrap the perforated pipe and at the top of the filtration unit to stop the filtration media being disturbed or discharged with the water flow (Figure 3.6). The filling of the filtration media inside the filtration unit was performed over three equal layers of the total depth of filtration media. After each layer was placed, a slight vertical shaking of the PVC pipe was performed to provide an equal degree of compaction through the filtration media.

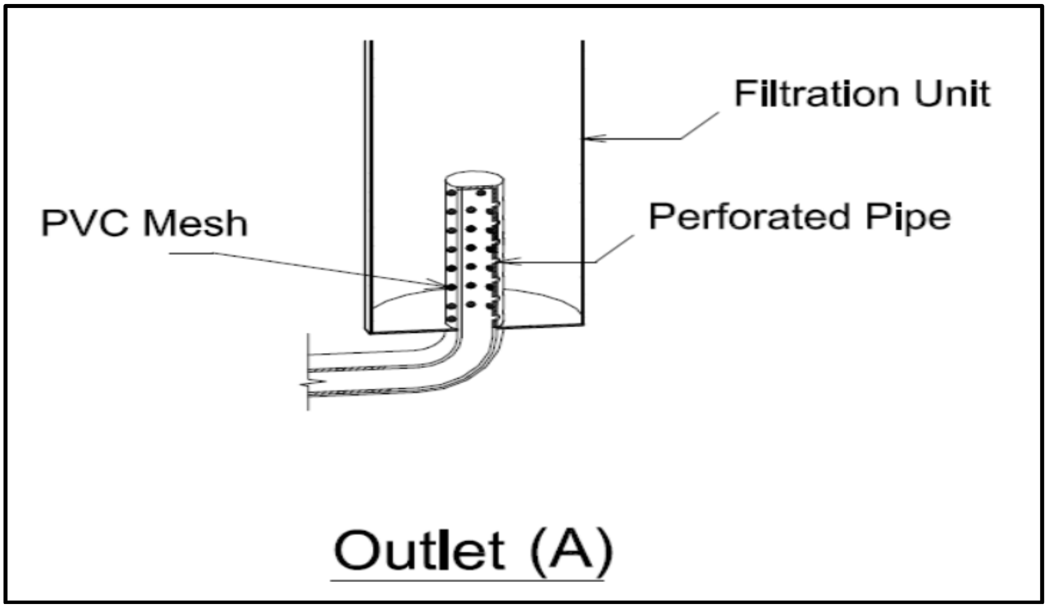


Figure 3.5: The perforated pipe configuration in the gravity-driven outlet.

Figure 3.6: The PVC mesh used at the top of each filtration unit to prevent the filtration media being discharged with the overflow through the breathing pipe.

3.2.2.3 Second Stage: 0.8 m Depths of Filtration Media (Siphonic Driven Outlet)

In this stage, 0.8 m depths of the TMS and UTMS was placed above the siphonic outlet as filtration media in each filtration unit. The siphonic outlet was formed by using an activator pipe (i.e. a closed vertical PVC pipe of 80 mm diameter with a total length of 0.2 m), and an internal vertical 40mm diameter PVC pipe and uniformed openings at the lower end of the filtration units (Figure 3.7). A similar PVC mesh with openings sizes of 1.0 mm was used to wrap the openings, and at the top of filtration media to stop the shells from being discharged with water flow.

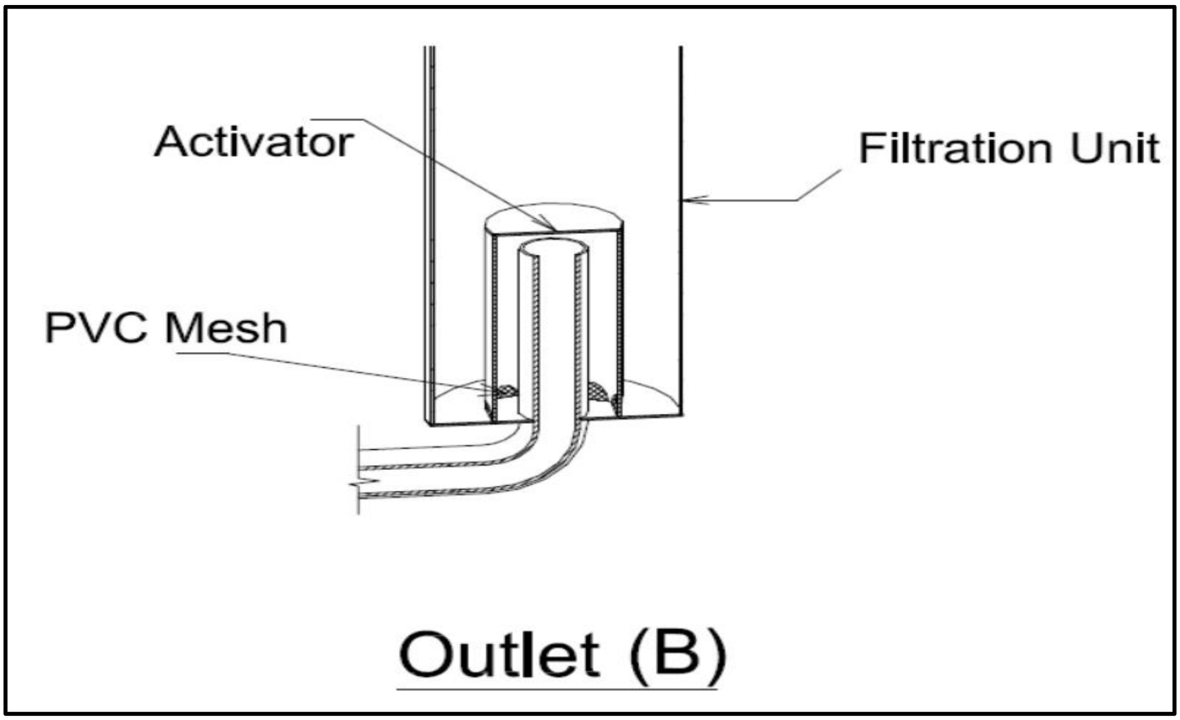


Figure 3.7: The siphonic outlet used in the second stage experiments.

3.2.2.4 Running the experiments

The following steps were performed to run both stages of experiments. The upper and lower valves were fully opened to flush the filtration media for 1 min and then closed. After that, the upper valve was fully opened to fill the filtration unit with water while the lower valve was closed. This step was performed to release all the trapped air via the breathing pipe and to ensure full saturation in the filtration media before running the tests. This step continued until the water level in the breathing pipe was stabilised to the same water level in the IBC tank. The lower valve then was partially opened and adjusted to a 1 L/min flow rate and left running for 10min before taking any water samples. This step was performed to refresh all the water that had longer contact time with the filtration media during the saturation process. The flow rate was then increased to 3 L/min and left running for 3 min. Similarly, the flow rate was then increased to 5 L/min and left running for 2 min and finally to 10 L/min and left running for 2 min. During each flow rate, plastic beakers of 500 ml volume were used to collect the samples from treated water. The samples were collected, and the pH was measured at the end of each flow rate setting. The whole process was repeated three times for each type of filtration media.

The preliminary assessment of the collected roof runoff showed that more 99% of zinc was in the dissolved form in roof runoff; consequently, all the treated samples were only analysed to measure the dissolved form of zinc. The collected samples were filtered directly by using 0.45 µm nylon filters into Inductively Coupled Plasma Mass Spectrometry (ICP-MS) tubes, and preserved by adding two drops of concentrated nitric acid and stored below 4C° until delivered to the ICP-MS laboratory for analysis at the University of Canterbury. During testing, the treated water was drained into a 20L container and a pH probe (i.e. HQ40D Portable Meter) was used to measure the pH readings during the testing. Random duplicates and blanks were included in every batch of samples to the ICP-MS, to detect any sources of error (i.e. human, instrumental) and to improve the precision of results analysis. The t-Test Paired Two Sample for Means method was used to perform the statistical analysis of results (i.e. Alpha = 0.05).

3.2.3 Characterisation of Mussel Shells

The physical and chemical characteristics of the TMS and UTMS were measured using a series of tests. Physical characteristics such as particle size distribution was measured using sieve analyse tests. Chemical characteristics and the microstructures of the TMS and UTMS were assessed using Scanning Electron Microscopy (SEM) and Energy Dispersive X-ray Spectroscopy (EDS) tests. The SEM tests gave magnified images of the microstructure of the shells by scanning the samples with electron beams (Figure 3.8), and the EDS tests gave an overall chemical concentration of each element present in the scanned shell (Goldstein et al., 2017). The main purpose of the

SEM/EDS tests was to compare between the chemical compositions and the microstructures of the TMS and UTMS, and to evaluate the presence of zinc on the shell after the filtration process.

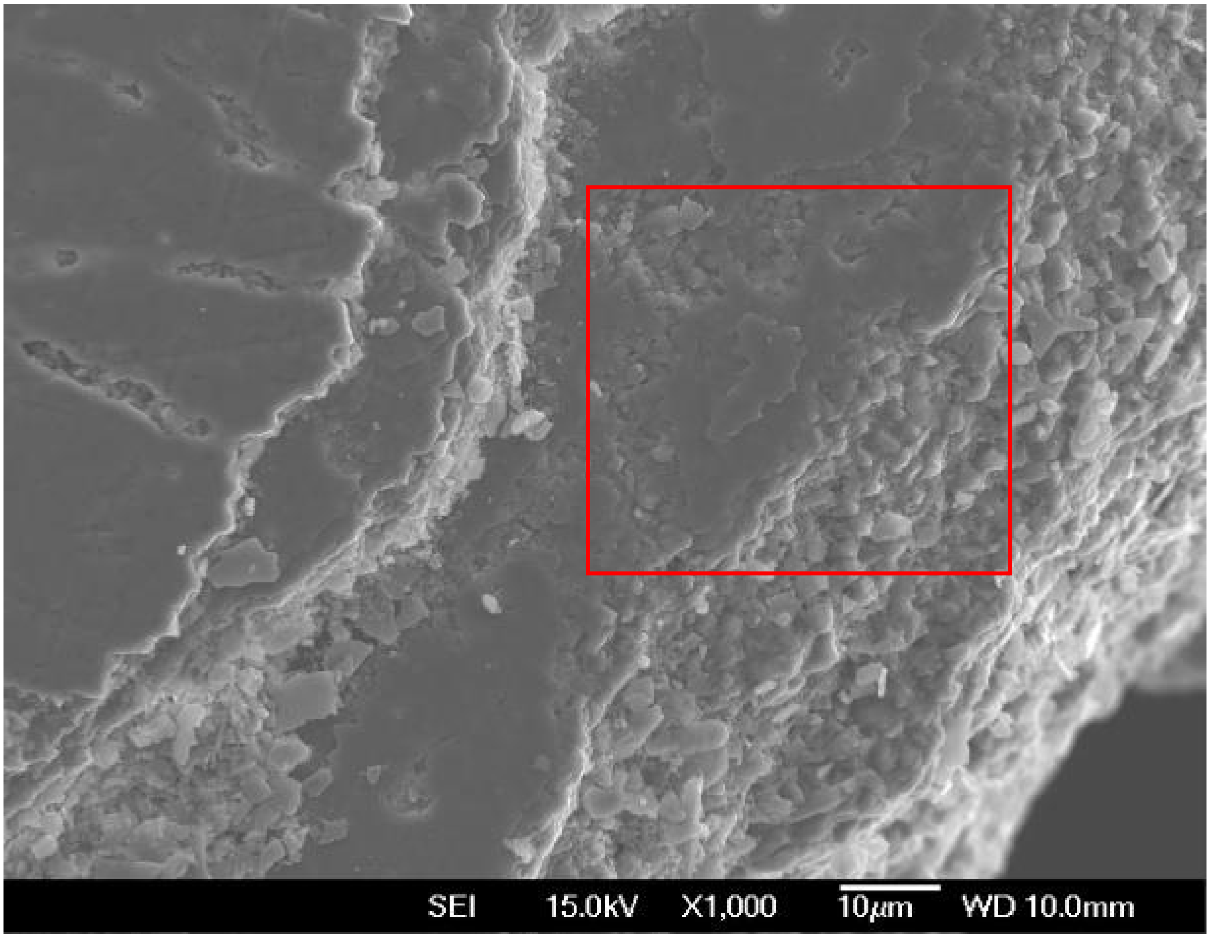


Figure 3.8: A SEM image showing the inner and middle layers of the shell. The highlighted rectangle shows the tiny particles that form the shell which provide the contact surface areas with water.

3.2.3.1 Sieve Analysis and Saturated Hydraulic Conductivity Test

A standard constant-head permeability test was used to measure the hydraulic conductivity of the selected particle size of the TMS and UTMS (Figure 3.9). The test unit was comprised of a standard clear acrylic pipe with an internal diameter of 44 mm and a total length of 300mm. Two manometers were connected to the upper and lower sides of the pipe in order to measure the difference in pressure in the upper and lower ends of the filtration media. Four tests were performed using different flow rates and collection times. The following equation was used to calculate the coefficient of hydraulic conductivity (K_{sat}):

$$K = \frac{Q * L}{\mathrm{A} * \mathrm{T} * \mathrm{h}} \qquad \text{(Eq. 3.1)}$$

Where:

K = coefficient of saturated hydraulic conductivity;

Q = quantity of water discharged;

L = distance between manometers;

A = cross-sectional area of the specimen;

T = time of discharge;

h = difference in pressure on manometers.

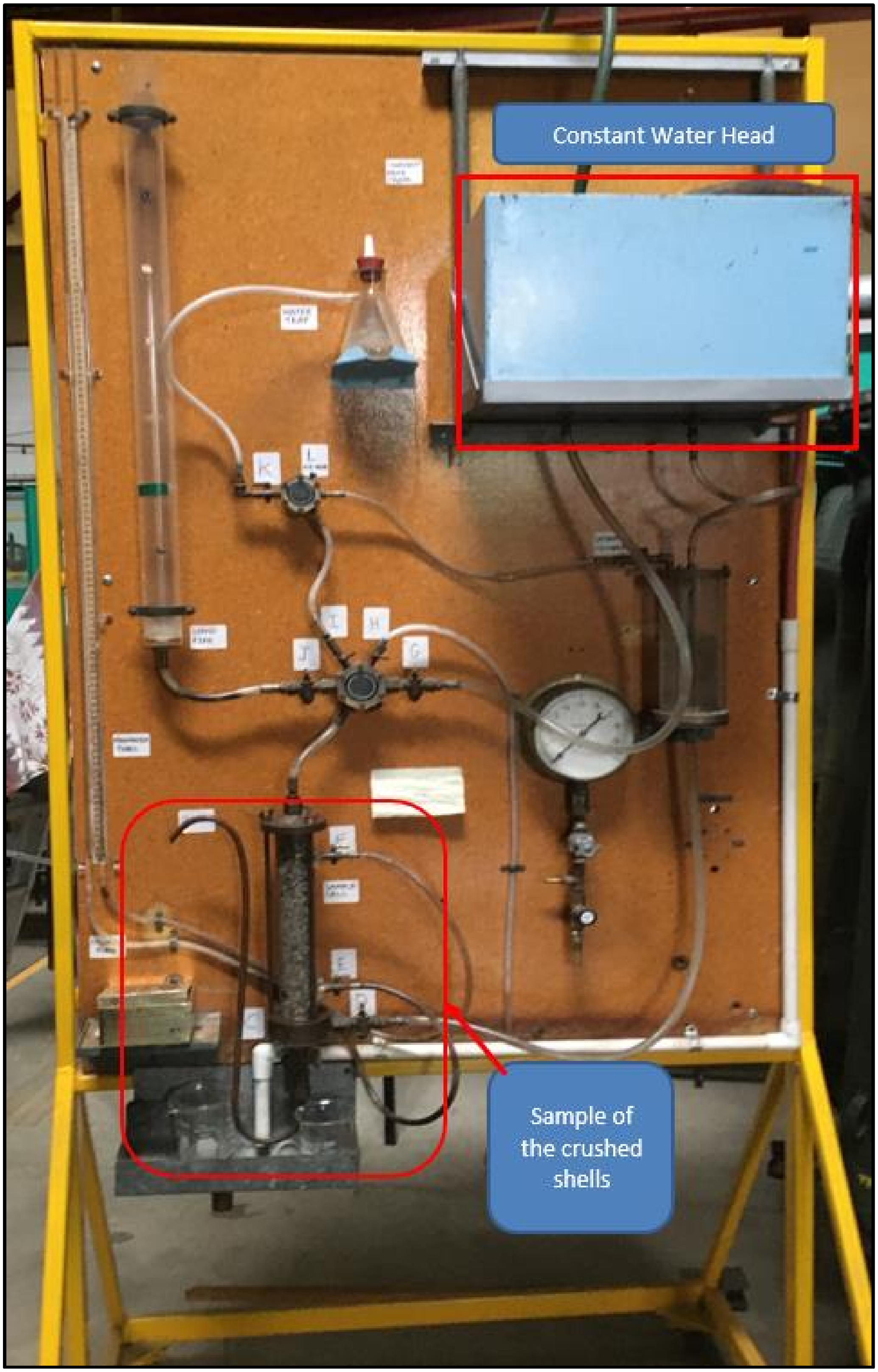


Figure 3.9: The unit used to measure the coefficient of hydraulic conductivity of the TMS and UTMS.

3.3 Results

3.3.1 Concentrations of dissolved Zinc in roof runoff

The collected roof runoff showed relatively high concentrations of dissolved zinc that ranged between 3071 – 3801 µg/L with an average concentration of 3347.2 µg/L. This value is ca. 200 times higher than the recommended concentration of zinc (i.e. 15 µg/L) to protect 90% of the freshwater organisms in urban streams according to the ANZECC's guidelines (ANZECC, 2000). All the treated runoff samples showed higher zinc concentration than the recommended concentration of 15 µg/L. The zinc concentrations for the 1.0 m and 0.8 m depths of TMS and UTMS during each flow rates and the associated removal efficiencies are shown in tables 3.1 and 3.2 respectively.

Table 3.1 The dissolved zinc (µg/L; mean ± SD), and the removal performance (%; mean ± SD) for the TMS and UTMD with 1.0 m depths of filtration media. Flow rate = 0 (n = 2), Flow rates =1, 3, 5, 10 (n=3). The t-Test Paired Two Sample for Means method was used to perform the statistical analysis of results (Alpha = 0.05).

	TMS		UTMS		Statistical Analysis
Flow (L/min)	**Zinc (µg/L)**	**Removal Performance (%)**	**Zinc (µg/L)**	**Removal Performance (%)**	**P value***
Untreated Runoff	**3582.95 ± 308**	**0.00**	**3582.95 ± 308**	**0.00**	__
1	**45.65 ± 8.88**	**98.80 ± 0.23**	**20.72 ± 3.05**	**99.38 ± 0.1**	**≤ 0.05**
3	**43.24 ± 4.53**	**98.86 ± 0.12**	**94.39 ± 8.61**	**97.19 ± 0.26**	**≤ 0.05**
5	**341.13 ± 40.92**	**91.03 ± 3.71**	**285.18 ± 71.34**	**91.52 ± 2.12**	**> 0.05**
10	**421.29 ± 46.86**	**88.92 ± 1.23**	**656.18 ± 132.05**	**80.45 ± 3.92**	**> 0.05**

** Correlation is significant at the 0.05 level (2-tailed).*

Table 3.2 The dissolved zinc (µg/L; mean ± SD), and the removal performance (%; mean ± SD) for the TMS and UTMD with 0.8 m depths of filtration media. Flow rate = 0 (n = 2), Flow rates =1, 3, 5, 10 (n=3). The t-Test Paired Two Sample for Means method was used to perform the statistical analysis of results (Alpha = 0.05).

	TMS		UTMS		Statistical Analysis
Flow (L/min)	Zinc (µg/L)	Removal Performance (%)	Zinc (µg/L)	Removal Performance (%)	P value*
Untreated Runoff	3111.66 ± 59	0.00	3111.66 ± 59	0.00	—
1	141.6 ± 28.9	95.4 ± 0.94	87.9 ± 13.5	97.2 ± 0.42	≤ 0.05
3	178.4 ± 28.1	94.2 ± 0.91	541.9 ± 60.64	82.8 ± 1.92	≤ 0.05
5	593.9 ± 114.19	80.7 ± 3.72	1155 ± 185.94	63.4 ± 5.9	≤ 0.05
10	1252.3 ± 150.17	59.2 ± 4.89	1690 ± 125.31	46.4 ± 3.97	≤ 0.05

** Correlation is significant at the 0.05 level (2-tailed).*

3.3.2 Removal Performance of Zinc

The results of both the TMS and UTMS showed high removal performance of dissolved zinc. The overall average removal performance for all the tested flow rates by using the TMS were 94% and 82% for the 1.0 m and 0.8 m depths of filtration media respectively. Alternatively, the overall removal performance of all the tested flow rate by using the UTMS were 92% and 72% for the 1.0 m and 0.8 m depths of filtration media respectively (Figure 3.10).

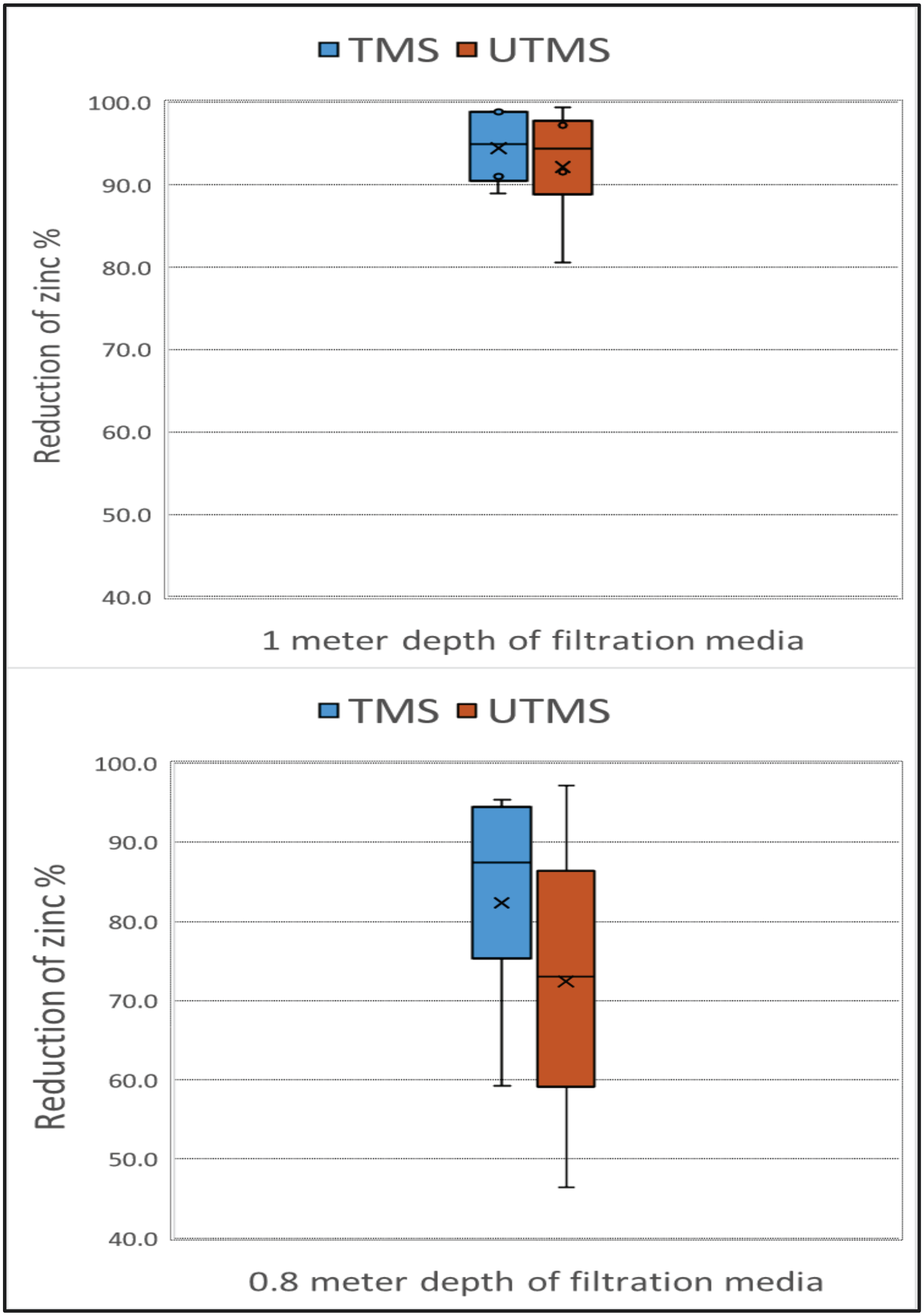


Figure 3.10: The average removal performance of zinc by using 0.8 and 1.0 m depths of the TMS and UTMS as media.

The removal performance decreased as the flow rates increased through both the TMS and UTMS in each type of filtration units (Figures 3.11 - 14). Similarly, the results showed higher removal performance for the 1.0 m depth of filtration media comparing to the 0.8 depth during all flow rates. In both depths, the TMS and UTMS showed higher removal performance of zinc during low flow rates (i.e. 1 L/min) with averages of 97%, and 98% respectively. During high flow rates (i.e. 10 L/min), the TMS showed higher removal efficiency with averages of 74% comparing to 63% for the UTMS.

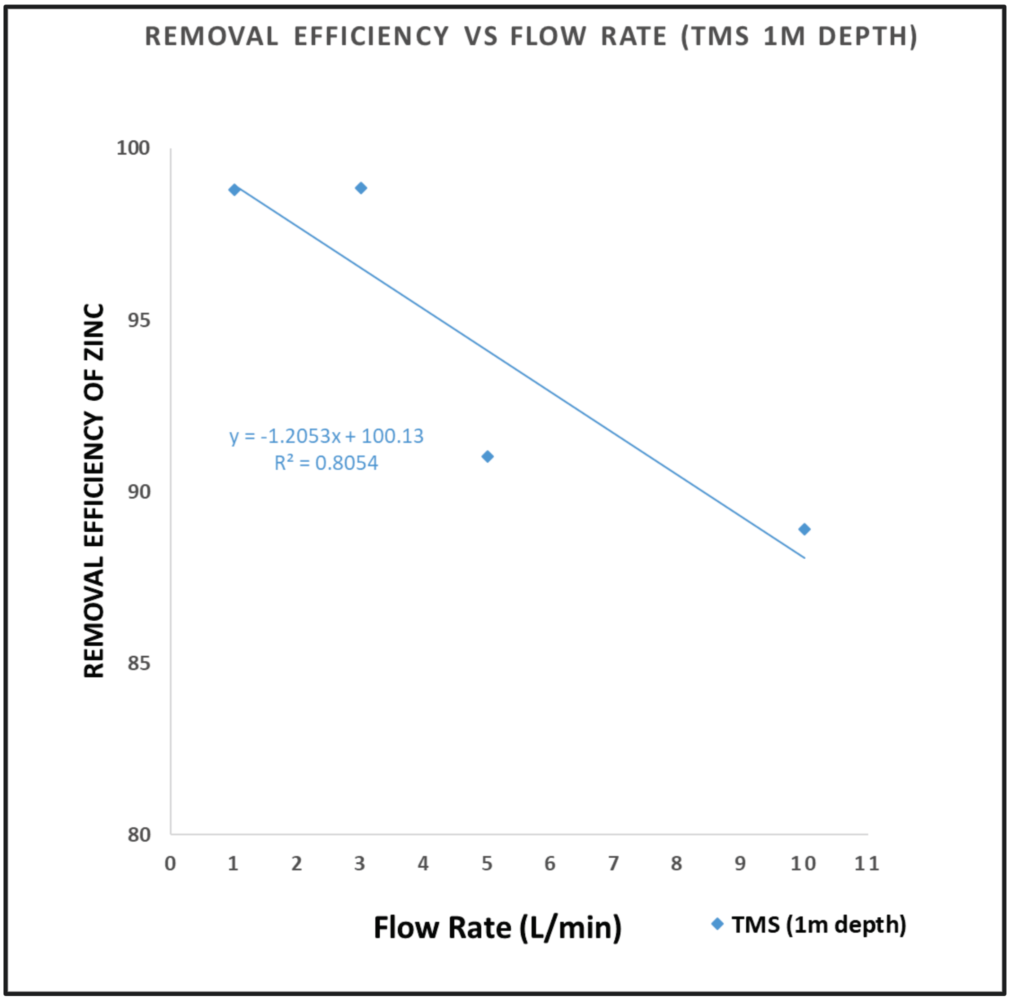


Figure 3.11: The Removal Efficiency Vs Flow Rate of the TMS (1.0 m) Depth.

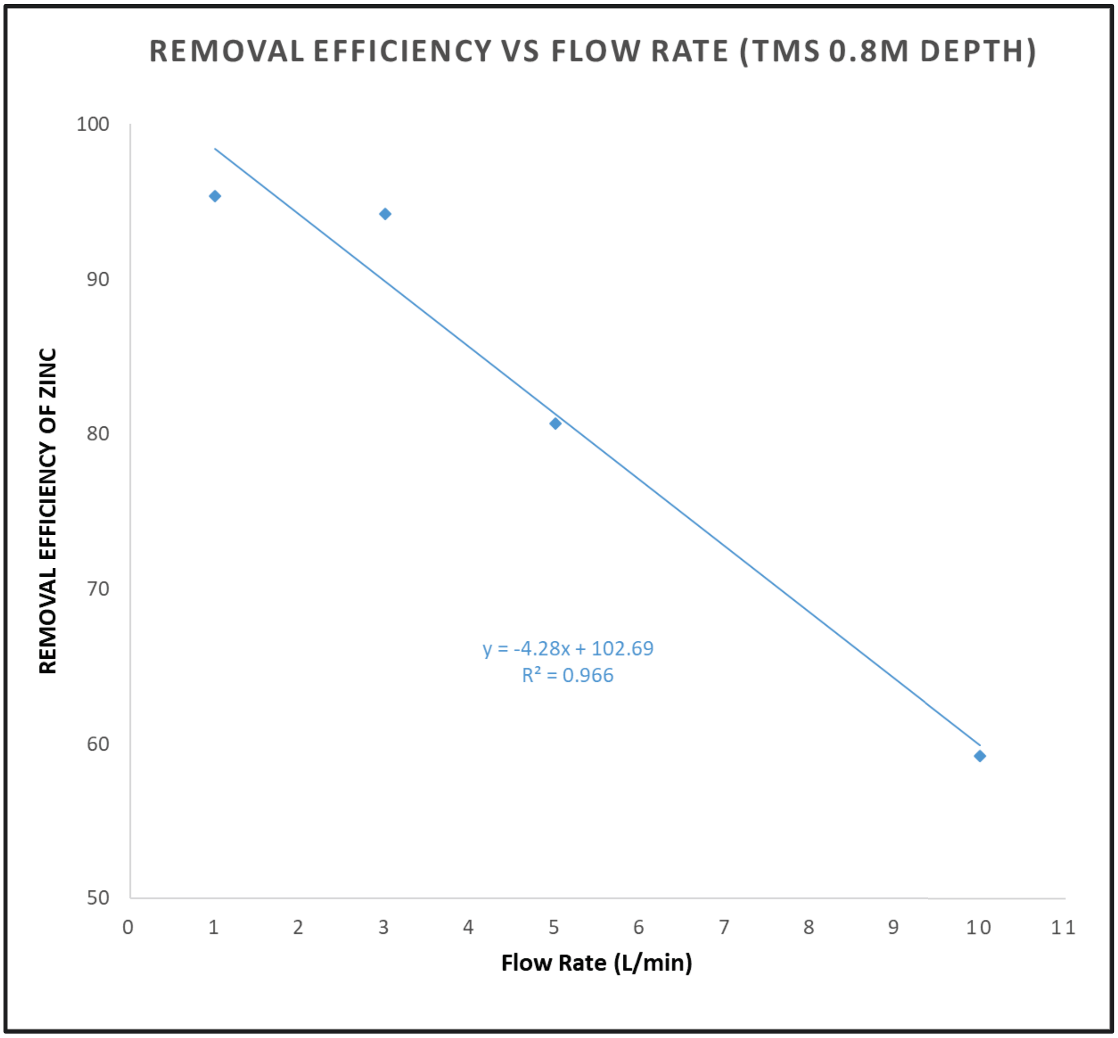


Figure 3.12: The Removal Efficiency Vs Flow Rate of the TMS (0.8m) Depth.

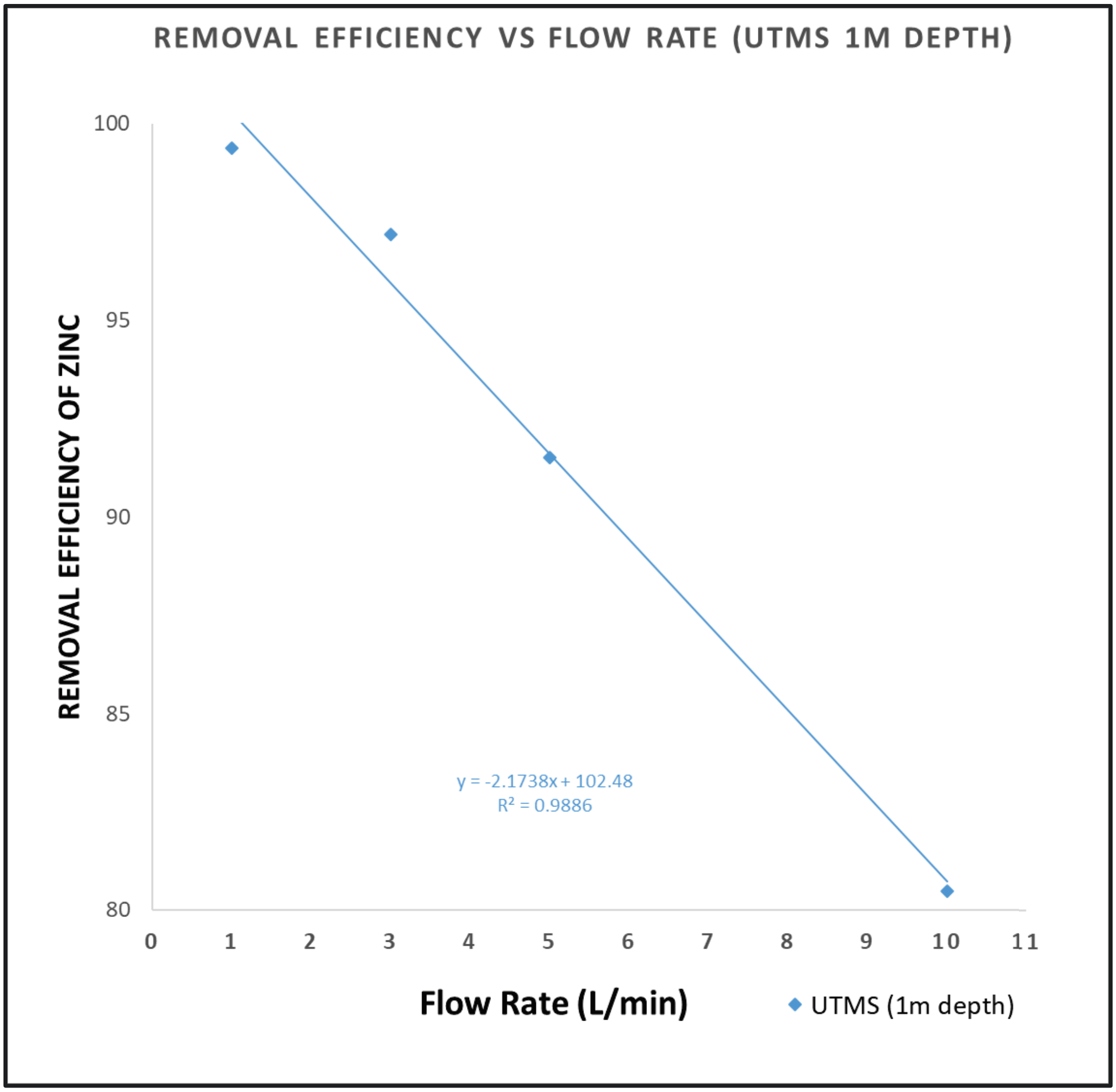


Figure 3.13: The Removal Efficiency Vs Flow Rate of the UTMS (1.0 m) Depth.

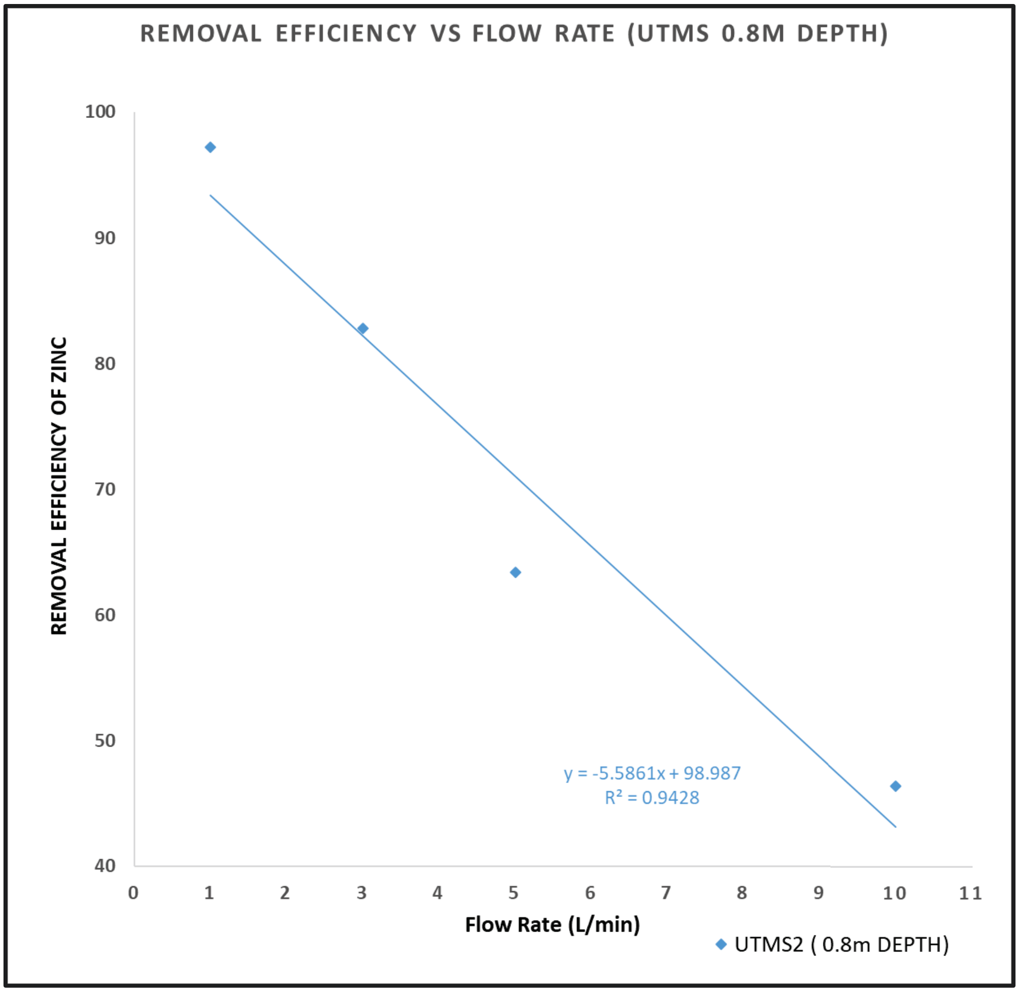


Figure 3.14: The Removal Efficiency Vs Flow Rate of the UTMS (0.8m) Depth.

3.3.3 pH changes

The pH levels of the collected roof runoff ranged between 6.35 – 6.97. The results showed increases in the pH levels for the treated runoff (Figure 3.15). The TMS showed greater changes to the pH levels of runoff compared to the UTMS. Using the TMS increased the pH of the runoff from an average of 6.75 to an average of 8.77, while using the UTMS increased the pH from an average of 6.75 to an average pH of 8.24 for all the tested flow rates and depths.

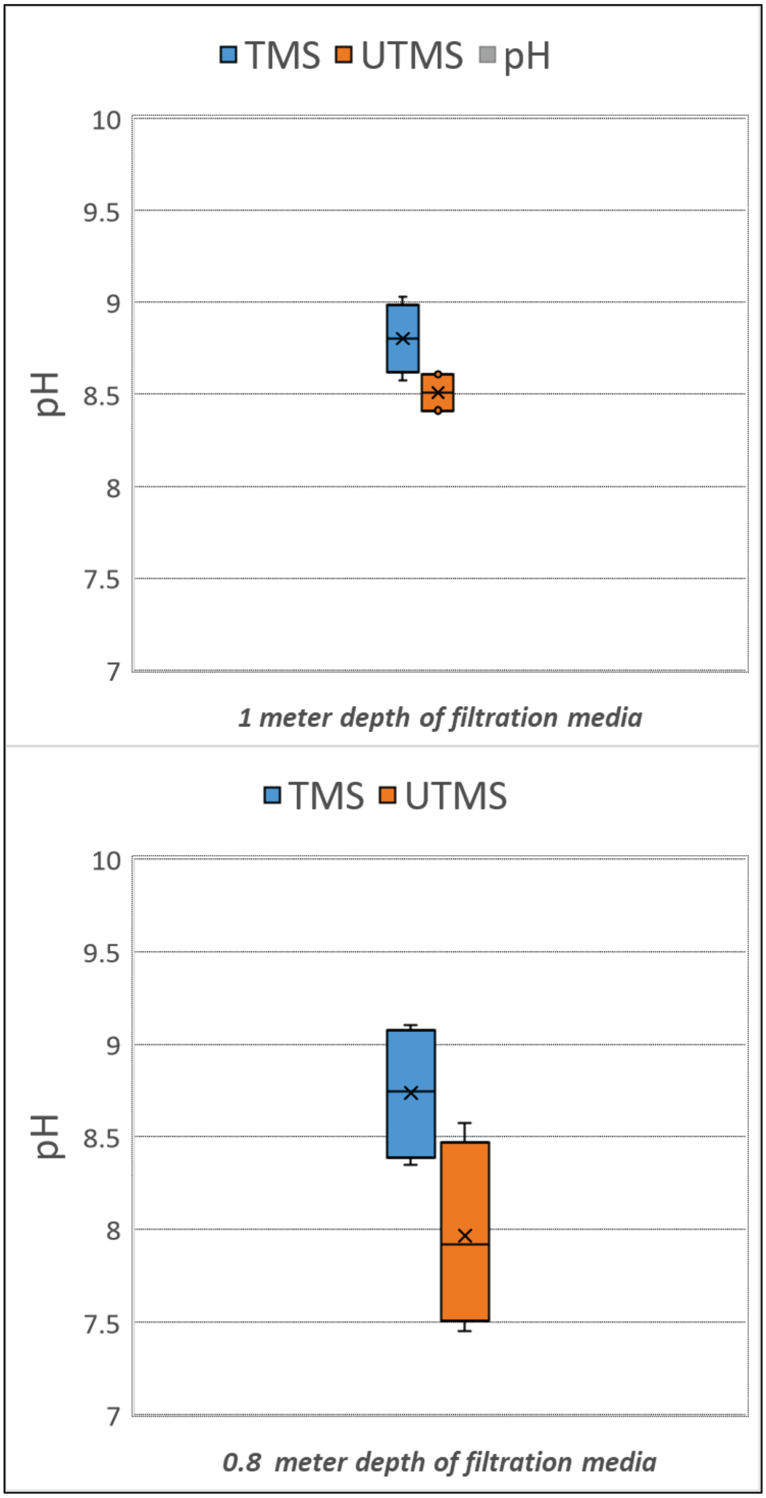


Figure 3.15: The pH levels for the treated runoff by using the TMS and UTMS.

3.3.4 Characteristics of Mussel Shell

The chemical characteristics and microstructure of the TMS and UTMS were found to be similar. The two main elements that constitute the shells were calcium (Ca) and oxygen (O) with average percentages of 51% and 46% respectively (Figure 3.16). Generally, there was no presence of zinc on the tested TMS and UTMS shells which were taken from the top of each filtration unit after conducting the filtration tests.

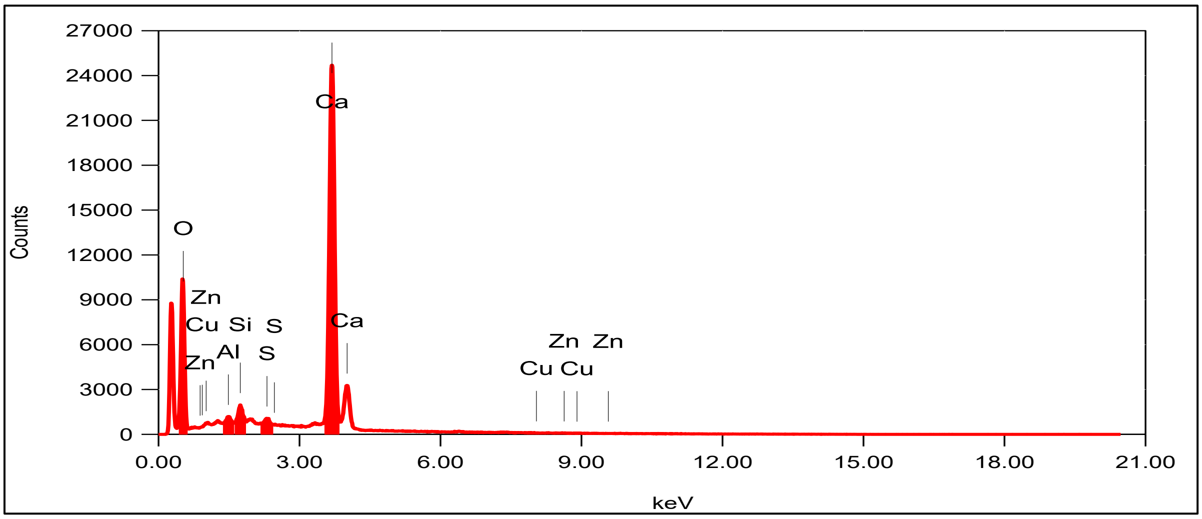


Figure 3.16: Chemical composition of the crushed shells tested after using it as filtration media.

The results of the SEM/EDS tests confirmed previous findings regarding the microstructures and the chemical elements that constituted of the shell. The microstructures of the TMD and UTMS captured during the SEM tests confirmed the internal structure of shell which is comprised of three layers (Mo et al., 2018). The outer layer (i.e. periostracum) is a green-papery cover and composed of proteins without any minerals. The middle layer (i.e. prismatic) is the main thick layer of the shell with structure consisted of parallel calcite prisms that oriented 30° toward the outer layer. The inner layer (i.e. nacre) is generally classified as a biomineralized layer with 5% of proteins (Gao et al. 2015). Calcium and oxygen were the main elements in the shell which support the hypothesis that CaCO3 is the main chemical compound that contributes to the removal of dissolved metals from runoff (Craggs et al., 2010).

3.3.5 Sieve Analysis and Saturated Hydraulic Conductivity

Figure 3.17 shows the particle size distribution of the TMS and UTMD used in the tests. This was produced by conducting a sieve analysis test for a representative sample of the TMS and UTMS. The associated hydraulic conductivity for both the TMS and UTMS shells ranged between 80.9 – 118.4 m/h (i.e. 16.67 L/min).

Sieve Number	Opening Diameter (mm)	Shells Retained (%)	Shells Passing (%)
1/2"	12.70	0.0	100.0
7/16"	11.20	0.2	99.8
3 1/2"	5.60	43.9	55.9
#4	4.75	8.0	47.9
#5	4.00	10.9	37.0
#7	2.80	14.9	22.1
#200	0.074	19.3	2.8
Pan		2.8	0.0
		100.0	

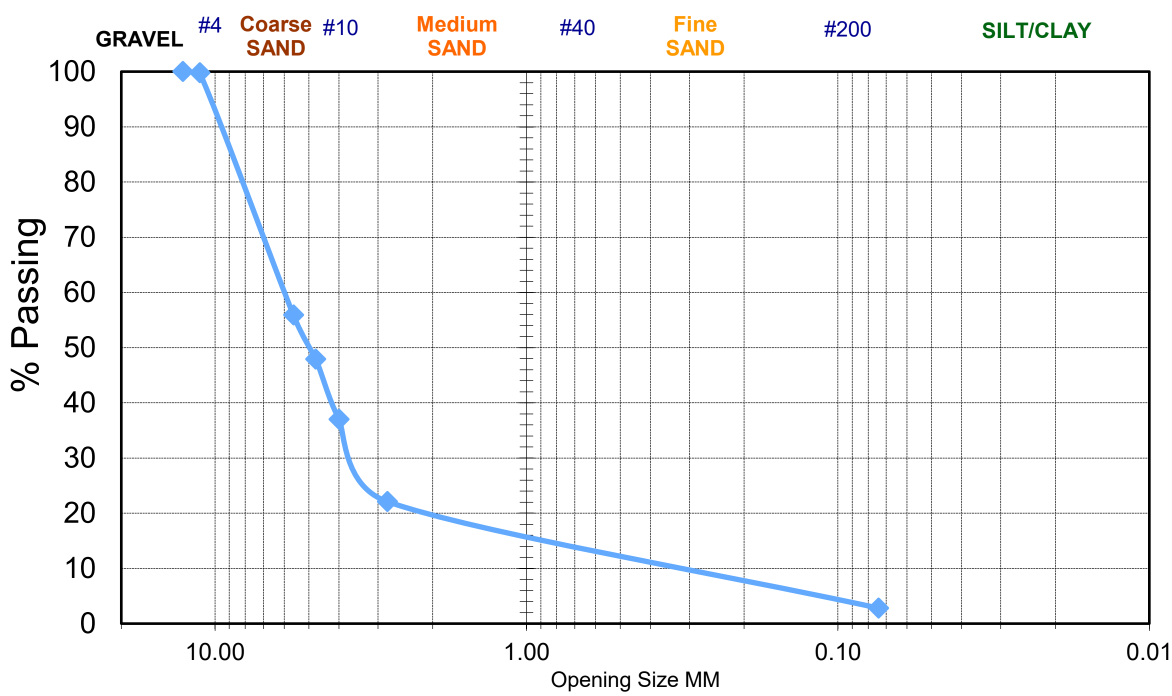


Figure 3.17: The particle size distribution of the used TMS and UTMS.

3.4 Discussion

3.4.1 Removal Performance of Dissolved Zinc

High removal efficiencies of dissolved zinc were observed by using both the TMS and UTMS in this research. In general, the heat treatment of mussel shells improved the removal performance of dissolved zinc. For example, the TMS media showed significant higher (i.e. $p \leq 0.05$) removal performance of zinc comparing to the UTMS during the 3, 5, and 10 L/min flow rates for the 0.8 m depths of filtration media. The increase in pH of the treated runoff supports previous findings regarding the high removal performance of zinc at high pH levels, which is perhaps due to ion exchanges process between metal ions in water and calcium ions in the shell along with the adsorption capacities of $CaCO_3$ at high pH levels (Barakat, 2008). Previous studies identified either adsorption and precipitation (i.e. hence filtration), or a combination of both, as the main mechanisms for the removal of dissolved metals by using mussel shells (Craggs et al., 2010 ; Good et al., 2014 ; Bremner C. et al., 2020). Interestingly, in this study, the EDS tests did not show a high presence of zinc particles on the surface of tested shell particles which was taken at the top of each filtration units after the filtration process. This support the theory that adsorption is perhaps, the main removal mechanism of dissolved metals and suggest the precipitation as secondary removal mechanism; however, future research is required to precisely demonstrate the removal mechanism of dissolved metals.

3.4.2 Contact time and Removal Performance

The TMS and UTMS showed direct correlations between the removal performance of dissolved zinc and the contact time between water and the TMS and UTMS. The increase in flow rates through the filtration media (i.e. shorter contact time) resulted in lower removal performance, while, for similar flow rates, the increase in filtration media depths (i.e. longer contact time) showed higher removal performance. The removal performance of zinc was consistent for each flow rate category which indicated that both the TMS and UTMS could be used as a reliable filtration media under controlled filtration conditions.

All the tested flow rates (i.e. 1, 3, 5, 10 L/min) were below the maximum flow rate of the filtration unit which was estimated at an average of 17 L/min. Therefore, using the filtration units with unrestricted flow conditions would result in lower removal performance of zinc due to the shorter contact time with the filtration media. The use of similar filtration units without valves to control flow rates would require longer depths of the filtration media or larger surface area to achieve a relatively high removal performance.

3.4.3 Characteristics of Mussel Shell

The SEM/EDS results showed similar microstructures and chemical elements in both the TMS and UTMS. The outer layer appeared to have the least contribution in the process for removing dissolved metals as most of this layer peeled off during the crushing process in the UTMS, while In the TMS most of the outer layer was burned during the heat treatment process, yet the TMS showed higher removal performance. The middle layer of the shell is perhaps the main contributor to the removal process of dissolved metals. This is likely due to its microstructure that comprised of multi arrays of aligned prisms of $CaCO_3$ parallel to each which provide substantial contact surface area for the ion exchange and adsorption reactions with water. The inner layer is generally classified as a biomineralized layer with 5% of proteins (Gao et al. 2015). Thus, the heat treatment of the shell perhaps denatured all proteins and organic matters from the three layers which provided larger surface areas of $CaCO_3$ to be exposed to runoff.

3.4.4 Gravity and Siphonic Outlets

A noticeable difference in flow patterns between gravity and siphonic outlets was observed during unrestricted flow conditions. The difference in flow patterns was observed during the flow tests of the transparent filtration unit which was designed to assess the difference in flow patterns of the two used outlets (Figure 3.18). The difference between the two outlets was observed at the start and end drainage. Where the sudden changes in water pressure in the siphonic outlet produce a slight reverse flow of air bubbles at the bottom of the filtration unit which resulted in flushing out the suspended fine particles on the PVC mesh in the outlet. In actual filtration units, using similar siphonic outlets during unrestricted flow conditions would be more beneficial to extend the life span of the filter. This is due to the observed self-cleaning process in the filtration unit. However, there was no difference in flow patterns during control flow conditions (i.e. using the valves). In both stages of the experiments, all water samples were taken during controlled flow conditions to eliminate the effects of the two outlets on water flow patterns.

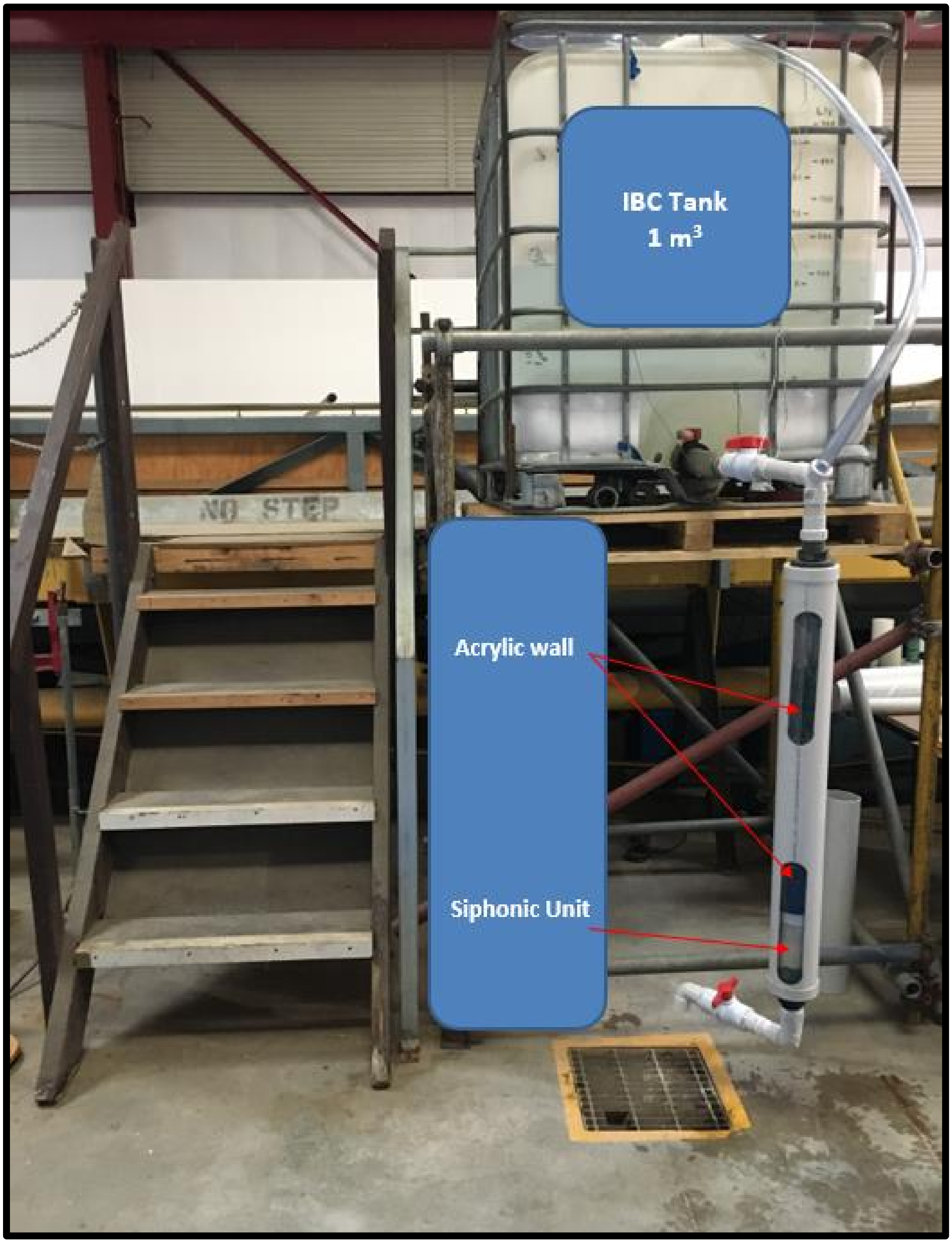


Figure 3.18: The transparent filtration unit which was designed to evaluate the flow patterns through the filtration media during the gravity and the siphonic flow conditions.

3.5 Chapter Summary

The collected roof runoff showed relatively high concentrations of dissolved zinc that ranged between 3071 – 3801 µg/L with an average concentration of 3347.2 µg/L. This value is ca. 200 times higher than the recommended concentration of zinc (i.e. 15 µg/L) to protect 90% of the freshwater organisms in urban streams according to the ANZECC's guidelines (ANZECC, 2000). After the filtration process, all the treated runoff samples showed higher zinc concentration than the recommended zinc concentration of 15 µg/L.

Both the TMS and UTMS showed high removal efficiency of dissolved zinc from roof runoff. The TMS media showed significantly higher ($p \leq 0.05$) removal performance of zinc comparing to the UTMS media for the tested flow rates during the 0.8 m depths, while the 1.0 depths of the TMS showed higher average removal efficiencies but with there was no significant difference ($p > 0.05$) comparing to the removal efficiencies of UTMS for the overall tested flow rates. The average overall removal efficiencies of zinc by using the TMS were estimated at 94% and 82% for the 1.0 m and 0.8 m depths of filtration media respectively. Similarly, the average overall removal efficiencies by using the UTMS were estimated at 92% and 72% for the 1.0m and 0.8 m depths respectively for all the tested flow rates.

The removal performance decreased as the flow rates through the filtration media increased, while for similar flow rates, deeper filtration media showed higher removal performance. In the same context, higher removal performance was achieved at higher pH levels.

The hydraulic conductivity of the selected particle sizes ranged between 80 - 110 m/h. The maximum saturated flow rate of the filtration media was estimated at 17 L/min. The removal performance of zinc was consistent for each flow rate category. Correlations between the flow rates and removal performance were developed for the TMS and UTMS and can be used to design future filtration systems with similar filtration media.

The treatment method of having PVC valves to control flow rates through the filtration media worked well to produce a consistent high removal performance of zinc for each flow rate category. During uncontrolled flow conditions (i.e. both valves fully open) the siphonic-driven outlet showed effective drainage performance and resulted in faster stable flow, and once water flow stopped the difference in water pressure caused sudden changes in water flow (i.e. reverse airflow) that removed suspended particles near the outlet openings.

The SEM/EDS results showed similar microstructures and constituents for the TMS and UTMS. The outer layer or periostracum appeared to have the least contribution in the process of removing dissolved metals. This was concluded as most of this layer peeled off during the crushing process in the UTMS, while In the TMS most of the outer layer was burned during the heat treatment process, yet the TMS showed higher removal performance. The middle layer of the shell is perhaps the main contributor to the removal process of dissolved metals. This was concluded due to its observed microstructures which were comprised of parallel arrays of aligned prisms of $CaCO_3$ that provide substantial contact surface area with water for ion exchanges and adsorption reactions.

Chapter 4: GIS-SWMM Modelling

4.1 Introduction

This chapter presents the GIS-SWMM modelling performed to investigate the use of RWH tanks to mitigate stormwater runoff volumes and peak flows in urban areas. This chapter included the methodology used to simulate the proposed management scenarios of the tanks, the simulation results, discussions, and conclusions in this chapter.

The use of rainwater tanks was simulated for two urban blocks located in the Wigram suburb in Christchurch, New Zealand. The first block represented typical residential land use in Christchurch, while the second block represented typical industrial land use. Three management scenarios were defined and simulated to evaluate the mitigation performance of the tanks.

The first simulated scenario is the Business As Usual (BAU) which represented the reference of actual stormwater outflow volumes and peak flows in the selected blocks during rainfall events without the use of rainwater harvesting tanks. The second simulated scenario was the Rainwater Harvesting for Toilet Flushing (RWH-TF) which represented the use of rainwater tanks to supply water for toilet flushing uses in the existing buildings in each selected block. The third simulated scenario was the Rainwater Harvesting for Stormwater Treatment (RWH-ST) which represented the use of rainwater tanks with filtration units as stormwater detention units without using the of the collected rainwater. The tanks in RWH-ST scenarios were simulated to detain, treat, and slowly discharge the treated roof runoff into the stormwater network.

The three management scenarios were simulated over a 12-year period starting from 4 November 2007 until 4 November 2019. This period was selected according to the available rainfall data for the 5-min resolution in Christchurch City which was required to accurately assess the reductions in peak and total runoff volumes (i.e. Rainfall data before 4 November 2007 were available with 1-hour resolutions only). During this period, 5-min time steps were used to analyse the hydrological and hydraulic responses of the selected blocks during each scenario. The mitigation performance was evaluated in the form of percentage reductions in peaks and total volumes of outflow for each rainfall event during the simulation period. The tank performance to treat roof runoff was also assessed during the RWH-ST scenario, while the tank performance to maintain water supply for toilet flushing uses in the buildings was assessed during the RWH-TF scenario.

4.2 Methodology

The following methodology was defined to evaluate the mitigation performance of RWH tanks as stormwater control measures in urban areas.

4.2.1 Sites selection

Two urban locations were selected in the Wigram suburb in Christchurch, New Zealand to simulate the use of RWH tanks (Figure 4.1). The Wigram suburb is located in the south-west of Christchurch and includes the main headwaters and tributaries of Heathcote River. Due to the topography of the Wigram subcatchment, large scale detention basins such as the Wigram East Basin were implemented to reduce the flood risks around properties near the Heathcote tributaries (CCC, 2008). Additional detention basins are currently under construction to further reduce flood risk in the area. The water quality of tributaries and streams have been severely degraded due to the high level of contaminants in surface runoff draining from surrounding developed lands, which in most cases are industrial lands (CCC, 2008). For instance, Haytons Stream is a significant tributary of the Heathcote River that flows through the Wigram suburb, and its long-term monitoring data of water quality showed consistent high zinc concentrations that significantly exceed the ANZECC's recommended guidelines (O'Sullivan & Charters, 2014). The Haytons Stream catchment has been classified as a priority catchment due to its high level of pollution and flood issues (Ecan, 2012), and therefore the two following locations were selected to simulate using rainwater harvesting tanks as stormwater control measures in the Wigram suburb.

The first selected location (Block A) represents residential land use, while the second location (Block B) represents industrial land use in Christchurch. The stormwater network specification of the selected blocks such as pipes diameters, locations, and depth of manholes were taken from the Water Asset Maps which were provided by Christchurch City Council (CCC, 2019). The land use of each selected block was classified into three categories: rooftops, pavement areas, and green areas. The associated analysis for the impervious and pervious ratio was calculated based on the aerial photography in the Water Asset Maps which was also provided by Christchurch City Council (CCC, 2019).

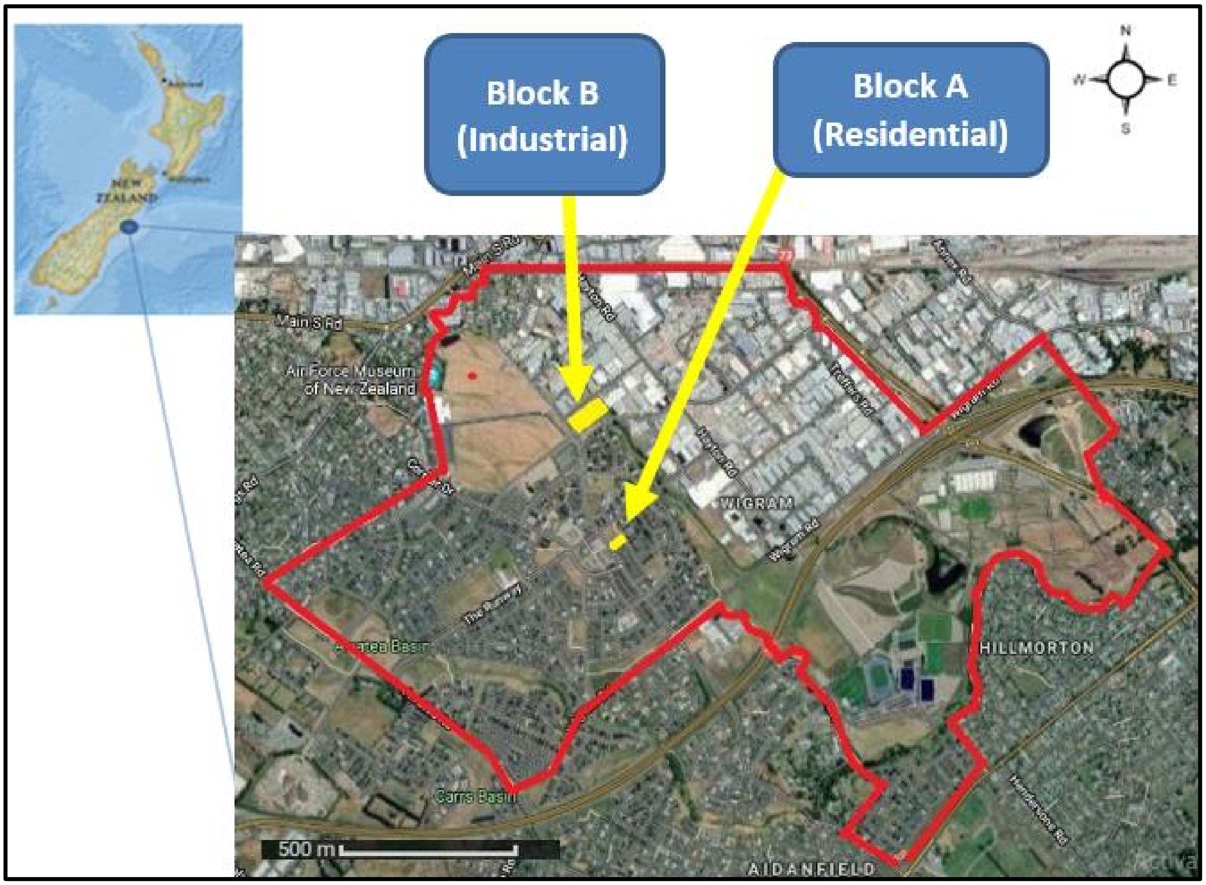


Figure 4.2 The boundary of Wigram suburb, and the locations of Block A, and Block B.

4.2.1.1 Urban Residential Area (Block A)

The residential block is comprised of three land parcels with a total area of 1,405 m^2. Each land parcel has one residential house surrounded by green areas and equipped with parking and pavement areas as illustrated in figure 4.2. The land use analysis showed that 59 percent of the residential block was covered with impervious surfaces, and 33 percent of the total area was used as roof areas as illustrated in Table 4.1.

Table 4.1 Land use characteristics of the residential block (Block A)

Land use	Area (m^2)	Area Ratio
Rooftops	535	0.38
Pavement Lot	300	0.21
Green Area	570	0.41
Total Area	1405	1.00
Total Impervious	835	0.59
Total Pervious	570	0.41

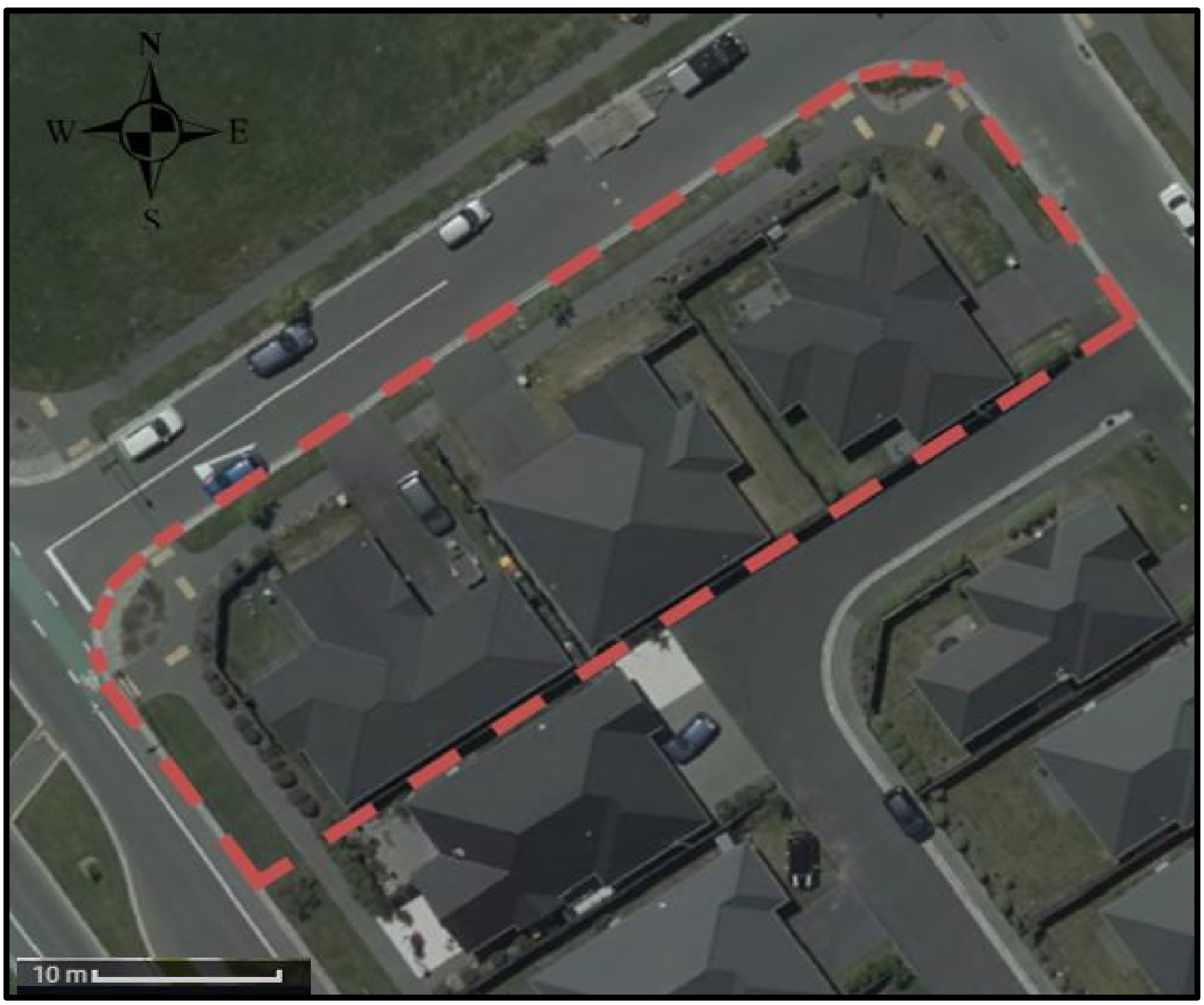


Figure 4.3: Aerial image showing the buildings and boundary of the residential Block A (CCC, 2019).

4.2.1.2 Urban Industrial Area (Block B)

The industrial block was comprised of three land parcels with a total area of 14,027 m^2. Each land parcel had a group of warehouses surrounded by parking areas and green areas as illustrated in figure 4.3. The land use analysis showed that 90 percent of the industrial block was covered with impervious surfaces, and 56 percent of the total area was roof area as illustrated in table 4.2.

Table 4.2 Land use characteristics of the industrial block (Block B)

Land use	**Area (m^2)**	**Area Ratio**
Rooftops	7850	0.56
Pavement Lot	4750	0.34
Green Area	1427	0.10
Total Area	14027	1.00
Total Impervious	5650	0.90
Total Pervious	1350	0.10

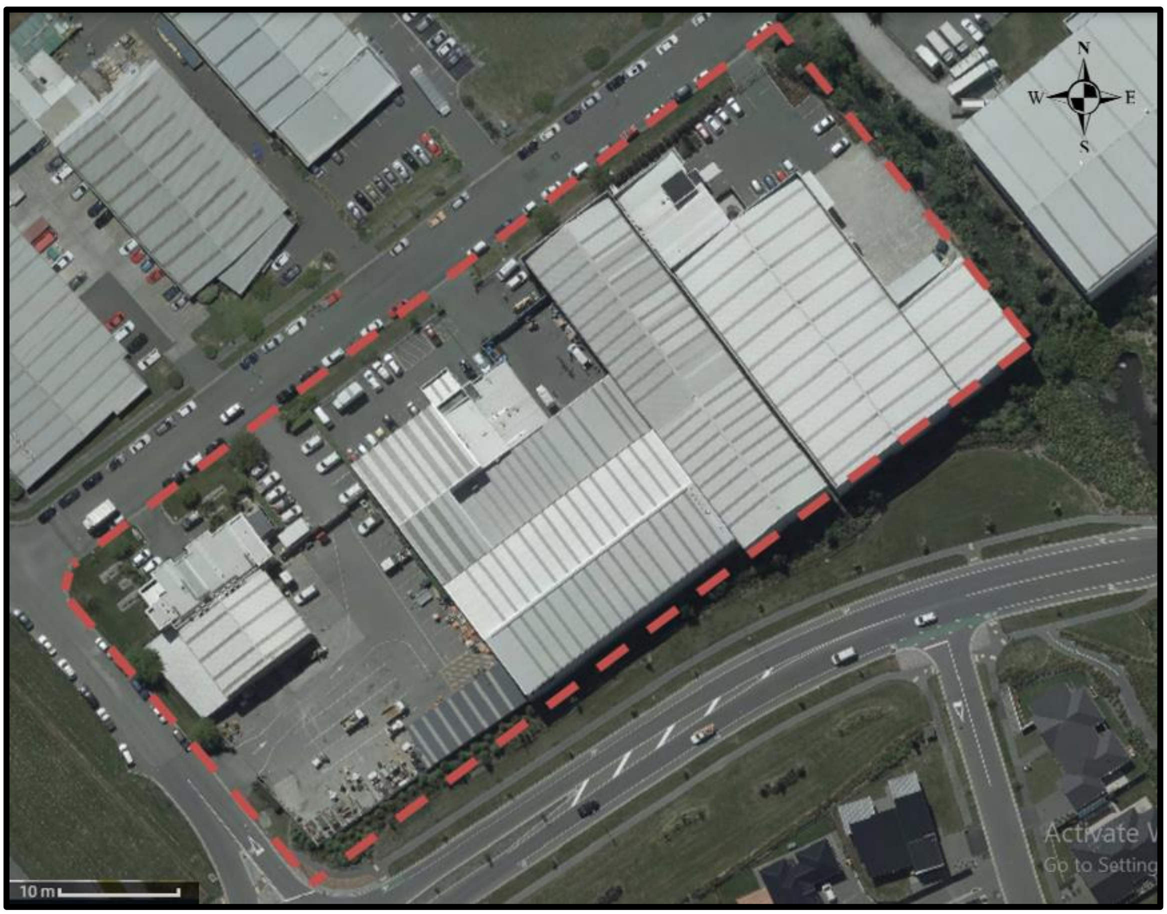


Figure 4.4: Aerial image showing the buildings and boundary of the industrial Block B (CCC,2019).

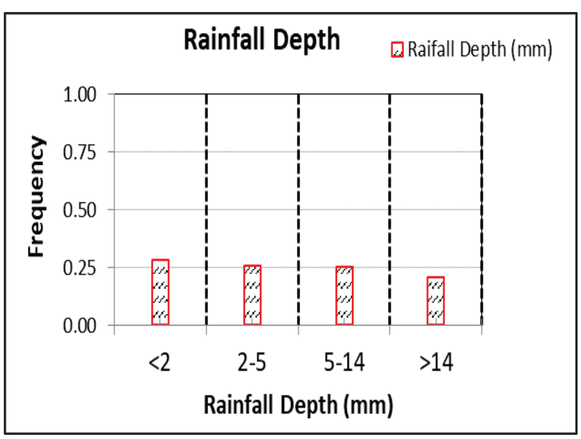
4.2.2 Rainfall Regime

The rainfall data for the selected blocks were provided by Christchurch City Council which collected from the Botanic Gardens rain gauge (43.531°S; 172.619°E) located in Christchurch city 5 km away from the selected blocks. The simulation period started on 4 November 2007 until 4 November 2019, and the rainfall events were filtered to select all the rainfall events with total depth exceeds 1.0 mm and an antecedent dry weather period of 6 h. Thus, 826 rainfall events were selected and used in the performance analysis. Table 4.3 summarizes the characteristics of the selected rainfall data during the simulation period. The frequency of the selected rainfall events was characterised into four categories based on the total rainfall depth and the Annual Recurrence Interval (ARI) in order to evaluate the performance for each category accordingly (Figure 4.4).

Table 4.3 Rainfall events characteristics during the simulation period in Christchurch, New Zealand (2007-2019).

Statistical Data	Annual Depth (mm)	Depth (mm)	Duration (hr)	Rainfall Event Intensity (mm/hr)
Maximum	875.6	150.20	89	67.2
Minimum	450.2	1.00	0.08	2.4
Mean	692.05	9.72	11.49	7.42
Standard Deviation	125.34	13.91	11.81	5.78

Note: the maximum intensity evaluated based on 5-min intervals.



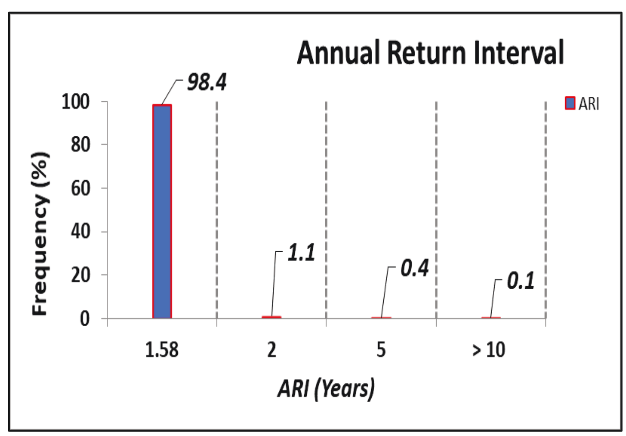


Figure 4.5: The rainfall events characteristics during the simulation period based on four categories of: total rainfall depth; and ARI.

4.2.3 Tanks Capacities and RWH Management Scenarios

The installation of one rainwater tank was assumed for each land parcel in both the RWH-TF and RHW-ST scenarios. Thus, in the residential block, a tank of 5m^3 volume was simulated and connected to the building area in each land parcel, while in the industrial block, a tank of 25m^3 volume was simulated and connected to the building areas in each land parcel. The assumed storage capacities were generally recommended by Hamilton City Council in New Zealand to design and install rain tanks as stormwater control measures in residential areas (HCC, 2016).

Two alternative management scenarios for using the collected rainwater were defined and simulated along with the Business As Usual (BAU) scenario. The RWH-TF scenario evaluated the use of the collected rainwater for toilet flushing uses in the buildings in each land parcel. For each residential house, the demand on the collected rainwater was assumed based on 4 individuals living in the house with three times of toilet flushing per day per inhabitant. In the industrial warehouses, the water demand for thirty workers was assumed (i.e. based on the approximate number of workers) with three toilet flushes per worker per day. Thus, the total daily water demand for toilet flushing was converted into a constant demand rate throughout the day. The calculated constant water demand for toilet flushing in the residential and industrial blocks respectively were 0.0014 L/s and 0.0116 L/s (i.e. 0.12 m^3/day and 1 m^3/day).

The RWH-ST scenario simulated the use of the tanks as stormwater detention and treatment units. Where each rainwater tanks was assumed to detain, treat and slowly discharge treated roof runoff through the filtration units into the stormwater network. Thus, the maximum discharge rate of the filtration units was restricted to 10 L/min in the SWMM models to represent the actual use of filtration units in RWH tanks in this research.

4.2.4 Hydrological and Hydraulic Modelling

The GIS-SWMM modelling of the three scenarios was performed by using the Personal Computer Storm Water Management Model (PCSWMM) which operate the EPA SWMM 5.1.013. PCSWMM supports importing GIS shapefiles and allows using general analysis tools to edit and analyse the imported GIS layers in SWMM models. SWMM is commonly used for the design, analysis and planning of stormwater runoff around the world, which include evaluating conventional stormwater control measures such as pipe and drain networks, and evaluating LID measures and other decentralised stormwater control measures in urban areas (Rossman, 2010). Regarding the hydrologic responses of the selected urban blocks, SWMM allows hydrological and hydraulic simulations at sub-hourly time steps; therefore 5-min time steps were used to analyse the hydrologic and hydraulic performance in the selected blocks throughout the simulation period.

The average daily evaporation per month in Christchurch was used to calculate evaporation losses during the simulation period (NIWA, 2011). The infiltration losses were estimated using the Soil Conservation Service Curve, and a CN value of 70 was assumed for the green areas. Manning's equation was used for all runoff calculations and outflows with an assumed coefficient of 0.012 for the rooftops and pavement areas. The actual boundaries and characteristics of the sub-catchments were analysed using ArcMap and then imported to PCSWMM. Each sub-catchment was identified by single land use and homogenous characteristics for the three scenarios were assumed to ensure consistent and accurate evaluation for the RWH performance as control solutions.

For both the residential and industrial blocks, each PCSWMM model consisted of six sub-catchments that received rainfall data and produced hydrological and hydraulic results such as, outflows, surface runoff, evaporation, infiltration, inflow and storages time series for each component in the sub-catchment. For modelling purposes, each block was linked to an outfall node in the SWMM model to schematize the end of stormwater drainage in each block. This was performed in order to evaluate the hydrological and hydraulic performance of each block (Palla et al., 2017).

Figure 4.5 shows the PCSWMM model schematic for the RWH-TF scenario in the industrial block. All rainwater tanks were simulated as storage nodes and connected to the roof sub-catchments. Then each storage node was equipped with a pump node and a weir link. The pump node represented the water demand for the toilet flushing uses, and the weir link was used to drain excess overflow from the tank into the drainage network (Petit-Boix et al., 2018). The daily water demands were assigned as constant flow rates and connected to the water pumps. Control rules were used to turn on the water pumps if the water depth exceeded 0.01 m^3in the tanks.

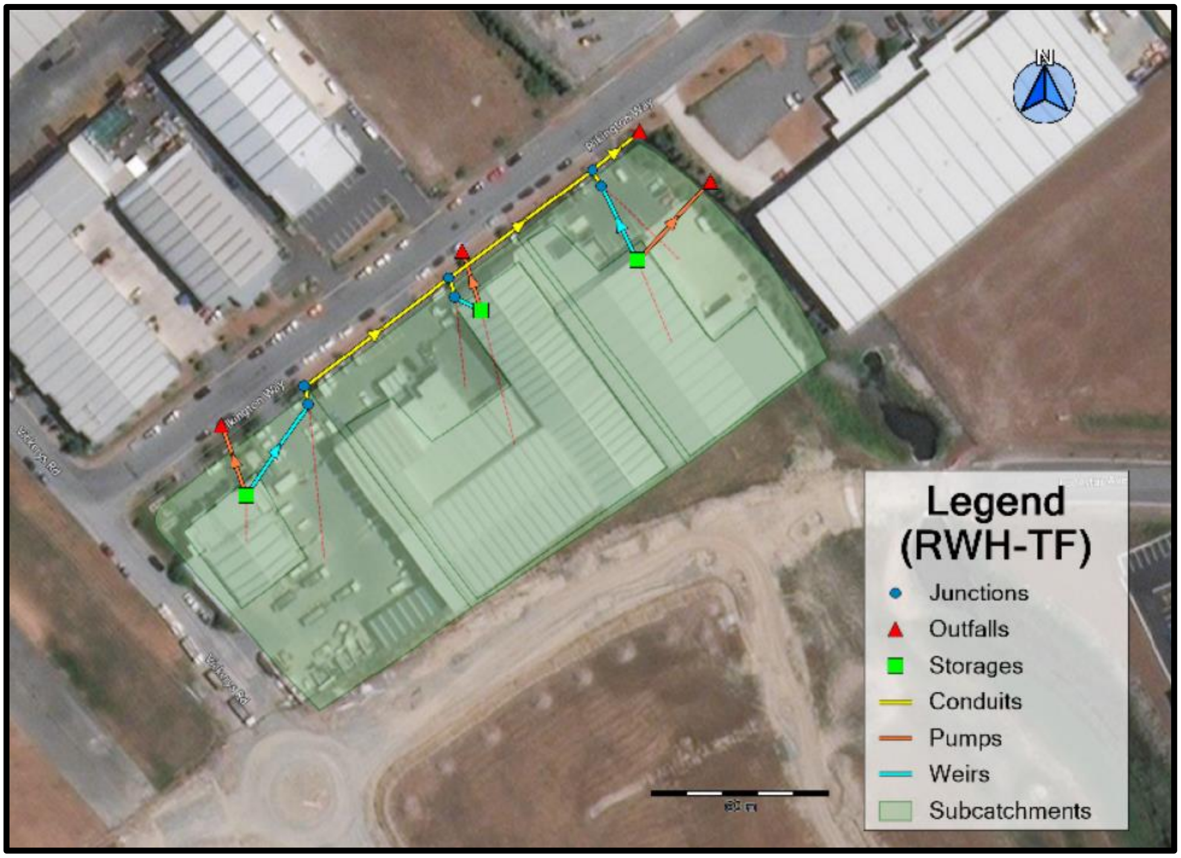


Figure 4.6: PCSWMM model schematic for the RWH-TF management scenario in the industrial block.

Similarly, figure 4.6 shows the PCSWMM model schematic for the RWH-ST scenario in the industrial block. All the rainwater tanks were simulated as storage nodes and connected to the roof sub-catchments. Each storage node was equipped with a weir link at the top of the storage node and an orifice (i.e. valve) at the bottom of the storage node. The weir link was used to drain excess overflow from the storage node into the drainage network, and the orifice represented the use of filtration unit in the RWH tanks. The maximum flow rate of the orifice was restricted to 10 L/min to simulate the actual use of the filtration units, and to ensure high removal performance of zinc from roof runoff.

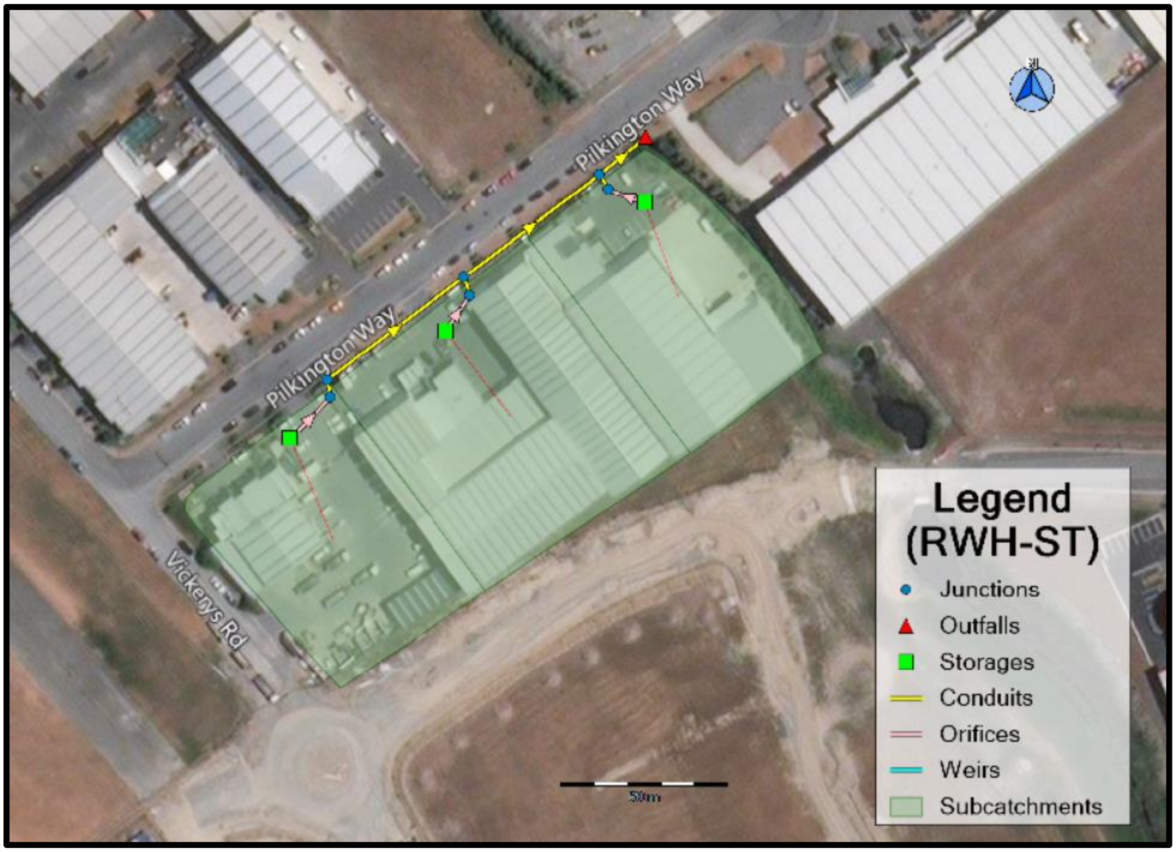


Figure 4.7: PCSWMM model schematic for the RWH-ST management scenario in the industrial block.

4.2.5 Performance Analysis

The BAU scenario was used as the reference to estimate the reductions in stormwater runoff volumes and peak flows during the RWH-TF and RWH-ST scenarios. The simulated management scenarios were assessed based on two performance indicators: the mitigation performance in stormwater runoff, and the tank performance in each scenario.

4.2.5.1 Mitigation Performance

The mitigation performance of stormwater runoff was evaluated for each rainfall event that occurred during the simulation period. Two indices were used to assess the mitigation performance (Palla et al., 2017). The first index was the Peak Reduction (PR) in peak outflows during each rainfall event. The second index is the Volume Reduction (VR) in total volumes of stormwater outflow blocks during each rainfall event period.

The following equations were used to calculate the peak and volume reductions during each rainfall event respectively:

$$Peak\ Reduction = \frac{Peak\ Outlow\ (BAU) - Peak\ Outflow\ (RWH)}{Peak\ Outflow\ (BAU)}\ x\ 100 \qquad \text{(Eq. 4.1)}$$

$$Volume\ Reduction = \frac{Outflow\ Volume\ (BAU) - Outflow\ Volume\ (RWH)}{Outflow\ Volume\ (BAU)}\ x\ 100 \qquad \text{(Eq. 4.2)}$$

The volume reduction, and peak reduction are calculated over the duration of each rainfall event for the BAU, RWH-TF, and RWH-ST scenarios.

4.2.5.2 Tank Performance

Previous studies have defined several non-dimensional parameters to evaluate the rainwater tank performance (Palla et al., 2017), however the use of each parameter depends primarily on the main purpose of rainwater tanks in each management scenario.

During the RWH-ST scenario, collecting and treating roof runoff through the filtration unit was the main objective of the tank, therefore the Overflow Ratio (OFR) index was used (Palla et al., 2017) to evaluate tank performance to collect and treat roof runoff before water overflow from tanks during rainfall events. The OFR was estimated as the ratio of excess flow (L^3) and inflow (L^3) of the tank at each time step (i.e. 1-min) throughout the simulation period (Eq. 4.3).

$$OFR = \frac{\sum_1^N E}{\sum_1^N Q} \qquad \text{(Eq. 4.3)}$$

Where:

E= excess flow from the tanks at each time step (L);

Q = water inflow into the tank at each time step (L);

N = total time steps (1-min).

During the RWH-TF scenario, supplying water for toilet flushing was the main objective of the tank, therefore the Water Saving (WS) index was used (Campisano et al., 2014) to evaluate the tank efficiency to maintain water supply for the assumed demand of toilet flushing throughout the simulation period. The WS index was estimated as the ratio of the yield water volume in the tanks (L) and water demand (L) at each time step of the simulation period (Eq. 4.4).

$$WS = \frac{\sum_1^N S}{\sum_1^N D} \qquad \text{(Eq. 4.4)}$$

Where:

S= yield volume in tanks at each time steps (L);

D = water demand volume at each time step (L);

N = total time steps (1-min).

4.2.6 Validation and Calibration

In the absence of actual stormwater outflow data to calibrate and validate the BAU scenario, the following steps were used to estimate the reductions in stormwater runoff volumes and peak flows (Petit-Boix et al., 2018). First of all, all SWMM models were simulated based on actual parameters such as roof areas, spatial locations, slopes, diameters and depths of the stormwater pipes. Secondly, in each SWMM model, the Runoff and Flow Continuity Error indicators were estimated to be <1%. The Runoff Error in SWMM represents the continuity of water in the model which is calculated as the percentage difference between the total precipitation, comparing to surface runoff, final storage, and total losses (i.e. evaporation, infiltration) in each subcatchment. The Flow Error in SWMM represents the continuity of flow in the model which is calculated as the percentage difference of between total of inflow (i.e. dry and wet conditions), and total outflow (e.g. flooding) in each component of the model throughout the simulation period.

Finally, the following simple methodological approaches were defined to compare between the outflow volumes in the RWH-ST, RWH-TF and BAU scenarios during the simulation period as per the following balance equations:

1) Total outflow volumes during the BAU scenario = Reference Outflow Volumes (ROV)

2) ROV = Outflow (RWH-TF) + Water stored in tanks + Water used in toilet flushing.

3) ROV = Outflow (RWH-ST) + Water stored in tanks + Water passed through filtration units.

There was no difference in the result of these equations throughout each time step of the simulation period which validates the simulation results of RWH-ST and RWH-TF scenarios comparing to the BAU scenario. Such a simple methodological approach was used to accurately assess the impact of using the rainwater tanks to mitigate stormwater runoff in urban residential areas (Palla et al., 2017).

4.3 Results and Discussion

This section presents the simulation results and discussion of the findings in the selected residential and industrial blocks. The simulation results include outflow hydrographs for the three simulated scenarios over the simulation period at 5-min time steps, and a group of 1-min time series for parameters related to the tank performance such as water storage volumes, water demand, inflow and outflow rates along with other several parameters.

4.3.1.1 Mitigation Performance - Residential Land Use

Figures 4.7 and 4.8 illustrate the mitigation performance during the three simulated scenarios in the residential block throughout the simulation period with an example of the performance for one rainfall event occurred on 17 September 2012. This rainfall event is characterised by a total rainfall depth and duration of 10.8 mm and 3.33 h respectively, and classified with 1.58 ARI which represented more than 98 percent of the rainfall events occurred during the simulation period according to the HIRDS V4 Depth-Duration-Frequency curves in Christchurch (NIWA, 2019). Similarly, figure 4.8 shows the level of water volumes in the tanks in throughout the simulation period and for the selected rainfall event. The peak and volume reductions are calculated over the event duration for the three scenarios.

The graph shows a clear reduction in the peak outflow during the RWH-ST scenario that was estimated at 56%, while the RWH-TF showed an insignificant reduction in the peak flow was estimated at 0.9% comparing to the BAU scenario. Approximately one hour after the start of the rain, all the tanks reached full storage capacity and the hydrological response started to mimic the hydrological response of BAU scenario as shown in Figure 4.7, whereas the tanks did not reach the full storage capacity during the RWH-ST scenario throughout the rainfall event. Figure 4.8 shows the water levels in each tank (i.e. each tank represent one land parcel) during the rainfall event for both the RWH-ST and RWH-TF scenarios.

The reductions in stormwater volumes during the RWH-ST and RWH-TF scenarios were estimated at 42% and 21% respectively. The volume reduction in stormwater outflow during the RWH-ST scenario was due to the roof runoff volume that was detained in the tanks during the rainfall event (i.e. by draining the collected roof runoff volume slowly through the filtration unit).

Similarly, in the RWH-TF scenario, the reduction in stormwater volumes was due to the roof runoff volume that was captured in the tanks in addition to the water volume that was used for toilet flushing during the rainfall event.

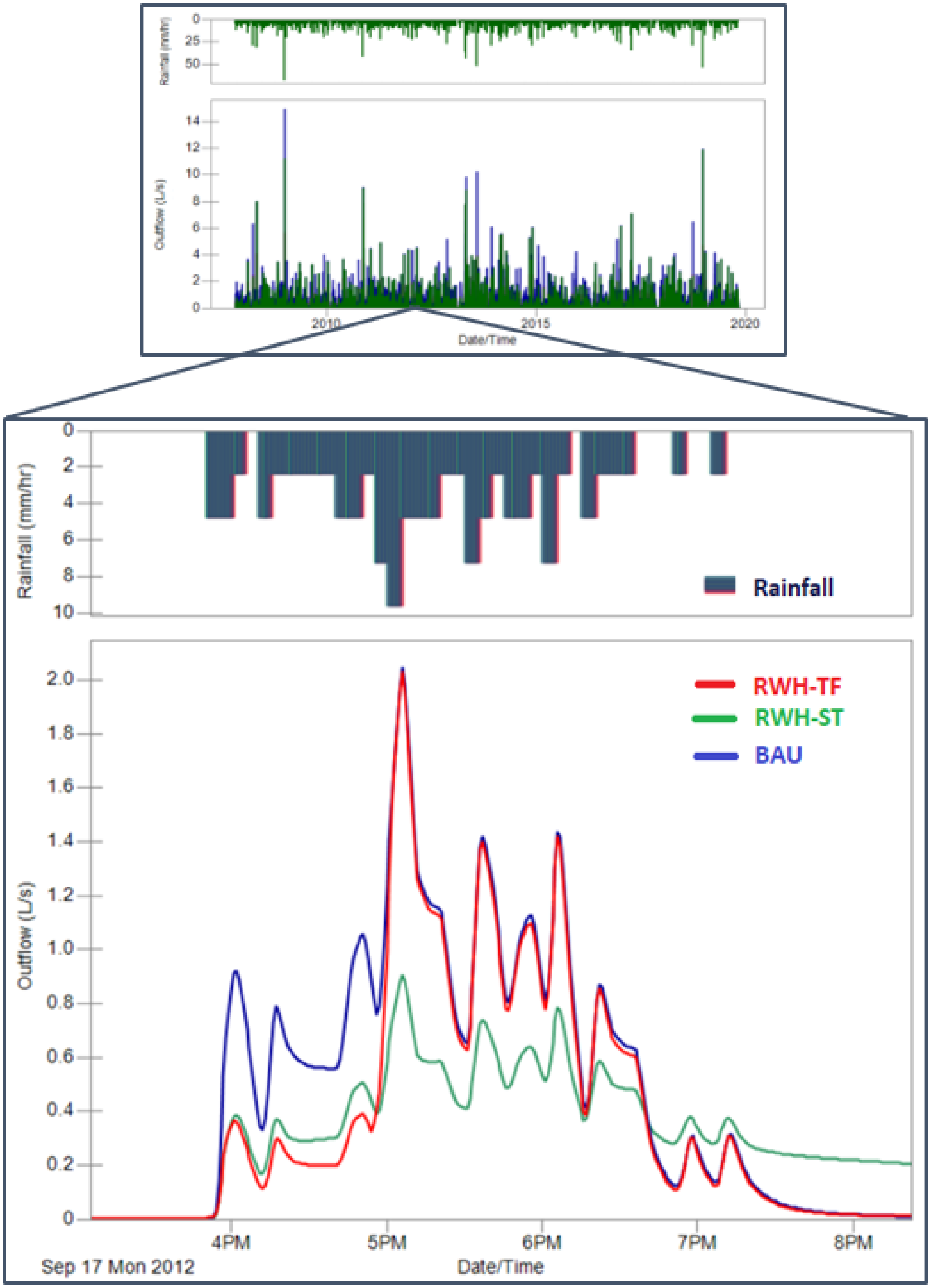


Figure 4.8: The hyetograph and the simulated hydrographs for the BAU, RWH-TF, and RWH-ST in the residential block including the 17th of September 2012 rainfall event.

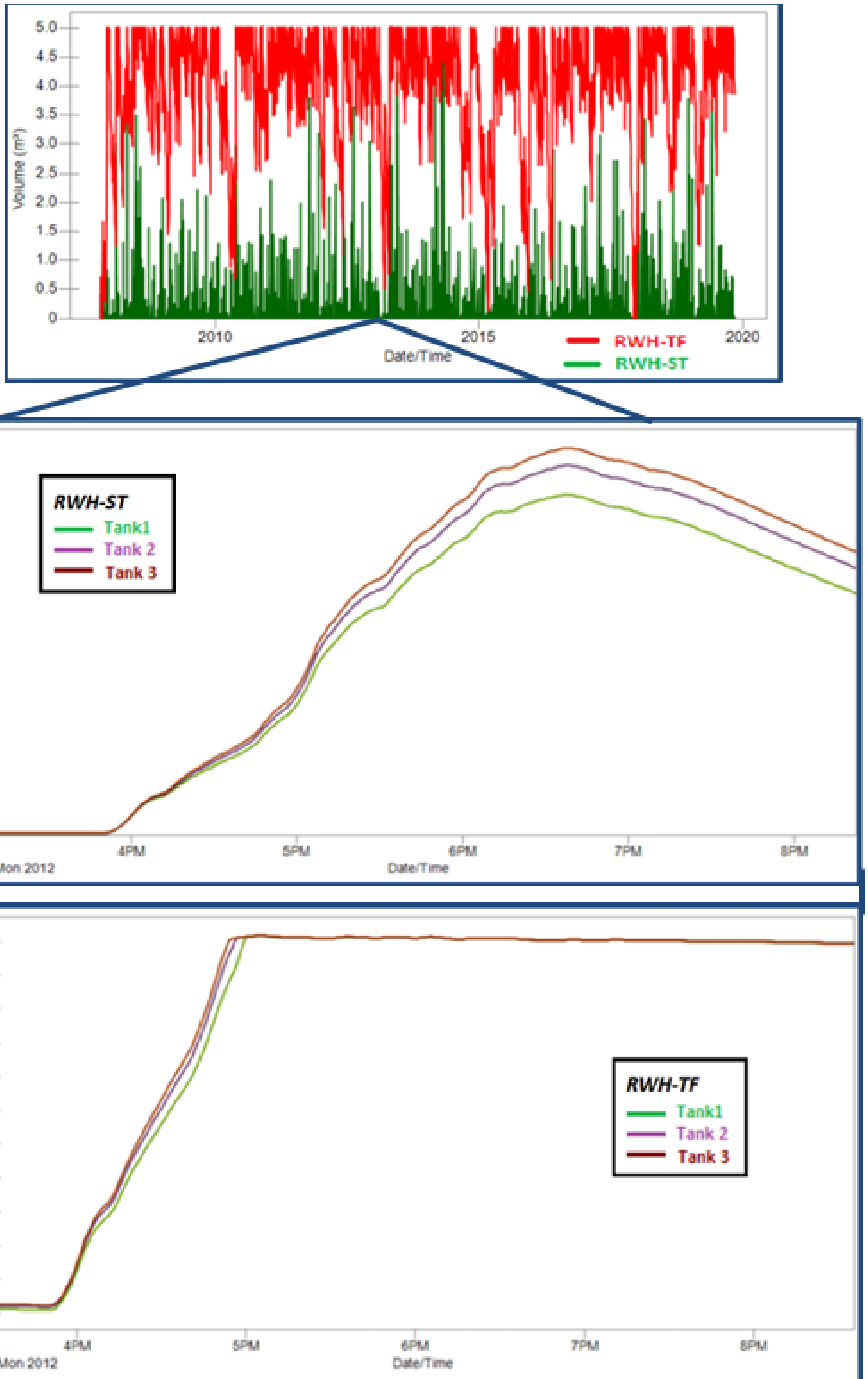


Figure 4.9: The storage volumes in the residential block for the simulated the BAU, RWH-TF, and RWH-ST scenarios including the 17th of September 2012 rainfall event.

4.3.1.2 Mitigation Performance - Industrial Land Use

Similarly, figure 4.9 illustrates the mitigation performance of the three management scenarios in the industrial block throughout the simulation period with an example of the mitigation performance for a rainfall event occurred on 15 of August 2011. The selected rainfall event was characterised by a total rainfall depth and duration of 12.4 mm and 12.92 h respectively, and classified with 1.58 ARI category (NIWA, 2019). Figure 4.9 shows a clear reduction in the peak flow during the RWH-ST scenario estimated at 44%, while the RWH-TF showed lower reductions in the peak flow estimated at 8% comparing to the BAU scenario.

In term of the volume reductions of outflow volumes during the RWH-ST and RWH-TF scenarios were estimated at 33% and 13% respectively. The volume reduction in stormwater outflow during the RWH-ST scenario was due to the roof runoff volume that was detained in the tanks during the rainfall event (i.e. by draining the collected roof runoff volume slowly through the filtration unit). Similarly, in the RWH-TF scenario, the reduction in stormwater volumes was due to the roof runoff volume that was captured in the tanks in addition to the water volume that was used for toilet flushing during the rainfall event.

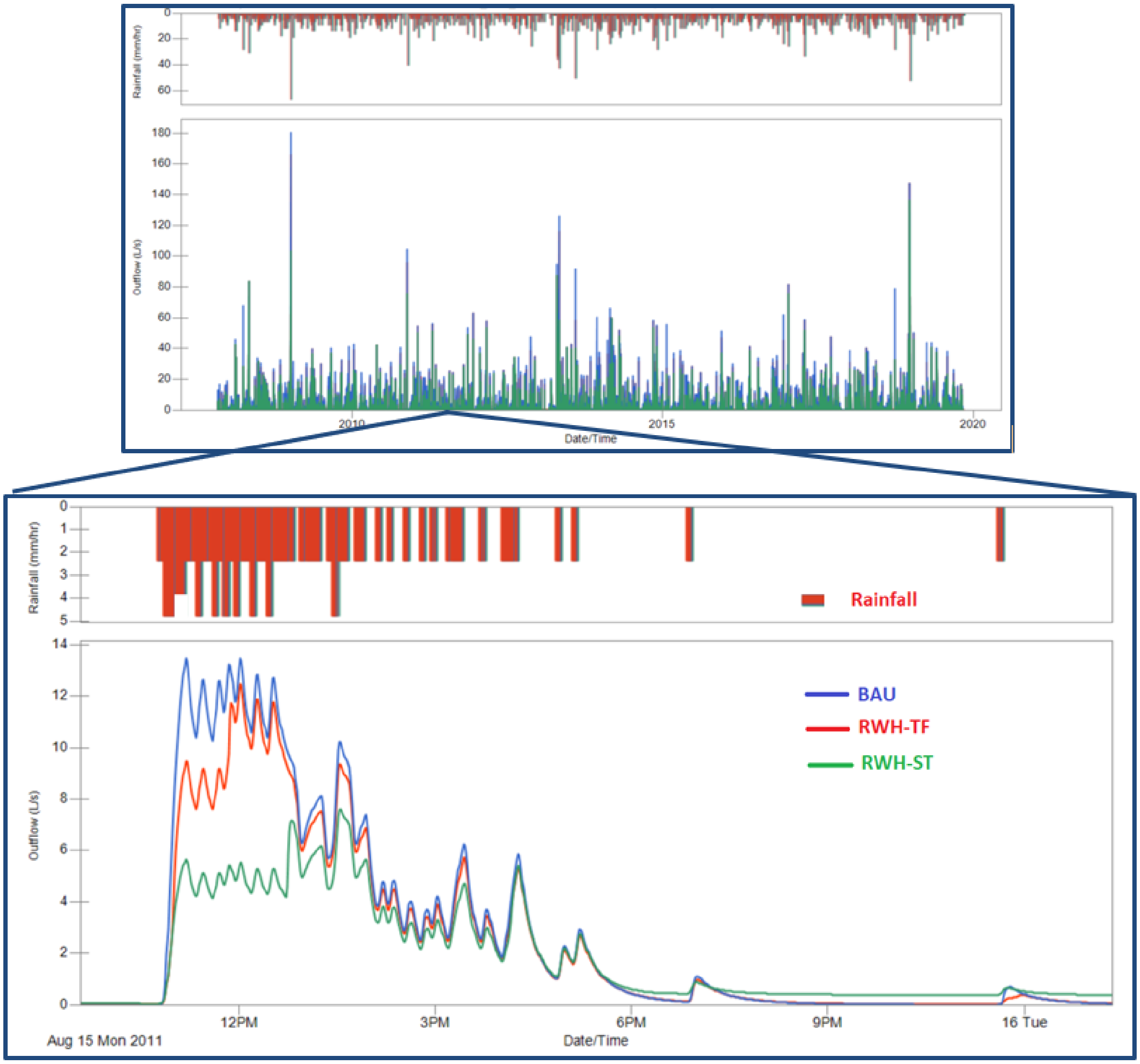


Figure 4.10: The hyetograph and the simulated hydrographs for the BAU, RWH-TF, and RWH-ST in the industrial block including the 15th of August 2011 rainfall event.

Figure 4.10 shows the water volumes in each tank during the rainfall event. As the rooftop areas in each land parcel relatively varied in the industrial block, the tanks showed different timing for reaching full storage capacities during the rainfall event period. Figure 4.10 also shows that the tanks had different water levels at the beginning of rain (i.e. residue of water in the tanks from previous rainfall events) which affected the reduction performance of tanks.

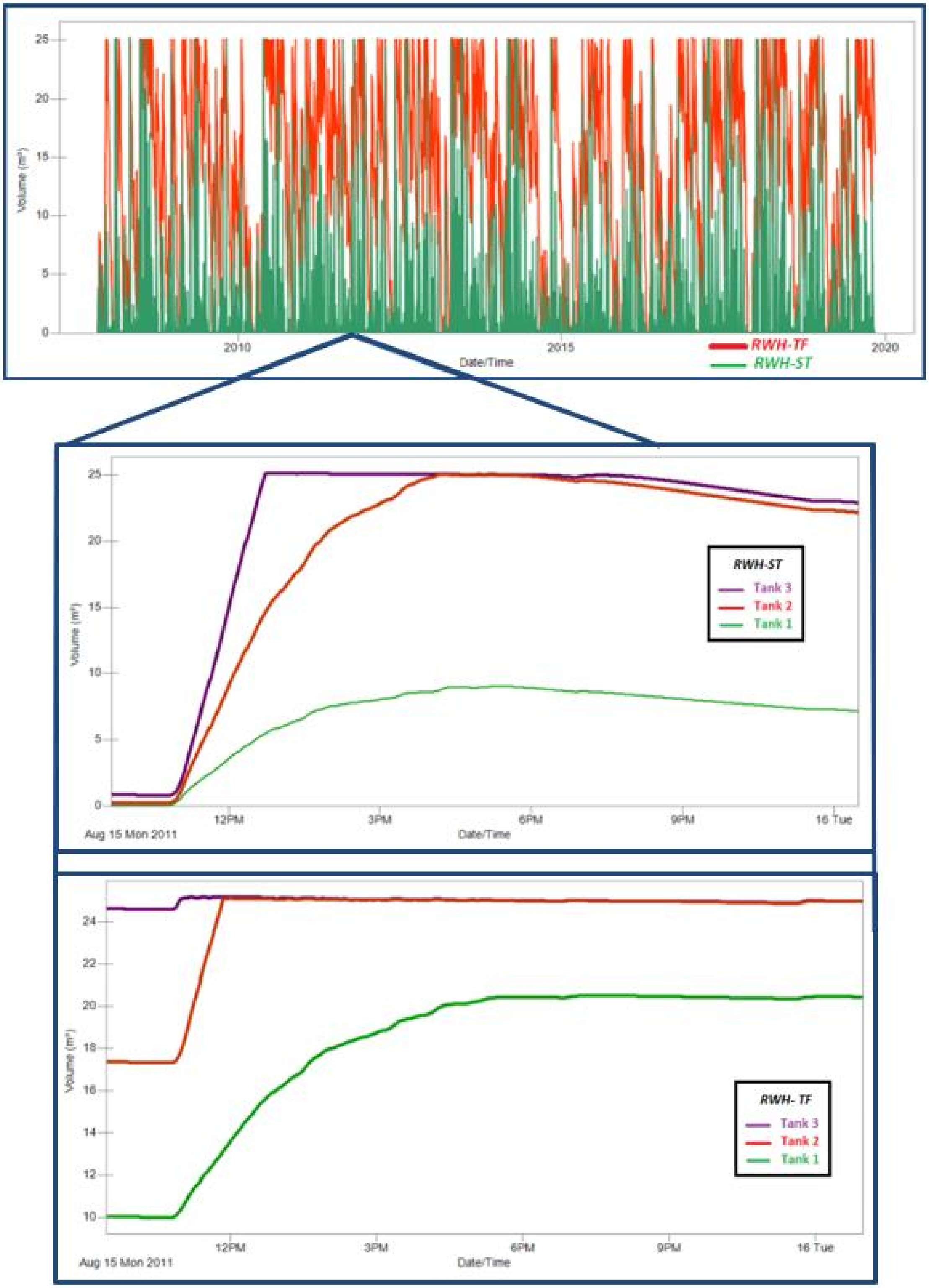


Figure 4.11: The storage volumes of the BAU, RWH-TF, and RWH-ST scenarios in the industrial block including the 15th of August 2011 rainfall event.

4.3.2 Overall Mitigation Performance

The simulation results of the three scenarios confirmed the ability of rainwater tanks to mitigate stormwater runoff in both the residential and industrial land uses. The average reductions in peaks and total volumes of stormwater runoff for the simulated rainfall events in the residential and industrial blocks are listed in tables 4.4 and 4.5, respectively.

In general, the simulation results of RWH-ST scenario showed higher peak reductions compared to the RWH-TF with averages estimate at 52.9% and 45% in the residential and industrial blocks respectively. Similarly, the RWH-ST scenario showed effective detention performance during the rainfall event period with average volume reductions in outflows estimated at 37.3% and 19.2% for the industrial and residential blocks respectively. The reduction in outflow volumes during the RWH-ST scenario represented the runoff volume that was detained in the tanks during the rainfall event period due to the restricted discharge of 10L/min (i.e. through the filtration unit).

The reason of variation in mitigation performance between the industrial and residential blocks is due to the larger outflow volumes from industrial roof areas compared to the residential roof areas which fill up the tanks in shorter time during rainfall events. Furthermore, the total outflow volume in the industrial block is relatively larger, and the restricted discharge out the filtration unit comprises an insignificant share of the total outflow volume. To validate this finding, another scenario of the RWH-ST-L was simulated in the residential block with a lower restricted discharge rate out the filtration unit (i.e. 5L/min) during the simulation period. The simulation results of the RWH-ST-L scenario showed higher mitigation performance in the residential block for both the peak and total volume comparing to the RWH-ST as illustrated in table 4.4. These reduction in peaks and outflow volumes were calculated for the duration of each rainfall event throughout the simulation period.

Table 4.4 Average PR and VR in the residential land use during the simulated period.

Residential Land Uses (Block A)			
Scenario	**RWH-TF**	**RWH-ST**	**RWH-ST-L***
Average Peak Reduction	36.3%	52.9%	55.5%
Average Volume Reduction	42.9%	19.2%	28.4%

**Note: in RWH-ST-L scenario the discharge from the filtration unit was restricted to 5 L/min.*

Table 4.5 Average PR and VR in the industrial land use during the simulated period.

Industrial Land Uses (Block B)		
Scenario	**RWH-TF**	**RWH-ST**
Average Peak Reduction	23.9%	45.0%
Average Volume Reduction	27.5%	37.3%

In general, the peak and volume reductions decreased as the total rainfall depths, and ARI categories increased for all the simulated scenarios. In both the industrial and residential blocks, the mitigation performance of the RWH-TF and RWH-ST scenarios were effective for all the rainfall depth categories that have similar ARI categories (i.e. extended rainfall events).

Both the RWH-ST and RWH-TF scenarios showed effective mitigation performance during rainfall events lower ARI categories; however, the mitigation performance varied between the industrial and residential land uses during higher ARI categories. The RWH-ST scenario showed effective peak reductions in the residential block for rainfall events with up to 5 ARI, while showed relatively low peak reductions in the industrial block for similar rainfall events with 5-ARI category. The reason for this variation is perhaps due to the combination of the tank sizes and runoff filling rates. For example, the tanks were filled up in shorter periods in the industrial block comparing to the residential block for similar rainfall events due to the larger rooftop areas in the industrial block.

To highlight the impact of rainfall events characteristics on the mitigation performance, the rainfall events were classified into four categories based on the total rainfall depths and the ARI.

Figure 4.11 and 4.12 presents the peak and outflow volume reductions in the residential and industrial blocks respectively during each category. Similarly, the reductions in outflow volumes during the RWH-ST scenario represented the runoff volume detained by the tanks due to the restricted discharge through the filtration unit during the rainfall event period, whereas the volume reduction during the RWH-TF scenario represented the runoff volume detained in the tanks in addition to the water used for toilet flushing. These reduction in peaks and outflow volumes were calculated for the duration of each rainfall event.

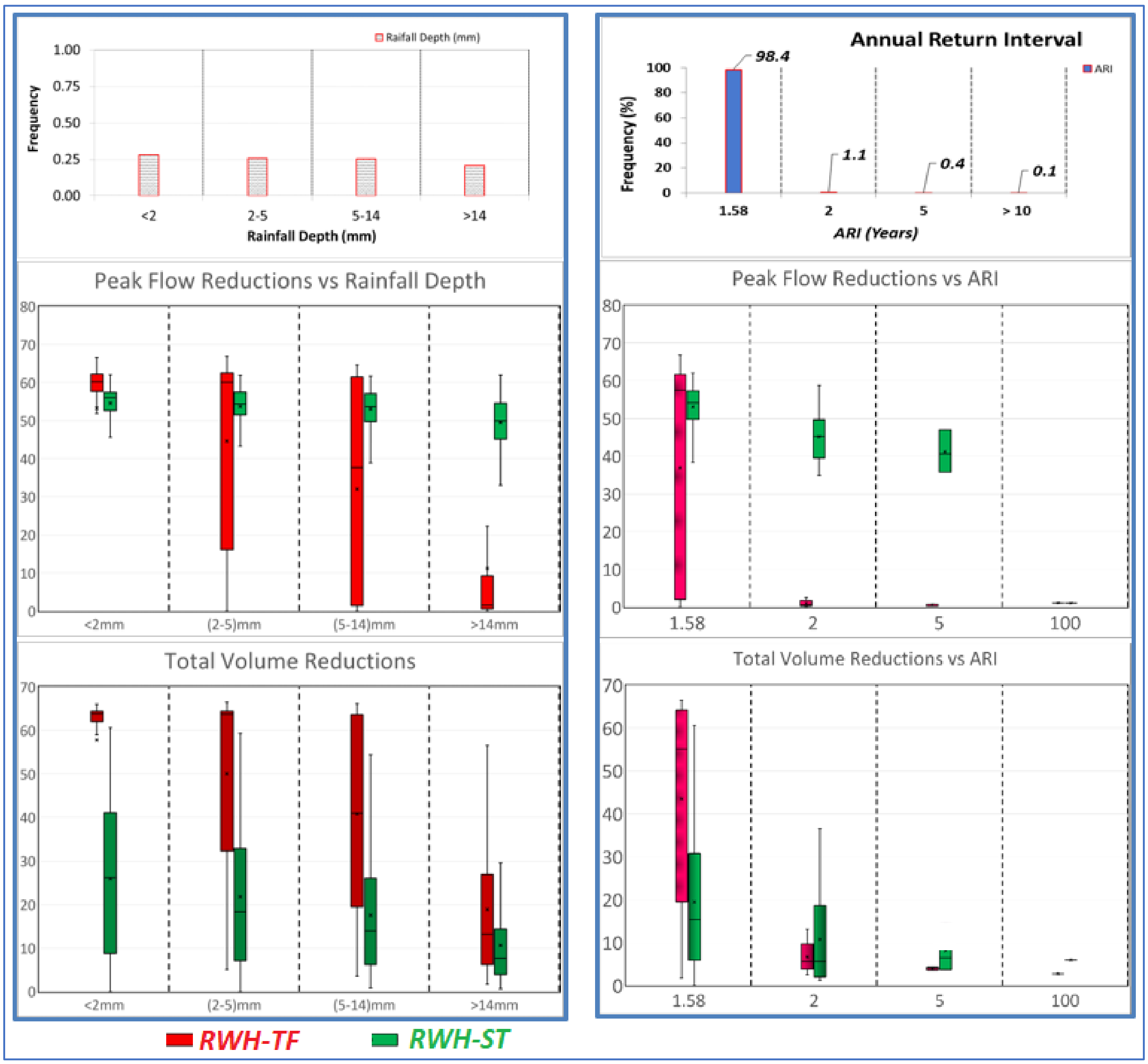


Figure 4.12: Non-parametric distribution of the reductions in peaks and total volumes of stormwater runoff for the RWH-TF and RWH-ST scenarios in the residential block based on total rainfall depths and ARI categories.

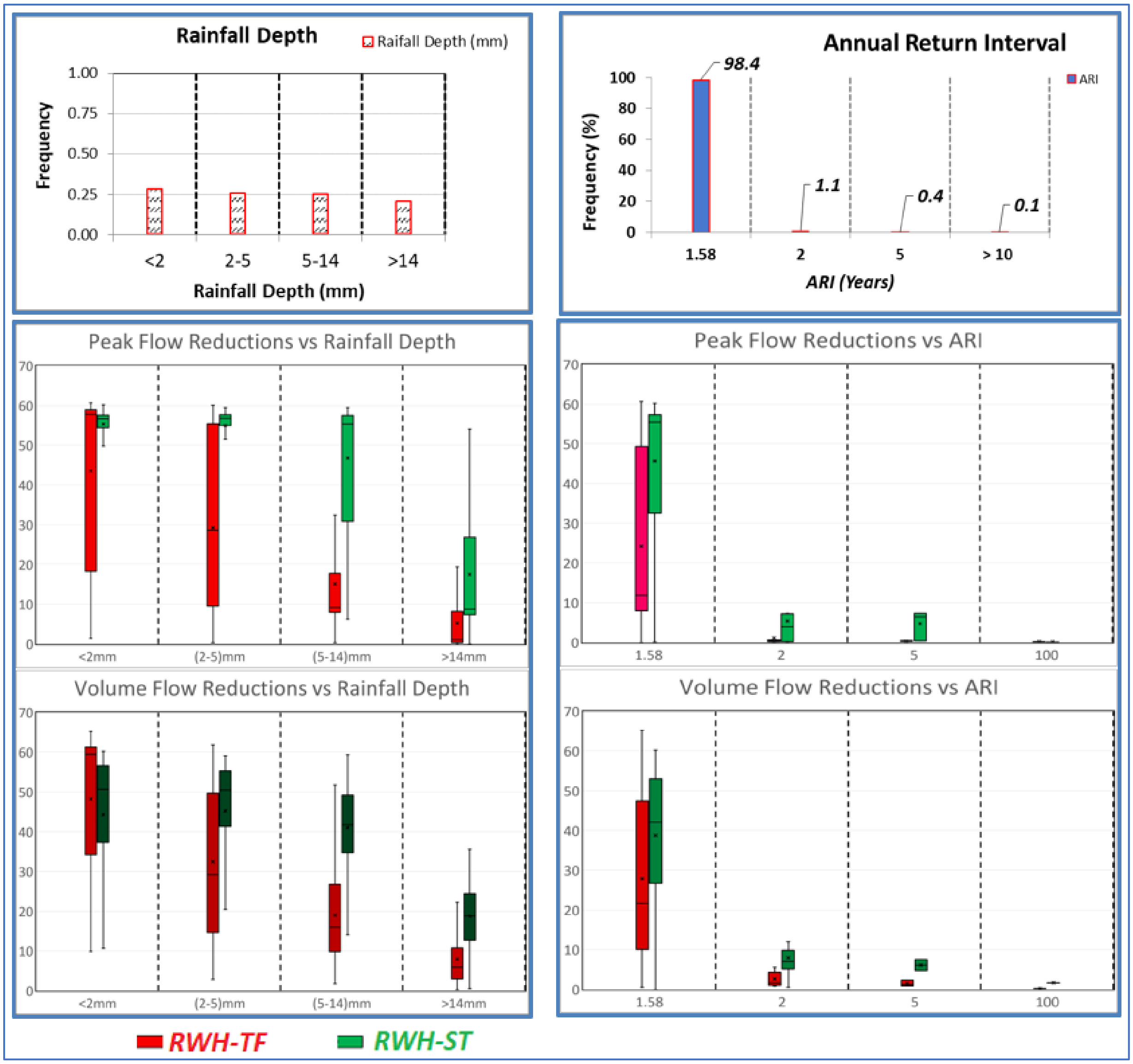


Figure 4.13: Non-parametric distribution of the reductions in peaks and total volumes of stormwater runoff for the RWH-TF and RWH-ST scenarios in the industrial block based on total rainfall depths and ARI categories.

4.3.3 Tank Performance

The performance of tanks during the RWH-ST scenario was assessed based on the OFR index during the simulation period in the residential and industrial blocks. In general, the selected tank sizes showed effective treatment performance for roof runoff. The average OFRs were estimated to <1% and 42% in the residential and industrial blocks respectively, which indicate that throughout the simulation period, more than 99% and 58% of the roof runoff passed through the filtration units to remove zinc volumes in the residential and industrial blocks respectively. The OFRs were calculated based on the assumed restricted flow rate of 10 L/min through the filtration unit, however using lower filtration rate in actual integrated RWH tanks would result in higher removal performance of zinc as well as higher mitigation performance for similar storage volumes.

The performance of rainwater tanks during the RWH-TF scenario was assessed by the WS index. In general, the tanks showed reliable sources of water supply for the assumed demand of toilet flushing. The averages of WS ratio in the residential and industrial blocks were estimated to be 52% and 83%, respectively. Previous research demonstrated similar water-saving efficiencies and linked that primarily to the tanks size and the rainfall patterns (Palla et al., 2011). The average OFRs of the residential and industrial blocks estimated at 64% and 72% during the 12-year period. This indicated that only 36% and 28% of the roof runoff volumes were used to supply water for the assumed toilet flushing demands.

In general, the RWH tanks showed satisfactory performance in both management scenarios, even though the tanks sizes were not specifically designed for each building requirements. Thus, using these results as guidelines to design site-specific rainwater tanks would improve the mitigation performance, as well as the tank's performance in each management scenario.

4.4 Chapter Summary

This chapter presented the GIS-SWMM modelling, the associated simulation results and discussion of the findings. The modelling was carried out to evaluate the mitigation performance of the RWH tanks in residential and industrial land use in Christchurch, New Zealand. Two urban blocks located in the Wagram suburb were selected and simulated to assess the mitigation performance and the performance of tanks under two proposed management scenarios.

The BAU, RWH-TF, and RWH-ST management scenarios were simulated throughout a 12-year period starting from 4 November 2007 until 4 November 2019. The BAU was simulated to represent the reference for the actual outflow volumes and peak flows of the two selected blocks. The RWH-TF scenario was simulated to represent the use of rainwater tanks to supply water for toilet flushing uses in the buildings in each block, while the RWH-ST scenario simulated to represent the use of rainwater tanks with filtration units as stormwater detention units that detain, treat, and slowly discharge the collected roof runoff into the stormwater network under a restricted flow rate of 10L/min to represent the actual use filtration units in rainwater tanks. The rainwater tank performance and the mitigation performance were assessed for each rainfall event during the simulation period.

The simulation results showed effective performance to mitigate stormwater runoff in urban residential and industrial land use by using rainwater tanks. The average reductions in peak outflows during the RWH-ST scenarios were estimated at 52.9% and 45% in the residential and industrial blocks respectively. Similarly, the RWH-ST scenario showed average volume reductions of 37.3% and 19.5% for the industrial and residential blocks respectively. The RWH-TF scenarios showed peak reductions estimated at 36.3% and 23.9% for the residential and industrial blocks respectively. Likewise, the RWH-TF scenario effective showed average volume reductions estimated at 42.9% and 27.5% for the industrial and residential blocks respectively.

In both RWH-ST and the RWH-TF scenarios, the mitigation performance decreased as the ARI category of rainfall events increased, however the RWH-ST and the RWH-TF showed effective mitigation performance for rainfall events with up to 5-ARI category. The mitigation performance showed consistent and effective performance for all total rainfall depths categories that have similar ARI categories (i.e. extended rainfall events with similar ARI category).

The integration between the rainwater tanks and filtration unit in the RWH-ST scenario showed an effective treatment performance with more than 99%, and 58% treatment ratio for the collected roof runoff in the residential and industrial blocks respectively. The assumed restricted 10L/min filtration rate through the filtration unit would results in 80% average removal efficiency of dissolved zinc from roof runoff as per the findings of chapter 3. Yet, reducing the discharge rate through the filtration unit (i.e. by using valves) would improve both the removal efficiency of zinc and the mitigation performance of stormwater runoff. This was confirmed during the RWH-ST-L scenario which demonstrated higher peak and outflow volume reductions by using a restrict filtration rate of 5 L/min in the residential block.

The rainwater tank sizes have a direct impact on the water saving efficiency and the OFR index. The selected tank sizes showed sufficient levels of to supply for the assumed demand of toilet flushing for 52% and 83% of the simulated period in the residential and industrial blocks respectively. However, selecting the tank storage capacity based on site-specific conditions would result in better water saving efficiency and lower overflow rates.

Finally, in future integrated RWH tanks, the optimal performance of tanks could be achieved by combining the benefits of water supply, roof runoff treatment, and the mitigation of stormwater runoff as per the proposed integrated tanks in figure 4.13.

Chapter 5: Conclusion and Recommendations

The main aims of this thesis were to investigate the potential use of TMS and UTMS to in the rainwater harvesting tanks remove zinc from roof runoff near to the pollution sources, and to evaluate the potential use of RWH tanks as stormwater control measures to mitigate stormwater runoff volumes and peak flows at urban residential and industrial scales.

5.1 The potential use of TMS and UTMS to remove zinc from roof runoff

The first phase of this thesis investigated the removal performance of zinc from roof runoff by using treated and untreated mussel shells. Both the TMS and UTMS showed effective removal performance during relatively short contact times with water. The abundant volumes of mussel shell waste in New Zealand, provide a promising opportunity to turn this industrial waste into a product that can be used in water treatment. The observed effective removal performance of mussel shell waste could be further improved by using different treatment methods such as heat treatment. This was confirmed as the TMS showed a noticeable improvement in the removal performance of zinc. Further research is required to investigate the use of different approaches of heat treatment with different particle sizes of mussel shell to improve its ability to remove dissolved metals from water.

The removal performance of zinc was directly related to the contact time with water. This was observed as the removal performance of zinc decreased as the flow rates through the TMS and UTMS increased. In the same context, the removal performance increased as the depth of filtration media increased for similar controlled flow rates. The TMS showed greater changes in pH levels which resulted in higher removal performance comparting to the UTMS. However, in actual filtration units, to counteract the change in pH levels, a buffering media can be added beneath the filtration media to balance the change in pH levels before discharging into the stormwater network.

The hydraulic conductivity of the selected particle size ranged between 80 - 110 m/h, and the maximum saturated flow rate through the used filtration units was estimated at an average flow of 17 L/min. The method of having PVC valves to control flow rates through the filtration media worked well to maintain a consistent high removal performance of zinc for each flow rate category. Further research is required to investigate the performance of the filtration media under continuous flow conditions during longer time periods (i.e. days, months) to determine the effectiveness of the TMS and UTMS to remove metals overtime and to determine the breakpoint of the filtration media. Note that the proposed testing configurations in this thesis can be used to run future similar experiments for longer periods of time and with different flow rates.

Unfortunately, due to the limited time and resources, the microstructure and other chemical characteristics of the mussel shell were briefly assessed using the SEM and EDS tests only, however in future research, detailed characterizations for the microstructures and the chemical compositions of the TMS and UTMS should be considered. This include using the X-ray diffraction (XRD) tests to identify the crystalline phases in the shell and to determine the chemical compositions and compounds of the TMS and UTMS.

5.2 The use of rainwater tanks to mitigate stormwater runoff

The second phase of this thesis evaluated the use of rainwater tanks to mitigate stormwater runoff at residential and industrial scales in urban areas. GIS-SWMM modelling was performed to simulate the use of rainwater tanks in two selected urban blocks which represented residential and industrial land uses in Christchurch, New Zealand.

Two management scenarios to use the collected rainwater were defined and simulated over a 12-year period starting from 4 November 2007 until 4 November 2019. The RWH-TF scenario was simulated to represent the use of rainwater tanks to supply water for toilet flushing uses in the buildings for each selected block. The RWH-ST scenario was simulated to represent the use of rainwater tanks with filtration units as stormwater detention units that detain, treat, and slowly discharge the treated roof runoff into the stormwater network with a filtration rate of 10L/min to represent the actual use of filtration unit in rainwater tanks.

Rainwater tanks in both the RWH-TF and RWH-ST showed effective reductions in peak and total volumes of stormwater runoff in the residential and industrial blocks throughout the simulation period. The ability of rainwater tanks to mitigate stormwater runoff decreased as the ARI categories of rainfall events increased, and the RWH tanks showed effective mitigation performance for rainfall events with ARI up to 5 years. The rainwater tanks showed consistent mitigation performance for the extended rainfall events with similar ARI categories, in particular during rainfall events with low ARI category (i.e. <2 years) which represented more than 98% of the rainfall events occurred during the simulation period.

The integration between rainwater tanks and filtration unit in the RWH-ST scenario showed effective performance to capture and treat roof runoff in residential and industrial urban areas. This was based on the restricted filtration rate of 10L/min and the assumed tank sizes, however future research should investigate the use of lower filtration rates (e.g. 1,3, 5 L/min) along with site-specific design for tank sizes which would substantially improve the mitigation performance as well as the treatment performance.

The optimal use of rainwater tanks in urban areas should include upper and lower storage volumes as illustrated in figure 4.13. The upper storage (i.e. detention) capacity should drain the collected runoff slowly through filtration units to remove dissolved contaminants from roof runoff in order to improve the quality of stormwater runoff and to protect the associated ecological conditions of urban waterways., The lower storage (i.e. re-use) capacity can be used to maintain the non-potable water demands in each site. Future research should also include the cost analysis of installing rainwater tanks at residential and industrial scales in urban areas.

Mussel shell waste, as filtration media, consistently showed high removal efficiency for dissolved zinc from roof runoff. This provides a promising opportunity to recycle an abundant and cost-effective waste product from the shellfish industry into the stormwater treatment industry for similar stormwater control measures. Furthermore, the use of rainwater tanks as a stormwater control measure provides potential benefits to improve the quality of stormwater runoff and reduce peak runoff flows near the sources in urban areas with a low-intensity rainfall climate.

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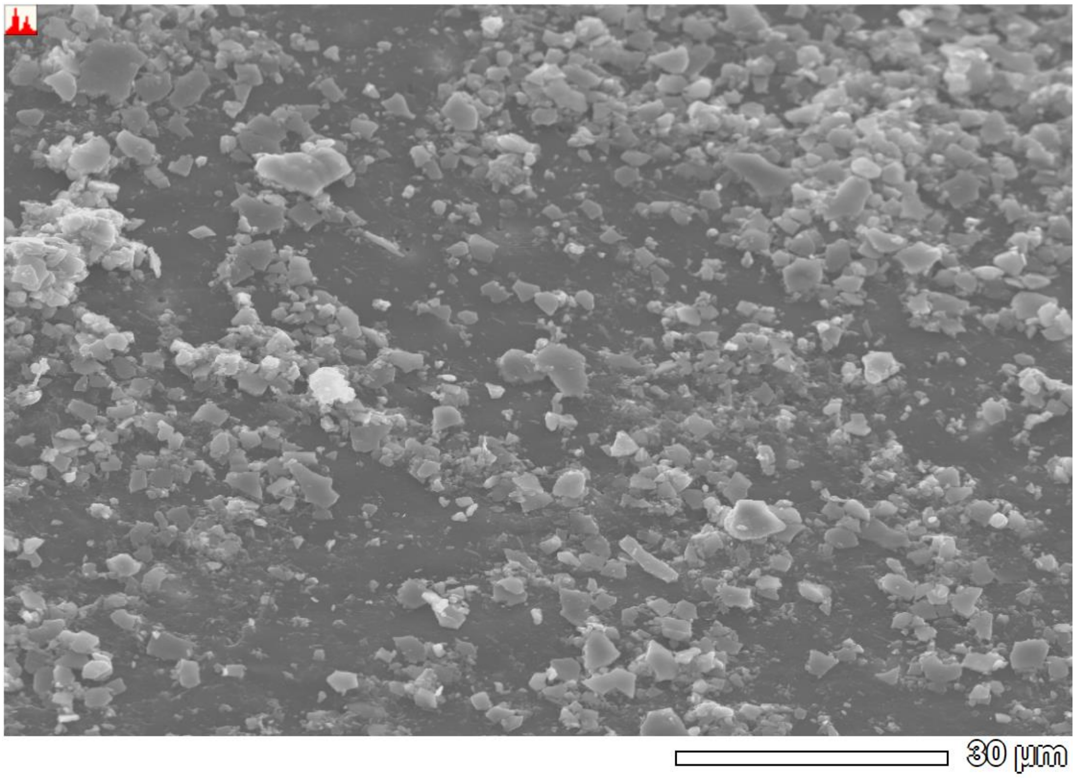
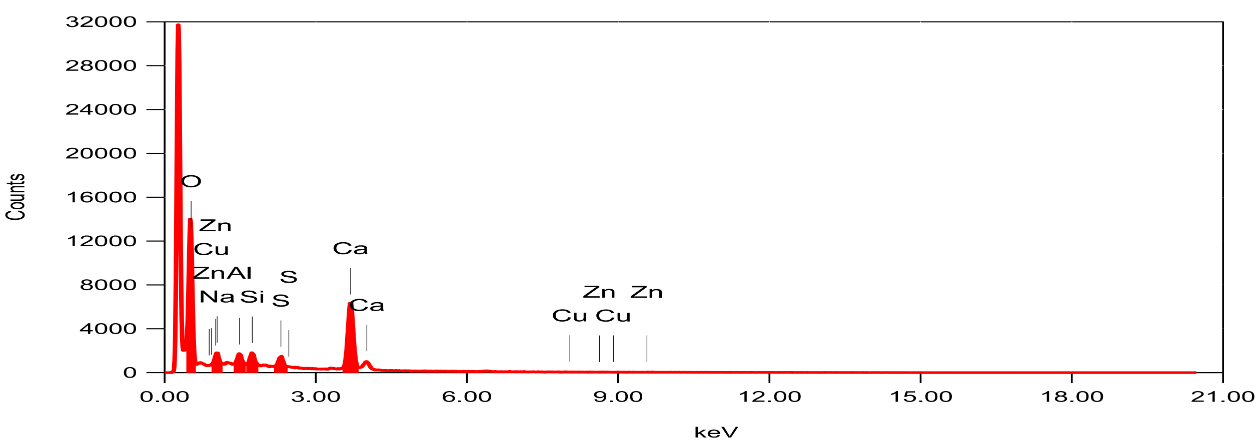
Appendix A : SEM – EDS Reports

A) TMS - Outside Layer

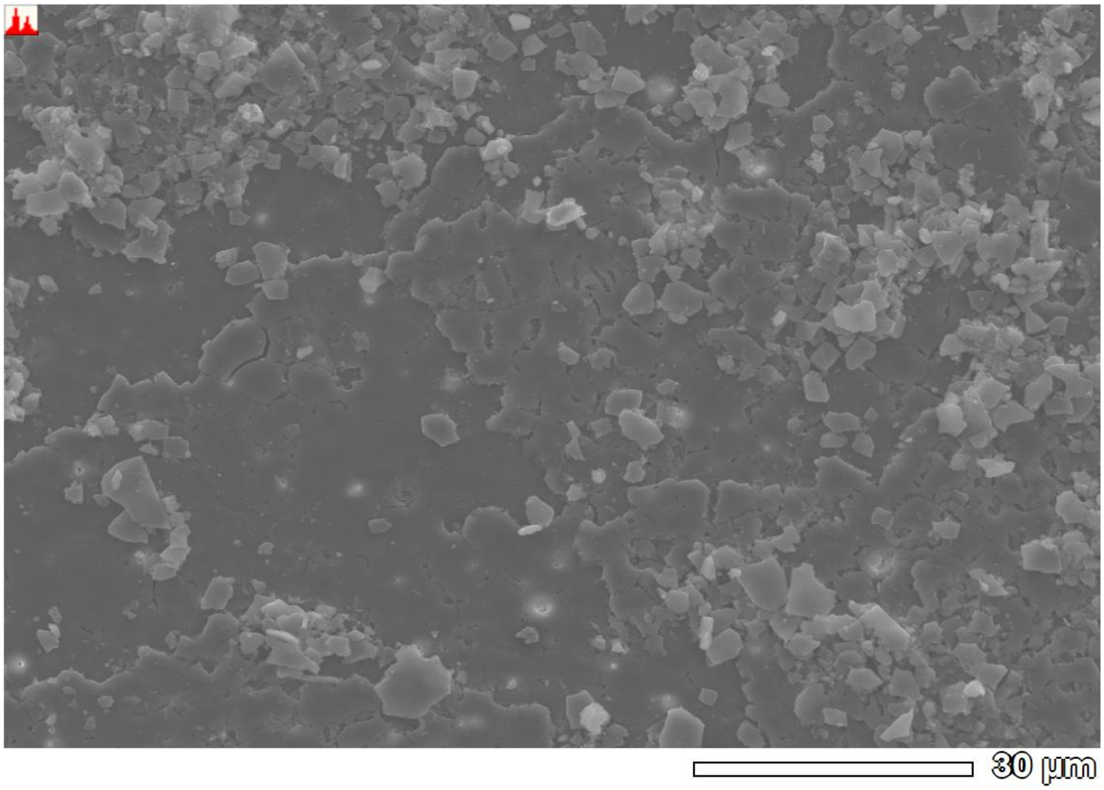


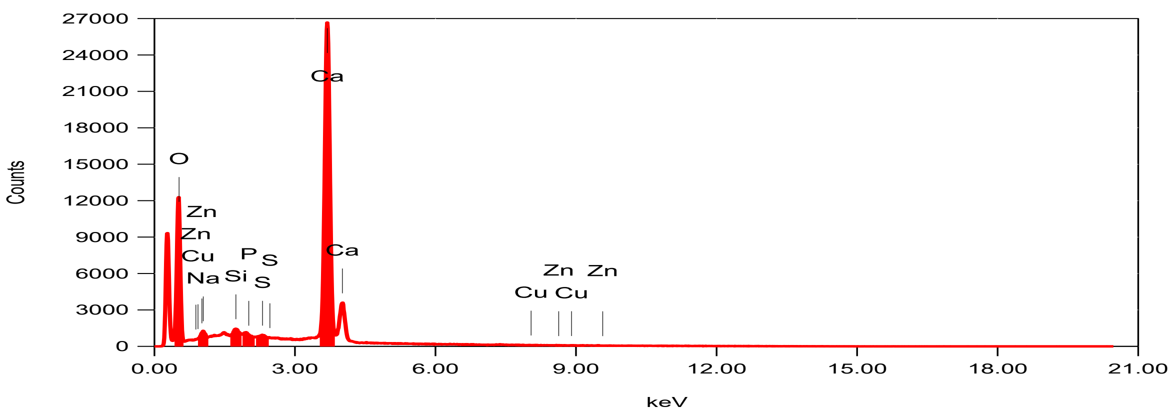


ZAF Method Standardless Quantitative Analysis
Fitting Coefficient : 0.6249

Element	(keV)	mass%	Error%	At%	Compound	mass%	Cation	K
O K	0.525	65.58	2.11	80.94				56.2543
Na K	1.041	2.72	2.13	2.34				2.1076
Al K	1.486	1.77	1.42	1.29				1.5943
Si K	1.739	1.97	1.47	1.38				2.0712
S K	2.307	2.20	1.32	1.35				2.7955
Ca K	3.690	25.77	2.87	12.70				35.1771
Total		100.00		100.00				

B) TMS - Inside Layer

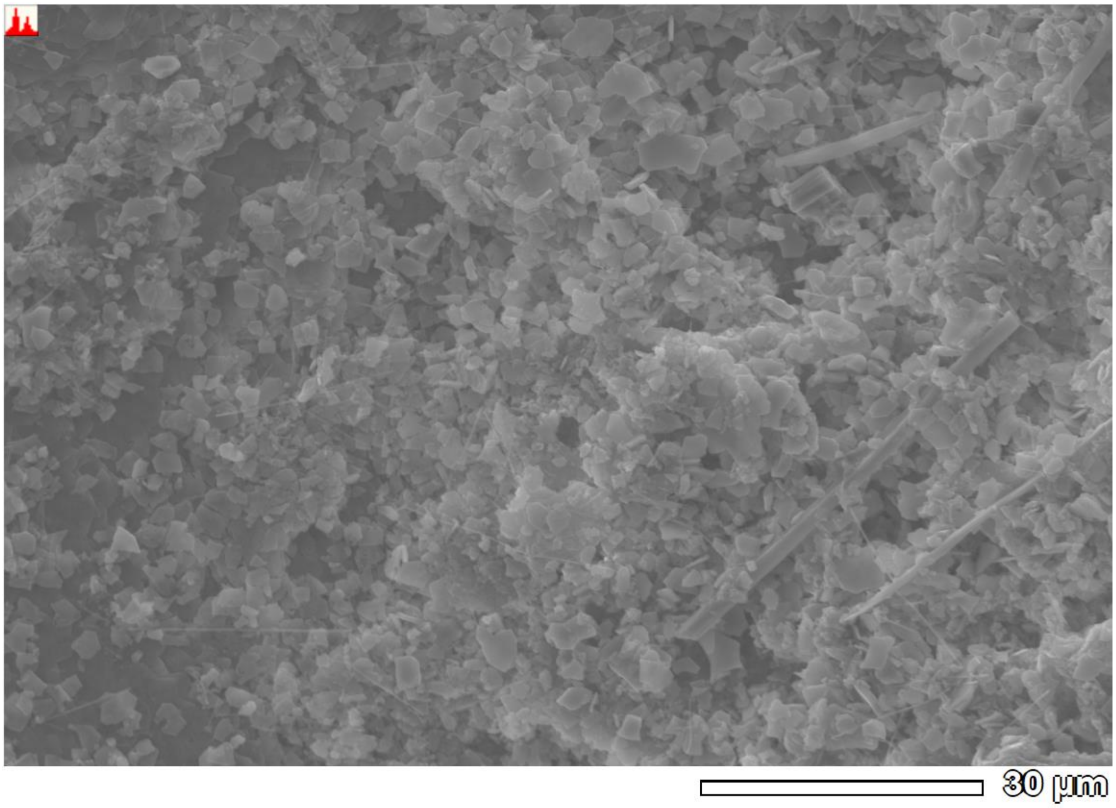


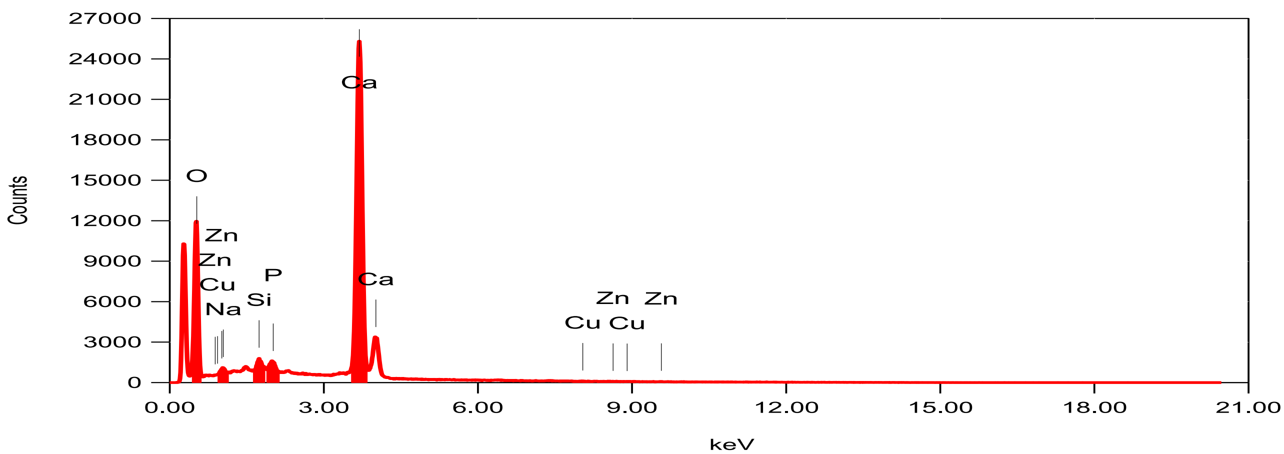


ZAF Method Standardless Quantitative Analysis
Fitting Coefficient : 0.2600

Element	(keV)	mass%	Error%	At%	Compound	mass%	Cation	K
O K	0.525	47.49	0.52	69.11				24.8450
Na K	1.041	0.66	0.31	0.67				0.5181
Si K*	1.739	0.36	0.21	0.29				0.3960
P K*	2.013	0.05	0.21	0.04				0.0589
S K*	2.307	0.18	0.18	0.13				0.2507
Ca K	3.690	51.18	0.40	29.73				73.8434
Total		100.00		100.00				

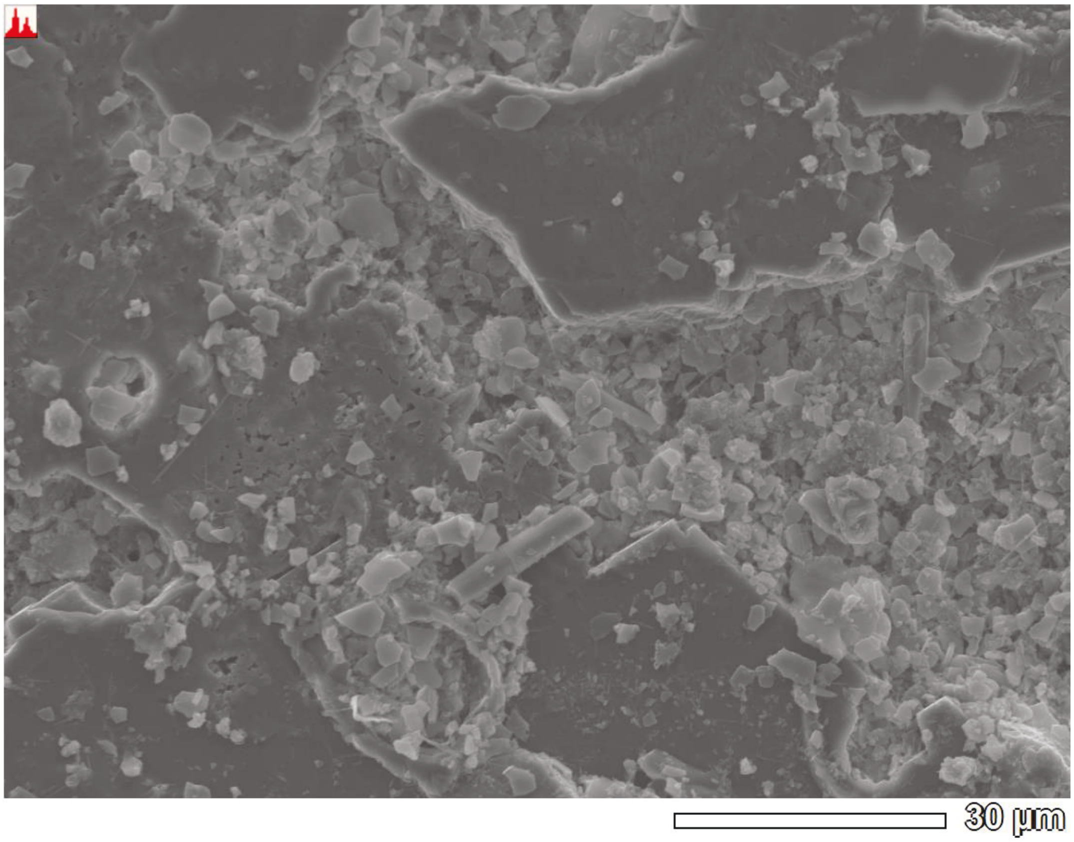
C) UTMS - Outside Layer

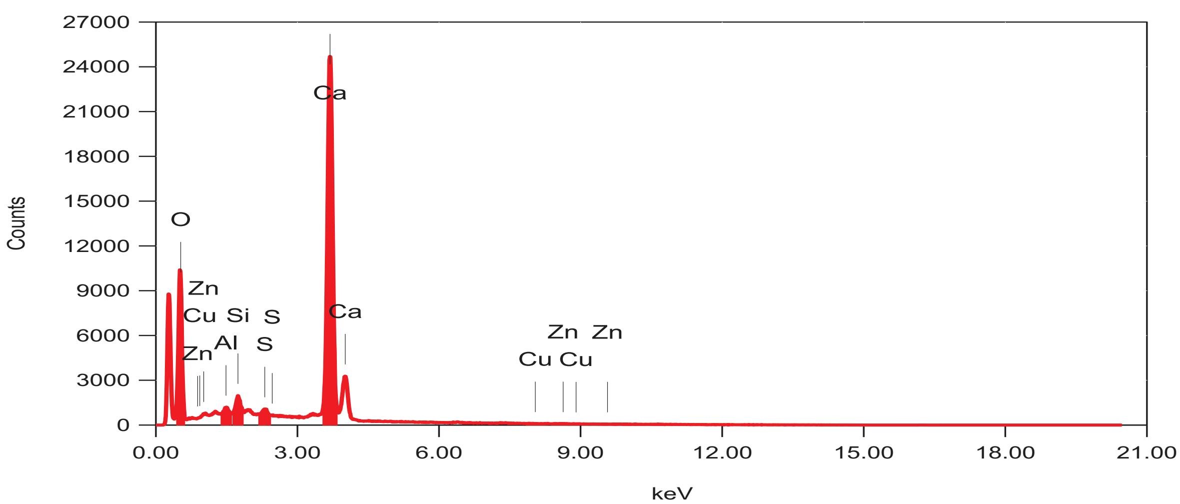





```
ZAF Method Standardless Quantitative Analysis
Fitting Coefficient : 0.2741
Element        (keV)   mass%   Error%     At%   Compound    mass%   Cation         K
 O K           0.525   47.31     0.58   68.95                                25.0084
Na K           1.041    0.47     0.35    0.48                                 0.3723
Si K           1.739    0.65     0.23    0.54                                 0.7269
 P K           2.013    0.76     0.24    0.57                                 0.9344
Ca K           3.690   50.33     0.46   29.28                                72.4402
Total                 100.00           100.00
```

D) UTMS - Inside Layer






```
ZAF Method Standardless Quantitative Analysis
Fitting Coefficient : 0.2622
Element        (keV)   mass%  Error%     At%  Compound    mass%  Cation        K
 O K           0.525   45.27    0.55   67.22                             22.7221
Al K           1.486    0.42    0.21    0.37                              0.3963
Si K           1.739    0.91    0.21    0.77                              1.0022
 S K           2.307    0.46    0.19    0.34                              0.6106
Ca K           3.690   52.55    0.42   31.15                             74.8553
Total                 100.00          100.00
```

Appendix B: Hydraulic Conductivity of The TMS and UTMS

Table C-2. The hydraulic conductivity tests for the selected particle sizes of the the TMS and UTMS (>1.0 mm and <12.7mm)

	Q	Time	Δh	Constant Head	L	A	Hydraulic conductivity		
Unit	L	SEC	CM	CM	CM	M^2	m/sec	m/hr	m/day
K1	0.11	15	4.5	82	30	0.00152	0.0322	115.8	2779.4
K2	0.41	60	6	82	30	0.00152	0.0225	80.9	1942.4
K3	0.14	30	3	82	30	0.00152	0.0307	110.5	2653.1
K4	0.1	60	1	82	30	0.00152	0.0329	118.4	2842.6
						Average	0.0296	106.4	2554.4

Appendix C : Average Daily Evaporation Data (2009-2011):

Table D-1 Average Daily Evaporation Per Months in Christchurch (NIWA, 2011):

Average Daily Evaporation (mm)							
Year					Year		
Month	2011	2010	2009	**Month**	2011	2010	2009
January	4.83 mm	4.27 mm	5.25 mm	**July**	0.61 mm	0.41 mm	0.46 mm
February	4.32 mm	4.11 mm	3.27 mm	**August**	0.82 mm	1.26 mm	0.95 mm
March	3.02 mm	3.09 mm	2.7 mm	**September**	1.61 mm	2.32 mm	2.13 mm
April	1.61 mm	1.65 mm	1.86 mm	**October**	2.19 mm	3.29 mm	2.88 mm
May	0.86 mm	0.83 mm	0.76 mm	**November**	3.46 mm	4.61 mm	4.29 mm
June	0.43 mm	0.35 mm	0.49 mm	**December**	3.48 mm	5.13 mm	4.65 mm