Using the Hydro-Brake® to Control and Attenuate Urban Stormwater Runoff, Improve WSUD Treatment Processes and Reduce Downstream Flooding Risk

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A joint project by the University of the Sunshine Coast, XP Solutions and Rocla Pipelines
Abstract

Water-sensitive urban design (WSUD) is a land planning and engineering design approach which integrates the urban water cycle, including stormwater, groundwater and wastewater management and water supply, into urban design to minimise the environmental degradation and improve aesthetic and recreational appeal (Kuhn, 2010).

Stormwater quality improvement devices (SQIDs) usually rely on sufficient residence times and low flow rates in order to filter efficiently. The increased urbanisation of Australia has added vast areas of impervious terrain across all built-up areas. This along with typical Australian high intensity, short duration storms is resulting in large amounts of runoff over a short period. These large rainfall events are causing inefficiencies within SQIDs, with pollutants gaining access to stream channels and causing problems with environmental flows. Important environmental characteristics can be maintained by constraining the volume, rate and timing of the filtration points (Hamsteed, 2007).

The Hydro-Brake® is a self-activating vortex flow control device that provides superior hydraulic performance over conventional flow regulators. The Hydro-Brake® can reduce on-site storage volume requirements by up to 30% (Rocla, 2015). The Hydro-Brake is a self-activating device that uses vortex principles to control and attenuate stormwater flow without the need for moving parts or external power requirements.

This study investigated the feasibility of the Hydro-Brake® attenuation device to control catchment runoff and reduce flow rates into SQID's to improve their pollution removal performance and reduce the risk of flooding. This study included both physical testing of Hydro-Brake® system in the laboratory to verify the hydraulic performance and computer modelling to assess the performance of these systems under real catchment runoff scenarios using typical rainfall data.

A major component of this research study was the use of XPSWMM modelling software in order to undertake a full hydrological and hydraulic analysis of the Hydro-Brake as a means of onsite detention and runoff attenuation. XPSWMM was used as the basis for catchment runoff calculations, peak flows, pipe geometry, and hydraulic performance of both the Hydro-Brake® and the pipeline throughout the system in order to determine flood risks and the overall successfulness of the design perimeters.

This study found that the Hydro-Brake could be successfully used to control stormwater runoff volumes by utilizing the storage volumes in existing drainage networks to act as a type of on-site detention (OSD) whilst reducing downstream outflows by up to 15.6%. The results of this research could potentially enable a reduction in the size of traditional downstream OSD systems resulting in significant economic benefits to developers and improved WSUD performance.
Acknowledgements

These acknowledgements are made to give due credit to the main partners of this investigation that have directly corresponded to the success of this project.

**Rocla Pipelines** - Rocla Pipelines is the leading Australian supplier of concrete solutions to the building and construction industry. Rocla provides a comprehensive and unique range of innovative engineered solutions for stormwater piping, pits, headwalls and box culverts, sewerage piping and access systems, irrigation, stormwater detention and treatment, rainwater harvesting, water storage, bridging and earth retention along with concrete poles, building columns, boardwalks and railway sleepers.

Rocla utilises many advanced engineering capabilities with a commitment to innovation, and product improvement. This along with a rigid quality assurance program ensures durable, high quality solutions.

Rocla is the supplier of the Hydro-Brake®, and the testing tanks used for this investigation. All necessary information regarding testing analysis has also been made available readily by request.

**Xp-Solutions** - Xp-Solutions is a world leading provider of industry standard sustainable drainage and flood hazard software for the civil engineering and environmental sectors. Their solutions are used every day around the world by government agencies, engineering companies and environmental management organisations to plan, design, simulate and manage the impact of human interaction with the natural world.

Xp Solutions has supplied the full use of the program XPSWMM for the purpose of this investigation. This program is a fully dynamic hydraulic and hydrologic modelling software that combines 1D calculations for upstream to downstream flow, with 2D overland flow calculations so that a true insight into what happens to stormwater systems. Its use over the last 25 years has made it one of the most stable and well used simulation software programs in the world. This program has enabled the comprehensive hydraulic analysis of the Hydro-Brake® in the interest of further developing WSUD.

**Project Supervisors** - Due thanks to both Dr. Terry Lucke and Dr. Peter Nichols for guidance, feedback and support throughout the project lifecycle.

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Chapter 1: Introduction

1.1 Outline of Investigation
This was a joint research undertaken by the University of the Sunshine Coast (USC), Rocla and XP Solutions as a means of using the Hydro-Brake® to control catchment runoff and reduce flow rates into stormwater quality improvement devices (SQID’s). This research also addressed the use of the Hydro-Brake® as a means of reducing downstream urban OSD systems and the chance of flooding resulting from Australian temporal patterns.

1.2 Background of the Study (Water Sensitive Urban Design)
Water-sensitive urban design (WSUD) is a land planning and engineering design approach which integrates the urban water cycle, including stormwater, groundwater and wastewater management and water supply, into urban design to minimise the environmental degradation and improve aesthetic and recreational appeal (Kuhn, 2010). Many organisations in Australia are undertaking projects to provide WSUD principles to support its adoption. Leading to WSUD becoming a key component in sustainable urban development as well as aiding flood prevention and water quality assurance and protection.

WSUD is dependent on the stormwater quality improvement devices (SQIDs) that are installed within the catchment due to the need to remove significant quantities of pollutants from entering the stormwater drainage system (BCC, 2008). Common SQIDs typically used include swales, bio-strips, wetlands, ponds, permeable pavements and bio-retention basins (Kuhn, 2010).

1.3 Project Objectives
This project was undertaken using both computer modelling (XP SWMM) and laboratory-based physical modelling components. The objectives of this research study included;

- Investigate the hydraulic performance of the Hydro-Brake® as a primary method of catchment runoff attenuation and control;
- Investigate the hydraulic performance of a series of flow control and attenuation devices operating under different flow conditions to understand their simultaneous operation and interactions;
- Investigate the flow reduction and flood attenuation potential of a series of devices operating simultaneously;
- Investigate the use of existing pipe storage as a feasible way to aid sufficient residence time in SQIDs:
• Model the hydraulic performance of a series of Hydro-Brake® devices using the computer modelling system XPSWMM:
• Verify the computer modelling results with laboratory based modelling and:
• Develop a set of simultaneously working control devices guidelines for use in development in applications and flood modelling studies in the subject area of Buderim to be future applied to works around Australia

1.4 Investigation Methods
This thesis presents the results of a feasibility study on using the Hydro-Brake® in WSUD as a method of flow control and runoff attenuation to successfully utilize the potential storage in existing underground stormwater drainage system pipes. This study gives and discusses the findings from varying permutations of catchments, pipe sizes and Australian Rainfall events as specified by the Bureau of Meteorology (BOM), through the use of practical testing and computer based modelling.

This investigation followed a standard process of investigation, design and test under differing testing rig configurations and levels of runoff attenuation and flow rate control.

This was undertaken using the following steps:

1. A literature review was undertaken to determine the scope of the study by identifying and examining previous research studies relevant to the current study. An appropriate range of hydraulic conditions that needed to be modelled were identified including common Australian drainage pipe sizes, rainfall events and catchment types to produce the desired study outcomes.
2. The findings from the Literature review were then used in conjunction with the computer modelling program XPSWMM to undertake a full hydrological and hydraulic analysis of the testing configurations.
3. The results from the computer based modelling were testing in a hydraulic laboratory setting to verify the results.

1.5 Project Outcomes
There is significant evidence shown from this thesis that there is potential for the advancement into the use of existing underground drainage pipes as a method of OSD, whilst improving the efficiency of SQID's.

The findings will enable the use of multiple Hydro-Brake®s working simultaneously to achieve a greater percentage of successful runoff filtration, economical and safer WSUD design and increased benefits to an increasing demand on land area.
1.6 Thesis Structure

1.6.1 Chapter 1: Introduction
This chapter introduces WSUD, with an underlying issue of inefficient residence times in SQUID's. The project objectives are outlined, with a description of the Hydro-Brake® and their future potential.

1.6.2 Chapter 2: Literature Review
In order to identify research and information relevant to this study, this chapter covers WSUD, stormwater treatment methods, Australian Rainfall and Runoff and average recurrence interval storms and typical underground stormwater drainage components. This chapter also reviews the throttle hose and vortex valve flow control devices and OSD.

1.6.3 Chapter 3: Methodology
This chapter covers all aspects of the testing apparatus. Both computer modelling and hydraulic laboratory investigations are discussed. This chapter covers hydraulic capacity testing, computer modelling for both single and multiple catchment peak inflows and the laboratory verification tests.

1.6.4 Chapter 4: Results
This chapter gives all the results for both the computer modelling and hydraulic laboratory investigation.

1.6.5 Chapter 5: Discussion
This chapter discusses all the findings from the results. The significance of these findings and how they relate to one another are investigated, with future potentials outlined.

1.6.6 Chapter 6: Conclusion and Recommendations
This chapter concludes on the points raised in the discussion, with emphasis on the trends in the data, the reduction in OSD and how the Hydro-Brake® could relate to future WSUD principles. Recommendations are given for the future use of the Hydro-Brake® as well as how future study could be undertaken to further investigate the significance of the Hydro-Brake® as an integral part of all WSUD design.

1.6.7 Chapter 7: References
This chapter gives due credit to all that have aided the production of this thesis.

1.6.8: Chapter 8: Appendix
This section gives all the raw results, for all tests. All areas that did not directly improve the discussion have been added to the appendix.
Chapter 2: Literature Review

In order to identify the scope of this study, a comprehensive literature review was undertaken. The literature review focused on the major principles of WSUD, stormwater treatment, Australian Rainfall and Runoff, ARI storms, flow control valves and typical underground stormwater drainage components. On site detention has also been briefly discussed.

In order to gain a full understanding of the research, methods of flow control, underground stormwater drainage systems including OSD have been reviewed. To give a scope of the project WSUD and stormwater treatment methods have also been studied.

2.1 Water Sensitive Urban Design

WSUD is about managing all water on an urban development in an intimately interconnected manner. It seeks to integrate management into the three strains of water (groundwater, wastewater and stormwater). Some key principles of WSUD are as follows (WBD, 2004);

- Protect natural systems, etc. downstream waterways ponds/lakes and wetlands.
- Protect and increase the quality of groundwater by treating stormwater and wastewater.
- Reduce runoff and peak flows and therefore provide opportunities to OSD and the reuse of stormwater in surrounding areas.
- Add social and ecological benefits to development whilst minimising construction costs and ecological degradation.

Many organisations in Australia are undertaking projects to provide WSUD principles to support its adoption (Kuhn, 2010). Leading to WSUD becoming a key component in sustainable urban development as well as aiding flood prevention and water quality assurance and protection. Increased urbanisation is resulting in a vast number of detrimental changes to stream channels such as (Wong, 2002);

- Increase in the rate of high water velocities
- Increased sediment supply and transport rates
- Increased removal of the more easily eroded materials.
- Increased frequency of bed erosion and bed erosion head retreat.

To best combat these issues WSUD adopts the following key objectives in order to minimise the effect urban construction has on both environmental quality and water management. (Healthy Waterways, 2014)

- Minimise impacts on existing natural features.
- Minimise impact on natural catchments (catchments before urbanisation)
- Minimise demand on reticulated water
- Add value whilst minimising costs, this can be done by mitigating the need for drainage systems by added infiltration.
- The reuse of ground and stormwater from catchment runoff.
- Improve quality and reduce pollutants in catchment runoff.

A key part for the need of WSUD design techniques is the urbanisation of catchments. Under natural (not constructed or urbanised) conditions, large amounts of rainfall soaks into the ground - this is known as infiltration. This replenishes groundwater and thus keeps the eco-system alive. Any rainfall which has not been used in the infiltration process drains into local streams and creeks before travelling through to the ocean (WBD, 2004).

In urban areas, what is known as an impervious area is introduced. Impervious areas result in no infiltration with the only losses occurring due to evaporation, resulting in much larger amounts of catchment runoff. This process can be seen in figure 1 below.

![Figure 1 - Comparison of Pre and Post Urbanisation Catchments (WBD, 2004)](image)

Forms of impervious areas are constructions such as roads, concrete and roofs. Instead of the infiltration process the runoff must travel through drains and underground pipes into creeks and rivers which eventually to the ocean. A major consideration of this is the amount of pollutants and sediment that is added to the waterway due to this process, with erosion issues also occurring. As the urbanisation process is being undertaken, loose soil and debris can be washed into the waterways. (BCC, 2008)
It is this issue that leads to WSUD, which regards urban stormwater runoff as a recourse rather than a design liability and hindrance. It is this key difference to the stormwater design that encourages economic benefit.

Some of the common barriers to the adoption of WSUD are as follows (WBD, 2004);

- Not broad enough policy, with inconsistencies within council guidelines.
- Finance and Cost
- Risk Management and Assessment
- Lack of incentives

Whilst there are incentives in place in order to combat this such as council rebates, a study of south east Queensland revealed these as the major factors against WSUD.

A good example of WSUD was the Sydney 2000 Olympic park construction site at Brownfield, Homebush. This site originally consisted of a landfill area, abattoirs and a navy depot. This was constructed into a multiuse Olympic site. A large scale scheme was set up in order to recycle non-drinkable water which included a large number of WSUD technologies (DEC, 2006). These WSUD principles were implemented in order to protect receiving waterways from stormwater and wastewater discharge.

These WSUD attributes were put into use to ensure onsite treatment, storage and recycling of stormwater and wastewater. The following stormwater quality improvement devices were used in this project;

- Pollutant traps
- Swales
- Wetlands

By implementing this system it was found that there was a 90% reduction in pollutants to the nearby Haslam Creek wetland. The WSUD principles adopted resulted in 850 million litres of water being conserved annually rather than being lost as runoff leading to a reduction of up to 50% reduced use of potable water (DEC, 2006).

This is one example of how the proper implementation of WSUD can lead to reductions in water demand and preventing environmental degradation.
2.2 Stormwater Quality Improvement.
Under any WSUD design scheme the aim is to achieve deliverable environmental flows. An environmental flow defines the quantity, timing and quality of water flows required to sustain ecosystems. Flow rates into rivers are being modified with increasing frequency when water is taken for urban use, such as development resulting in catchment runoff not returning to the rivers as groundwater (World Bank, 2015).

In order to achieve the deliverable environmental flows for a catchment area removing pollutants is key. The main stormwater pollution constitutes include sediment, trash, debris, nutrients and metals (Hamsteed, 2007). To combat this, depending on the sites particular needs (target pollutant, local requirements and site characteristics), filtration devices are used stormwater quality improvement devices (SQIDs).

Two main characteristics that define a stormwater quality device is that of peak flow and treatment flow. Treatment flows are the flow in which a SQIDs can successfully treat flow through a drainage system. These are based on the site area and are usually calculated from a 3 month ARI peak flow (Rocla, 2014). A common issue for SQIDs is there allowable flow through the system without being overrun and causing damage to the system and the surrounding eco-system. These flows must be designed to in order to allow successful filtration and achieve environmental flow design parameters.

Some common SQIDs include;

**Bio-retention Systems** - This system uses vegetation as a filtration and treatment device. The vegetation provides uptake of contaminants such as nitrogen and phosphorus. These systems are commonly used to treat stormwater runoff before entering street drains (CSRIO, 2008).

**Bio-retention Swales** - This system is usually installed into the median strip of divided roads or at the sides of residential roads. These provide stormwater treatment and conveyance functions. The stormwater runoff usually passes through a filter and proceeds through a perforated pipe to downstream waterways or storage sections (CSRIO, 2008).

**Bio-retention Basins** - These basins generate similar water quality and flow control to that of the bio-retention swales. These basins work in the same way as the bio-retention systems, however they also provide extended detention of stormwater to increase runoff treatment in small and medium environmental flows. These basins are often used along streets at regular intervals to treat runoff prior to entry into drainage. However larger systems have been used to treat drainage outfall (CSRIO, 2008).
Infiltration trenches and systems - These are shallow trenches/excavated structures filled with permeable materials such as gravel. The excavated are results in essentially an underground reservoir. These hold stormwater runoff within the trench and gradually releasing it into the surrounding soil (BMT WBM, 2009). Peak discharge flows from impervious areas are reduced by capturing and infiltrating flows. These trenches are designed to be located as the last stage of a WSUD system as a method of flow distribution to aid the mitigation of downstream flooding. (CSRI, 2008)

Sand Filters - Sand filters are a mix between the infiltration trenches and the bio-retention systems. Stormwater passes through the sand filter before being discharged downstream. Sand filters are generally used in high built up urban areas with high impervious area percentages (BMT WBM, 2009). This filter consists of a sedimentation chamber to remove contaminants such as litter and debris, a weir and a sand layer that filters sediments and dissolved pollutants.

Wetlands - These are also known as constructed wetlands and are designed to remove stormwater pollutants associated with dissolved contaminants and fine particles. These wetlands comprise of three zones; an inlet zone, a vegetation zone and a high flow bypass area. These usually require large amounts of space along with high costs. Mosquitoes and the infestation of litter are also common problems of this WSUD principle (Melbourne Water, 2010).

Ponds/Lakes - These are artificial bodies of water through the construction of a dam or weir outlet structure. Ponds and lakes are similar to wetlands and can be used to treat runoff by providing extended detention to allow the process of sedimentation to happen (MCC, 2008). Another common use of the pond and lake system is to decrease the risk of downstream flooding by allowing higher retention time up stream.

These methods are considered secondary stormwater treatment devices and are primarily used for removing fine sedimentation such as bacteria and fine debris. In some designs this is not enough due to the runoff also resulting in course sedimentation. To counter these the following primary methods of stormwater treatment have been adapted (EPA, 1997);

- **Litter Basket** - These area a basket installed within an inlet pit to collect rubbish directly entering the stormwater system from road surfaces.
- **Litter Control Pits** - These are a basket located in a stormwater pit. These collect litter and debris from upstream piped drainage system.
- **Litter Trash Rack** - This is a vertical rack installed across a stormwater channel/ downstream of a sediment trap.
- **Gross Pollutant Trap** - These are one of the most common form of primary stormwater treatment. These are a sediment trap consisting of a litter rack, located downstream of the trap.

- **Litter Boom** - These are a floating device installed in waterways to collect floating litter, debris and chemical pollutants such as oil.

Whilst there are other primary stormwater treatment methods, these are the most commonly used in Australia.

SQIDs can be incorporated into both new development and existing redevelopment and infill projects with its implementation founded on site design, pollutant source control and runoff treatment control (Hamsteed, 2007). The land development industry in Australia has generally incorporated ponds and wetlands into urban design under a natural environment theme in their development plans (Wong, 2002). SQIDs usually rely on sufficient length residence times and low flow rates in order to reach environmental flow targets based on the treatment flow throughout Australia. However, as urbanisation throughout Australia adds to the decrease of natural catchments more catchment runoff occurs. Compounding this issue is Australia’s common short duration, high impact storms. These rainfall events are causing SQIDs to be insufficient with OSD systems needed to be made larger and pollutants still entering the waterways (Wong, 2002). By maintain a constraining volume through these systems importance environmental characteristics can be held (Hamsteed, 2007).

It is these principles that lead to the need of an external attenuation device that can either take the place or be used in conjunction with the WSUD devices outlined above to increase the success rate of a WSUD treatment train. This is where the use of the Hydro-Brake® - a device that allows for desired flows through certain storage points will come into effect. It is hypothesised that the performance of SQIDs could be improved significantly with the adoption of upstream on site detention techniques.

## 2.3 Australian Rainfall and Runoff and Average Recurrence Interval Storms

Under any drainage and flood plain modelling, hydrologic and hydraulic design requirements are evident. Hydraulic analysis is dependent on the hydrological outcomes from a catchment and therefore its correct calculation is of vast importance.

A primary part of a hydrological analysis is the calculation on peak flow rates for the design of drainage systems and downstream OSD principles based on the rainfall intensity of an area (QUDM, 2013). Hundreds of millions of dollars are spent annually on hydraulic structures - such as underground stormwater drainage throughout Australia. The rainfall intensity, frequency and duration (IFD) define the design of these structures (BOM, 2015). To assist with the calculations of these structures the
bureau of meteorology provide design rainfall data based on these IFD’s collected from real storm events. These IFD’s have been calibrated into average recurrence interval (ARI) storms which measures the rarity of a storm event occurring. An ARI storm burst is defined as the average or expected interval between exceedance of a given rainfall total over a given duration (BOM, 2015). These durations are given as part of the ARI (e.g. 10 year 15 minute ARI). It is this definition that also leads to the fact that an ARI storm is considered a storm burst and not a full storms, as rainfall volumes often change within a storm event.

Whilst a major aspect of the hydrological analysis is the rainfall events and temporal patterns of a storm, other parameters such as rainfall losses and catchment peak inflow calculations are also required.

Rainfall losses occur due to infiltration into pervious areas of a catchment - such as soil, tree and canopy cover, with losses also occurring due to evaporation. These losses are accounted for in the Australian Rainfall and Runoff guidelines through a lump loss method. This method specifies that calculation of infiltration losses are broken into two phases - initial losses and continuing losses. The time required for runoff to enter a pipe system is classified as the initial period and is assumed to be a 7 minute period for residential areas and 5 minute period for side entry pits (Pilgrim, 1987). A key aspect of these factors is that ARI storm burst are calculated as part of a storm - not an entire storm and therefore the assumption of pre-occurring rain is taken.

Peak catchment runoff can be calculated by a number of ways with the most common method being the rational method. The rational method is generally used to calculate design peak inflow from a catchment providing the area is less than 400 hectares. The following formula depicts the rational method. (Melbourne Water, 2010)

\[ Q = C_y I_y A / 360 \, \text{m}^3/\text{s} \]

Where:-

\[ Q = \text{Peak inflow resulting from an ARI storm} \]

\[ C_y = \text{Runoff coefficient for design event having a certain ARI} \]

\[ A = \text{Area of catchment in hectares} \]

\[ I_y = \text{Rainfall intensity (mm/hr) corresponding to a particular ARI storm event.} \]

Whilst the rational method is commonly used, peak inflows calculated from this formula are very broad calculations based on the number of variances between catchments. Other methods such as
Laurenson’s can be used by stormwater modelling software which takes into account surface roughness (XP Solutions, 2015).

### 2.4 Typical Underground Stormwater Drainage Components

Stormwater drainage is a system that channels rainwater as it comes off impervious areas in the form of catchment runoff. In order to undertake this research an analysis of a typical underground stormwater drainage system was needed.

A typical stormwater drainage system comprises of the following three functions;

- **An inlet** - There are two main types of stormwater inlets - a side inlet and a grated inlet. Side inlets are commonly found at the side of residential roads. Grated inlets are the most common form of stormwater drainage inlet due to their ability to prevent large pollutants and debris from entering the storm drain. These inlets must be strong enough to support the weight of all traffic and pedestrian activity in the area.

- **Pipe work** - Drainage pipes consist of many different design techniques including shape, joints, diameter and length. These pipes can also be made from many different materials such as reinforced concrete, PVC and brick.

- **An Outlet** - The final part of a stormwater drainage system is the outlet. Stormwater outlet points usually release in rivers and lakes eventually entering the ocean. It is common for the use of SQIDS between the outlet point and eventual ocean entry in differing methods as can be shown in figure 2 below:

![Possible Outlets into SQIDs](EPA_1997)

**Figure 2 - Possible Outlets into SQIDs (EPA, 1997)**

#### 2.4.1 Pipe Size and types

Under Queensland regulation, pipe sizes must be designed for the minimum capacity in accordance with minor drainage requirements as specified by table 1 and 2 below. Under all circumstances
underground drainage piping made from reinforced concrete must be at a minimum of 375mm diameter. For all low density residential areas pipe classes must be in line with standard AS 1254 - pipes and fittings for stormwater and surface applications. For all other types of areas (commercial, medium and high density residential etc..) pipe classes must be in line with standard AS 1260 - Pipes and fittings for drain, waste and vent application (BCC, 2008)(QUDM, 2013).

Table 1 - Design Drainage Standards Part A (BCC, 2008)

<table>
<thead>
<tr>
<th>Development Category</th>
<th>Design Parameter</th>
<th>Drainage Design Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Major Roads (District, arterial, suburban route)</td>
<td>Minor Drainage System</td>
<td>Culvert System flow 2% AEP where crossing overland flow flooding</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>1% AEP where crossing creek/waterway</td>
</tr>
<tr>
<td>Minor Roads (Local neighborhood)</td>
<td>Minor Drainage System</td>
<td>2% AEP where crossing overland flow flooding</td>
</tr>
<tr>
<td></td>
<td></td>
<td>1% AEP where crossing creek/waterway</td>
</tr>
</tbody>
</table>

Table 2: Design Drainage Standards Part B (BCC, 2008)

<table>
<thead>
<tr>
<th>Development Category</th>
<th>Design Parameter</th>
<th>Design Standard</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rural Areas (2-5 dwellings per hectare)</td>
<td>Minor Drainage System</td>
<td>Minimum 50% AEP</td>
</tr>
<tr>
<td></td>
<td>Major Drainage System</td>
<td>Minimum 2% AEP</td>
</tr>
<tr>
<td>Residential Developments (Low Density)</td>
<td>Minor Drainage System</td>
<td>Minimum 50% AEP</td>
</tr>
<tr>
<td></td>
<td>Major Drainage System</td>
<td>Minimum 2% AEP</td>
</tr>
<tr>
<td></td>
<td>Roof Water Drainage</td>
<td>Level 2 QUDM</td>
</tr>
<tr>
<td>Residential Developments (Medium-high density)</td>
<td>Minor Drainage System</td>
<td>Minimum 10% AEP</td>
</tr>
<tr>
<td></td>
<td>Major Drainage System</td>
<td>Minimum 2% AEP</td>
</tr>
<tr>
<td></td>
<td>Roof Water Drainage</td>
<td>Level 3 and 4 QUDM</td>
</tr>
<tr>
<td>Industrial Uses</td>
<td>Minor Drainage System</td>
<td>Minimum 50% AEP</td>
</tr>
<tr>
<td></td>
<td>Major Drainage System</td>
<td>Minimum 2% AEP</td>
</tr>
<tr>
<td></td>
<td>Roof water and Lot drainage</td>
<td>Level 4 QUDM</td>
</tr>
</tbody>
</table>

In accordance with this the following minimum pipe classes have been adopted for stormwater drainage in Queensland. (QUDM, 2013)

- Steel Reinforced Piping - Class 2
• Fibre Reinforced Piping - Class 1
• Flexible Piping - Class SN8

The use of flush joint pipes recommended due to the fact that infiltration is encouraged (Rocla, 2015).

All underground stormwater drainage must follow a minimum grade in accordance with QUDM section 7.12.

Storm drains cannot always handle and manage the quantity of rainfall and runoff into the system, often resulting in street flooding. Urban flooding is the primary cause of backups in sewers and affects thousands of homes each year. It is these principles that lead to the need of an external attenuation device that can be used in conjunction with the existing underground system to increase the success rate of drainage systems (CNT, 2014). As specified above, the minimum diameter for an underground stormwater pipe is 375mm based on design storm calculations. As these pipes seldom reach capacity, the potential for using the drainage line as OSD is evident.

2.5 Throttle Hose
The automatic throttle hose is a type of flow regulation device (Figure 3), which delivers an almost constant discharge independently of fluctuations and upstream water levels (Volkart, 1985). This device was used in conjunction with the early stages of WSUD.

![Figure 3 - Automatic Throttle Hose Plan View (Volkart, 1985)](image)
The Automatic Throttle Hose has also been used for regulating flow rates in irrigation schemes and outflows from storm water retention basins. In every field of application evaluated, a nearly constant outflow discharge was obtained despite upstream water pressure or head variations (F.De. Vries 1984). When the surface level of a reservoir or retention area is higher than that of the outlet pipe as water flows through the pipe, an excess pressure acts on the outer surface of the piping part within the reservoir. By making this section of the pipe deformable a buckling effect is utilized to narrow the effective pipe diameter due to the pressure differences. The more the effective pipe diameter decreased the more regulation is put onto the outflow. The throttle hose is used as a short part of an outlet pipe, where the use of a rubber hose restricts as a greater amount of pressure travels through the piping system (figure 4). The advantage to using this system is that at low levels of pressure the throttle hose acts in the same way as a standard outlet pipe but as the pressure reaches a critical head level the pipe deforms causing regulation. The throttle hose has also been installed in parallel to meet specific discharge limits and head levels. A major advantage to this system is the ability to act without an external power supply (Volkart, 1985).
The throttle hose is commonly used in small sized stormwater retention basins, however are generally not suitable for larger basins and most WSUD due to the flooding risk. The maximum head in a small water basin does not usually exceed 4 to 5 metres and therefore makes the throttle hose a good edition to flow regulation (F.De. Vries 1984). With larger heads however the throttle hose begins to be ineffective due to the large pressure differences between the outlet pipes can begin to cause elastic failure and pulsations causing unreliable flow rates (figure 5). The rubber membrane of the hose must also be changed periodically due to the change of the rubber characteristics and therefore the discharge characteristics caused by chemical and UV influences.

![Figure 5 - Flow Regulation with Differing Head Levels (Volkart, 1985)](image)

![Figure 6 - Failure Point of Throttle Hose (Volkart, 1985)](image)
2.6 Vortex Valve

Another form of flood prevention and runoff attenuation is the vortex valve. These valves were first used for stormwater attenuation and runoff control in the 1970's and have become more commonly used through the 1980's (Faram et al., 2010). The vortex valve uses the principle of pressure loss when flow is accelerated into a swirling motion. This principle results in an air core through the axis of the valve resulting in a spiral fan flow discharge (figure 7).

![Figure 7 - Fanning Discharge of a Vortex Valve](image)

The physical clearances through a vortex valve for any given head-flow can be up to five times larger than that of a standard orifice plate resulting in an 'S' shaped head-flow characteristic (figure 8). This aspect of the valve allows for less risk of pipe blockages caused by biological clogging of infiltration and filtration systems and cause significant problems in the case of stormwater systems (Kandra et al., 2015). The larger physical clearance also means that at low heads, vortex valves act in the same way as orifice plates and do not hold back flow unnecessarily allowing for the future more needed use of upstream storage. The 'fanning' type discharge also significantly reduces the effect of lateral erosion that can occur through a standard linear discharge devices like an orifice plate.

![Figure 8- Vortex Valve Head-Flow Characteristic (Faram et al., 2010)](image)
Tests to undertake a comparison of a Vortex Valve against a standard orifice plate show the following outcomes under the same flow rate conditions; under low flow rate conditions such as the beginning of a storm or light rainfall the head loss through the orifice plate is significantly larger than the vortex valve due to the orifice plate’s smaller clearances. This results in a higher head in a chamber with an orifice plate then one with a vortex valve.

![Figure 9 - Low Flow Rate Head-Flow Characteristics. (Faram et al., 2010)](image)

As the flow increased the orifice plate chamber continues to have higher head until the point at which the vortex valve us fully submerged. At this point the flow controls of the orifice and vortex chambers converged to a near same head-flow characteristic. However due to the fact that the vortex valve discharge more in the early stages the storage chamber has been used more economically.

![Figure 10 - Higher Flow Rate Head-Flow Characteristics (Faram et al., 2010)](image)

Vortex valves have been successfully applied around the world with design flow rates from a few litres per second to large fast flow rates to pipe lines as small as 75mm through to 2m diameter (Faram et al., 2010). In all cases these valves have yielded practical and economic benefits in aiding storage requirements and flood alleviation. The use of the vortex valve has been extended to aid filtration and the use of storage systems. Figure 11 depicts the use of a three stage system utilizing the vortex valve to great success (Deahl et al., 2002). These three stages are as follows;

- A stormwater treatment stage using the Downstream Defender®;
- A stormwater storage stage using the Storm cell® Storage System; and
- A flow control / attenuation stage using the Reg-U-Flo® Vortex Valve.

This system shown in figure 11 removes 95% of grit particles at a 300L/s flow rate, with the use of the storm cell blocks utilizing 95% voids rather than commonly used 40% aggregates. (Deahl et al., 2002)

2.7 Hydro-Brake®
The Hydro-Brake® is a self-activating vortex flow control device that provides specific hydraulic performance over conventional flow regulators. The brake is the newest innovation (Rocla, 2014) in vortex valves and this study will investigate the use of the Hydro-Brake® to improve on the benefits of these devices discussed above. The focus of the investigation will be to study the simultaneous effect being utilised to mitigate blocking through the valve (which is already 400-600% lower than orifice plates) (Rocla, 2014).

The Hydro-Brake® can reduce on-site storage volume requirements by up to 30%. The self-activating device uses vortex principles to control and attenuate stormwater flow without the need for parts or external power requirements. The openings on the Hydro-Brake® also have clearance up to 600% larger than a conventional orifice plate and therefore reduces the risk of blockages caused via sediment in the flow.

The brake used in this study was the RUF-002 large flow control device. This device could handle any flow up to 20L/s with the following head/flow curves out of the storage chambers.
These brakes had to be mounted using a small scale curved mount (RUF-004) which could only be mounted in a tank with a diameter of 1000mm-1200mm. These the RUF-002 Hydro-Brake® also allowed a maximum outlet pipe of a 300mm diameter.

The Hydro-Brake® acts in the following phase process.

1. **Pre-Initiation Phase:** Under low flow conditions the Hydro-Brake® flow control behaves like a traditional orifice with an aperture cross sectional area similar to that of the Hydro-Brake® flow control outlet aperture. The flow is gentle, with minimal turbulence inside the volute of the Hydro-Brake® flow control or the outlet pipe.
2. **Post-initiation Phase:** As the acting head increases and the unit is covered by water, the Hydro-Brake® flow control enters the post initiation phase. Because of the tangential inlet flow and the fixed geometry of the unit, a vortex flow regime develops in the volute and outlet pipe. An aerated core, accompanied by substantial backpressure, effectively chokes the flow through the outlet aperture.

The Hydro brake is being increasingly used as a lower-impact technology for upstream runoff attenuation and flood defence with the largest schemes controlling flows in excess of 33 m³/s and holding back millions of cubic metres of water (Rocla, 2014).
The self-activating hydrodynamic vortex control technology was first invented and pioneered in the UK by Hydro-International over 30 years ago and has been used extensively in the UK to prevent surface water and watercourse flooding.

The Hydro-Brake® is most frequently used in small scale urban designs as part of WSUD approach, with the technology being scalable from small dispersed schemes with flow rates as low as a few litres per second to giant planning schemes with runoff attenuation and flow rate control for major urban areas.

Hydro-Brake® flood alleviation installations are part of Scotland's award winning and largest flood alleviation scheme. This scheme is known as the White Cart Water Flood prevention scheme which consists of 1,750 urban and suburban properties are protected. These properties include both residential and commercial land. In this scheme the flow control holds back water behind three dams creating storage basins on agricultural lands in the highlands of Glasgow. The water is released at a controlled rate using a Hydro-Brake® so that the flood defences are not overrun and properties downstream are protected.

The Hydro-Brake® is also part of other multi-award winning projects such as the following (Rocla, 2014);

1. A scheme in the Valley of River Douglas in Wigan which protects 610 properties from flooding in the city centre nearby using the same attenuation technique outlined in the White Cart Flood Prevention scheme (Rocla, 2014).

2. River Gaunless in Co. Durham. This scheme consists of smaller communities of 70-300 properties which have benefited. For example at Portpatrick, Argyllshire, which was subject to flash flooding in the streets is now protected due to the introduction of the Hydro-Brake®. Other small community areas in the UK that have been protected from flash flooding due to the introduction of the Hydro-Brake® are that of the Weedon Bec in Northants and Devil's Bridge near Sheffield (Rocla, 2014).

The use of the Hydro-Brake® throughout the UK has also become increasingly used to a smaller scale to achieve more manageable flows and aid limited space in already existing drainage systems and urban design (Andoh et al, 2002). An example of this is a residential development in London, which had to small of available space to allow for the planned storage tanks. To combat this the Hydro-Brake® along with the downstream defender and storm cell unit were used and these significantly improved performance characteristics and enhanced attributes compared with conventional BMP's.
(Deahl et al., 2002). It shows that these methods often offer a flexible integrated way to treat, contain and control stormwater specific to urban areas when space is a major constraint (Andoh et al, 2002).

The Hydro-Brake® has also been used for unique specific needs such as the 'Eden Project' in Cornwall (Andoh et al, 2002). This project had dilemmas on how to successfully control, capture and re-use rainwater for spraying in the constructed rainforest. The installation of the Downstream Defender and a Hydro-Brake® to regulate the flows through the treatment device into purpose built storage tanks (Andoh et al, 2002). Due to the site being a former clay pit, usual infiltration methods could not be used and therefore a large scale technical device was installed to remove contaminants from the flow.

It is these schemes that have led to his undertaken study of using multiple Hydro-Brakes® in the one system to further flood prevention and optimise WSUD.

2.8 On Site Detention
As the storage capacity of any drainage system is limited, on-site detention (OSD) is a method of ensuring runoff does not cause flooding downstream of a development. On site stormwater detention provides storage of stormwater run-off. The significance of onsite detention is the ability to control run-off rates and volume to ensure downstream systems are not overloaded past capacity (Sydney Water, 2014). OSD is not the only way to ensure that developments do not occur flooding, however when an area is already urbanised it is the most practical (Sydney water, 2014).

OSD is regulated by the local councils through its powers to control developments, with most new developments required to have OSD. Size and dimensions vary depending on the catchment size and impervious area of the developments. OSD facilities must be adequately maintained to continue to function correctly. An example of an OSD system can be found on the Upper Parramatta River catchment. Flood retarding basins have been built, which act in a similar way to large OSD systems by temporarily detaining water behind a wall and releasing the water at a controlled rate (Sydney Water, 2014).

A common factor with all OSD systems is the downstream space requirement and up until now OSD modelling in Australia has been far too simplistic producing single, end of catchment solutions which only passes the benefits to the downstream network. To maximise the utilisation of the drainage system during small storm events (20 year ARI and below) Hydro-Brake® flow regulators can be used at systematic positions along the drainage line in order to utilise the drainage network as an OSD system (Bryant, unknown), as has been the case in the UK for years.
2.9 Conclusion
The literature review shows an opportunity to study how a flow control device such as the Hydro-Brake® can improve WSUD practices in Australia.

Using the findings from this literature review, an investigation into adapting upstream OSD systems into drainage lines via the use of the Hydro-Brake® was developed.
Chapter 3: Methodology

3.1 Introduction
The design of any project requires considerable attention to research methods and the proposed data analysis methods. Therefore this chapter covers the technical design and attributes of all the testing apparatus used for this research thesis. Both computer modelling and hydraulic laboratory investigations are explained. This chapter covers hydraulic capacity testing, computer stormwater modelling for both single and multiple catchment peak inflows and the laboratory verification tests.

3.2 Computer Modelling
A major component of this research study was the use of XPSWMM in order to undertake a full hydrological and hydraulic analysis of the Hydro-Brake as a means of onsite detention and runoff attenuation. XPSWMM was the basis for catchment runoff calculations, peak flows, pipe geometry, and hydraulic performance of both the Hydro-Brake® and the pipeline throughout the system in order to determine flood risks and the overall successfulness of the design perimeters.

Below a description on each of the steps in the computer modelling is outlined, along with the testing configuration for all calculations.

3.2.1 Hydrology (Runoff)
The first phase of the computer modelling was to determine the runoff characteristics (peak flows into each of the proposed catchments) for the test area. These characteristics were designed in line with the Maroochy Shire council integrated water management guidelines and the Australian Rainfall and Runoff Guidelines 1987 as discussed in the literature review chapter. The following infiltration parameters were used:-

- Initial Losses: 0mm/hr
- Continuing Losses: 2.55mm/hr (0mm/hr for impervious areas)

To generate hydro-graphs and calculate peak discharge from drainage basin runoff, Laurenson’s method of rafting was used. This method is an extension of the commonly used rational method, specifically designed for stormwater modelling software such as XPSWMM.

3.2.2 Hydraulics
The second phase of this research was to undertake a full hydraulic analysis of a drainage system being used by multiple catchments of differing size. Once the peak flows into the catchments were calculated by undertaking a hydrological analysis due to rainfall events, XPSWMM was used to determine the peak flows through the Hydro-Brake® along with the head levels inside the pipes.
To allow a constant testing method, the pipe work was set as 100m lengths between each Hydro-Brake® with a constant slope of 1%. A manning's value of 0.014 was used for all concrete pipes. This allowed for a real-world investigation through hydraulic modelling.

The outcome of the hydraulic analysis was to determine the feasibility of using the Hydro-Brake® as a runoff attenuation and flow control device along with the ability to use already existing pipe work as storage.

**3.2.3 The Testing Configurations**
This investigation was undertaken using a hypothetical drainage system with both a single upstream inflow and multiple inflow points in a sub-catchment design. For all models, the catchment sizes were based on their impervious area, with a model minimum of 30m² pervious area used for all tests. A hydraulic time stamp of two hours was used in order to fully understand the effects of the short duration rainfall.

Following Hydro-Brake® performance curves generated from the in laboratory testing, hydrological and hydraulic modelling was undertaken using XPSWMM. The aim was to investigate the effect of using the Hydro-Brake® flow control over key points of a catchment to produce a more efficient design. In order to do this the Hydro-Brake® was tested under a full range of storm events with differing catchment characteristics. The testing for this began with a single catchment, multiple Hydro-Brake® system before studying the effects of multiple catchment inflows.

**3.2.3.1 Single Catchment Inflow Drainage System**
This modelled hypothetical drainage system had a single upstream inflow based on rainfall events in the Maroochy shire area. The test consisted of three lengths of 100m, 375mm concrete pipes. Each of the pipes were placed at a 1% slope and a constant Manning's value of 0.014 was adopted. Each of the three 100m lengths were separated by a Hydro-Chamber which housed a RUF-002 Hydro-Brake®, which have been assumed to have no storage capacity. The upstream catchment size was increased by 100m² until failure was reached. Failure has been considered the point in which storage limits in the drainage pipe had reached capacity. Each storm ARI tested was considered to run for 15 minutes due to the effect of short duration, high impact storms being common problems for SQIDs. The study investigated outcomes for peak inflow, head levels throughout the system and exiting flow rates.

**3.2.3.2 Multiple Inflow Drainage System**
This modelled hypothetical drainage system had a multiple sub-catchment inflow based on rainfall events in the Maroochy shire area. The test consisted of three lengths of 100m, 375mm concrete pipes. Each of the pipes were placed at a 1% slope and a constant manning's value of 0.014 was
adopted. Each of the 100m lengths were separated by a Hydro-Chamber which housed a RUF-002 Hydro-Brake®, which have been assumed to have no storage capacity. These points were also considered to take the place of a manhole in which further inflows occurred. As was the case with the single inflow system, the catchment sizes were increased systematically until failure. Failure has been considered the point in which storage limits in the drainage pipe had reached capacity. Each storm ARI tested was considered to run for 15 minutes due to the effect of short duration, high impact storms being common problems for SQIDs. This study investigated outcomes for peak inflows, head levels throughout the system and the exiting flows.

3.3 Physical Laboratory Based Modelling
The second aspect of this research was raw testing the Hydro-Brake® in a laboratory setting. This was broken into two aspects; an initial hydraulic capacity investigation and a drainage system verification.

3.3.1 The Hydro-Chamber
To undertake the in lab modelling, some Hydro-Chambers were needed. These chambers consisted of a 1.2m diameter tank with a height of 1.5m made of Plastream. The RUF-002 unit was installed 100mm from the bottom of these tanks. 600mm length of DN300 Perspex outlet pipes were used for each of the tanks.

Figure 15 - Individual 'Hydro-Chamber Tank'
3.3.2 Hydraulic Capacity Testing
The first step of this investigation was to test the Hydro-Brake® on hand (RUF-002) and make a comparison between the data sent by the manufacturer and the practical testing in the laboratory. This was undertaken due to the entire investigation revolving around the unique "s" shaped hydrograph shown in the literature review meaning that a verification of these hydro-graphs were needed in order to give validity to the testing undertaken.
To determine the hydraulic capacity of the Hydro-Brake® units in order to use their flow conditions in the XPSWMM modelling, one of the hydro chambers was connected to a reticulating tank via the means of a Fly GT submersible pump. A piezometer was used to measure the head level at all times. This was set up from the bottom of the Hydro-Brake® opening. The flow into the tank was adjusted until a constant head was reached at varying points between 0-1200mm. As it was necessary to determine where the Hydro-Brake® would begin to choke the flow, 100mm intervals were used. This information could then be portrayed in a hydrograph to determine the resultant performance curves. When the head became constant, hydraulic law states that the outflow is equal to the inflow, allowing of the outflow to be recorded.

![Figure 18 - Piezometer](image)

The Hydraulic capacity tests consisted of an investigation into four different orifice size openings on the Hydro-Brake® to enable an investigation into how they effected the hydraulic performance. The chosen orifice opening sizes tested were; 65mm, 104mm, 121mm and the full open Hydro-Brake® measured at 170mm.

### 3.3.3 Laboratory Verification Tests
A primary part of the physical modelling of this study was the design and construction of a fully functioning performance testing rig (PTR). The PTR included a fully adjustable flow distribution which enabled the model results obtained from XPSWMM to be verified in a laboratory setting. The flow distribution system allowed the performance of the attenuation and individual flow control devices to be investigated under a range of rainfall levels as specified by the Bureau of Meteorology. Perspex
outlet pipes were utilized from each of the storage tanks to get a visual real time view of the outflow from the Hydro-Brake®. A key difference for the tanks in this test was that the height was reduced to 800mm to ensure OH&S standards under the testing apparatus.

Each of the Hydro-Chambers were set at incremental levels of 800mm so that a free outflow into the downstream tanks could occur. Peak inflows were pumped into the upstream tank, with a free outfall downstream. The inflows were generated for a 15 minute period for each test to represent the modelled storms.

![Figure 19 - PTR Configuration](image)

The water required to operate the PTR was received from a 10kL tank via reticulated water supply system. A flow meter was used to measure the flow into the model components, with the head in each of the tanks and exiting flow rates recorded. This allowed for an analysis and verification of the XPSWMM stormwater modelling. A major part of this study, and that of WSUD is that of the drainage pipe being used.
Chapter 4: Results
This chapter gives all results obtained from the hydraulic capacity testing as well as both of the XPSWMM investigations as outlined in chapter 3. This chapter then gives the findings of the laboratory based hydraulic analysis as a verification method.

4.1 Hydraulic Capacity Testing Results

4.1.1 65mm Orifice
The first of the orifice openings that was tested was 65mm, the following results were obtained:

![65mm Orifice Performance Curve](image-url)

*Figure 20 - 65mm Orifice Performance Curve*
4.1.2 104mm Orifice
The second choice of orifice openings that was tested was 104mm, the following results were obtained.

Figure 21 - 104mm Orifice Performance Curve

4.1.3 121mm Orifice
The last orifice opening before testing the open Hydro-Brake® that was tested was 121mm, the following results were obtained:

Figure 22 - 121mm Orifice Performance Curve
4.1.4 Open Hydro-Brake® (170mm Orifice)
The last of the hydraulic capacity tests was the open Hydro-Brake®. The following results were obtained:

![Hydrobrake Performance Curve](image)

**Figure 23 - Open Hydro-Brake® Performance Curve**

4.2 XPSWMM Modelling Results

4.2.1 Single Catchment Inflows Results

<table>
<thead>
<tr>
<th>Table 3 - 100m² Hydraulic Conditions</th>
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</thead>
<tbody>
<tr>
<td>Peak Inflow</td>
</tr>
<tr>
<td>1 year 15 Minute ARI</td>
</tr>
<tr>
<td>--------------------------------------</td>
</tr>
<tr>
<td>3.1</td>
</tr>
<tr>
<td>3.9</td>
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<tr>
<td>4.9</td>
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<tr>
<td>5.4</td>
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<td>6.7</td>
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<tr>
<td>Pipe 1 Effective Head</td>
</tr>
<tr>
<td>0.13</td>
</tr>
<tr>
<td>0.14</td>
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<tr>
<td>0.16</td>
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<td>0.19</td>
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<td>0.23</td>
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<td>Hydro-Brake 1 Flow Rate</td>
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<tr>
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<td>3.4</td>
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<tr>
<td>3.7</td>
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<tr>
<td>4</td>
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<tr>
<td>4.3</td>
</tr>
<tr>
<td>Pipe 2 Effective Head</td>
</tr>
<tr>
<td>0.13</td>
</tr>
<tr>
<td>0.13</td>
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<td>0.14</td>
</tr>
<tr>
<td>0.16</td>
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<tr>
<td>0.18</td>
</tr>
<tr>
<td>Hydro-Brake 2 Flow Rate</td>
</tr>
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<td>2.7</td>
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<tr>
<td>3.1</td>
</tr>
<tr>
<td>3.5</td>
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<tr>
<td>3.8</td>
</tr>
<tr>
<td>Exiting Flow Rate</td>
</tr>
<tr>
<td>2.7</td>
</tr>
<tr>
<td>3.1</td>
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<tr>
<td>3.5</td>
</tr>
<tr>
<td>3.7</td>
</tr>
<tr>
<td>3.8</td>
</tr>
</tbody>
</table>

Table 1 shows the hydraulic outcomes resulting from a 100m² catchment. The effective head includes the effect of the pipe slope.
Table 4 - 200m² Hydraulic Conditions

<table>
<thead>
<tr>
<th></th>
<th>1 year 15 Minute ARI</th>
<th>5 Year 15 Minute ARI</th>
<th>10 Year 15 Minute ARI</th>
<th>20 Year 15 Minute ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Inflow</td>
<td>5.7</td>
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<td>10.5</td>
<td>12.1</td>
</tr>
<tr>
<td>Pipe 1 Effective Head</td>
<td>0.2</td>
<td>0.9</td>
<td>1.14</td>
<td>1.22</td>
</tr>
<tr>
<td>Hydro-Brake 1 Flow Rate</td>
<td>4.3</td>
<td>6</td>
<td>6.7</td>
<td>6.7</td>
</tr>
<tr>
<td>Pipe 2 Effective Head</td>
<td>0.16</td>
<td>0.21</td>
<td>0.36</td>
<td>0.72</td>
</tr>
<tr>
<td>Hydro-Brake 2 Flow Rate</td>
<td>3.7</td>
<td>4.3</td>
<td>4.3</td>
<td>5.3</td>
</tr>
<tr>
<td>Pipe 3 Effective Head</td>
<td>0.14</td>
<td>0.19</td>
<td>0.15</td>
<td>0.19</td>
</tr>
<tr>
<td>Hydro-Brake 3 Flow Rate</td>
<td>3.7</td>
<td>3.9</td>
<td>3.7</td>
<td>4</td>
</tr>
<tr>
<td>Exiting Flow rate</td>
<td>3.7</td>
<td>3.9</td>
<td>3.7</td>
<td>4</td>
</tr>
</tbody>
</table>

Table 2 shows the hydraulic outcomes resulting from a 200m² catchment. The effective head includes the effect of the pipe slope.

Table 5 - 300m² Hydraulic Conditions

<table>
<thead>
<tr>
<th></th>
<th>1 year 15 Minute ARI</th>
<th>5 Year 15 Minute ARI</th>
<th>10 Year 15 Minute ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Inflow</td>
<td>8.3</td>
<td>13.7</td>
<td>15.4</td>
</tr>
<tr>
<td>Pipe 1 Effective Head</td>
<td>0.53</td>
<td>1.27</td>
<td>1.3</td>
</tr>
<tr>
<td>Hydro-Brake 1 Flow Rate</td>
<td>4.3</td>
<td>6.8</td>
<td>7.1</td>
</tr>
<tr>
<td>Pipe 2 Effective Head</td>
<td>0.16</td>
<td>1.04</td>
<td>1.05</td>
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<tr>
<td>Hydro-Brake 2 Flow Rate</td>
<td>3.8</td>
<td>6.7</td>
<td>6.7</td>
</tr>
<tr>
<td>Pipe 3 Effective Head</td>
<td>0.15</td>
<td>0.35</td>
<td>0.68</td>
</tr>
<tr>
<td>Hydro-Brake 3 Flow Rate</td>
<td>3.7</td>
<td>4.3</td>
<td>5.34</td>
</tr>
<tr>
<td>Exiting Flow rate</td>
<td>3.7</td>
<td>4.3</td>
<td>5.34</td>
</tr>
</tbody>
</table>

Table 3 shows the hydraulic outcomes resulting from a 300m² catchment. The effective head includes the effect of the pipe slope.

Table 6 - 500m² Hydraulic Conditions

<table>
<thead>
<tr>
<th></th>
<th>1 year 15 Minute ARI</th>
<th>5 Year 15 Minute ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Inflow</td>
<td>13</td>
<td>17.3</td>
</tr>
<tr>
<td>Pipe 1 Effective Head</td>
<td>1.25</td>
<td>1.33</td>
</tr>
<tr>
<td>Hydro-Brake 1 Flow Rate</td>
<td>6.8</td>
<td>7.7</td>
</tr>
<tr>
<td>Pipe 2 Effective Head</td>
<td>1.04</td>
<td>1.08</td>
</tr>
<tr>
<td>Hydro-Brake 2 Flow Rate</td>
<td>6.7</td>
<td>6.84</td>
</tr>
<tr>
<td>Pipe 3 Effective Head</td>
<td>0.36</td>
<td>1.04</td>
</tr>
<tr>
<td>Hydro-Brake 3 Flow Rate</td>
<td>4.3</td>
<td>6.7</td>
</tr>
<tr>
<td>Exiting Flow rate</td>
<td>4.3</td>
<td>6.7</td>
</tr>
</tbody>
</table>

Table 4 shows the hydraulic outcomes resulting from a 500m² catchment. The effective head includes the effect of the pipe slope.
Table 7 - 700m² Hydraulic Conditions

<table>
<thead>
<tr>
<th>1 Year 15min ARI</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Peak Inflow (L/s)</td>
<td>17.3</td>
</tr>
<tr>
<td>Pipe 1 Effective Head (m)</td>
<td>1.36</td>
</tr>
<tr>
<td>Hydro-Brake® 1 Flow Rate (L/s)</td>
<td>8.1</td>
</tr>
<tr>
<td>Pipe 2 Effective Head (m)</td>
<td>1.11</td>
</tr>
<tr>
<td>Hydro-Brake® 2 Flow Rate</td>
<td>6.8</td>
</tr>
<tr>
<td>Pipe 3 Effective Head (m)</td>
<td>1.04</td>
</tr>
<tr>
<td>Hydro-Brake® 3 Flow Rate</td>
<td>6.7</td>
</tr>
<tr>
<td>Exiting Free Outflow</td>
<td>6.7</td>
</tr>
</tbody>
</table>

Table 5 shows the hydraulic outcomes resulting from a 700m² catchment. The effective head includes the effect of the pipe slope.

Figures 24-31 have resulted from a 1 year 15 minute ARI storm.

![Figure 24 - Inflow into 700m² Catchment](image)

![Figure 25 - 700m² First Pipe Effective Head](image)
Figure 26 - 700m$^2$ Second Pipe Effective Head

Figure 27 - 700m$^2$ Third Pipe Effective Head

Figure 28 - 700m$^2$ Flow Rates through Hydro-Brakes®

Figure 29 - 700m$^2$ First Pipe Long Section
4.2.2 Multiple Catchment Inflow Results

The below results are from the last possible underground stormwater drainage system before failure. For all other results see chapter 8.

<table>
<thead>
<tr>
<th>Table 8 - 400m² - 300m² -300m², 1 Year 15 Minute ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Year 15 Minute ARI</td>
</tr>
<tr>
<td>Catchment 1 Peak Inflow (L/s)</td>
</tr>
<tr>
<td>Pipe 1 Effective Head (m)</td>
</tr>
<tr>
<td>Hydro-Brake® 1 Flow Rate (L/s)</td>
</tr>
<tr>
<td>Catchment 2 Peak Inflow (L/s)</td>
</tr>
<tr>
<td>Pipe 2 Effective Head (m)</td>
</tr>
<tr>
<td>Hydro-Brake® 2 Flow Rate (L/s)</td>
</tr>
<tr>
<td>Catchment 3 Peak Inflow (L/s)</td>
</tr>
<tr>
<td>Pipe 3 Effective Head (m)</td>
</tr>
<tr>
<td>Hydro-Brake® 3 Flow Rate (L/s)</td>
</tr>
<tr>
<td>Exiting Flow Rate (L/s)</td>
</tr>
</tbody>
</table>

The above table shows the hydraulic outcomes resulting from a 1 Year 15min ARI storm. The effective head includes the effect of the pipe slope.
Figure 30 - 400m² - 300m² - 300m² Catchment Flow Rates through Hydro-Brakes

Figure 31 - 400m² - 300m² - 300m² Catchment Long Section Pipe 1

Figure 32 - 400m² - 300m² - 300m² Catchment Long Section Pipe 2

Figure 33 - 400m² - 300m² - 300m² Catchment Long Section Pipe 3
4.3 Lab Testing and Verification
The below results were obtained from the laboratory hydraulic testing and verification process.

**Figure 32 - Head in each Tank Comparison**

**Figure 33 - Outflow in Each Tank Comparison**
Chapter 5: Discussion
This chapter discusses all the findings from the results. The significance of these findings and how they relate to one another are investigated, with future potentials outlined.

5.1 Hydraulic Capacity of the Hydro-Brake®
The initial part of this investigation was to test the hydraulic capacity of the Hydro-brake® and compare this to the results supplied by Rocla. This testing was broken into 4 different tests all with differing orifice sizes. The essential difference between orifice sizes was the flow rate at all points of the hydrograph.

5.1.1 65mm Orifice
The 65mm orifice hydrograph showed a decrease in the outgoing flow rate as the head increased, however the desired ‘s’ shape was not very pronounced with the flow control process of this model occurring between 100 and 300 head. The increase in flow rate after this hydraulic efficiency point was also very sudden and pronounced resulting in the almost vertical curve.

5.1.2 104mm Orifice
The 104mm orifice hydrograph from this shows a more pronounced ‘s’ shape to the 65mm opening, with a larger decrease in outing flow at a higher head level. The hydraulic efficiency point has also shifted further upwards. The increase in flow rate once past the hydraulic efficiency point has also been slowed with the slope of the graph greatly decreased to that of the 65mm orifice.

5.1.3 121mm Orifice
The 121mm orifice hydrograph shows the largest choke out of the orifice cuts with a flow reduction of over 1L/s as shown by the larger 's' shape. The flow rate increase after this point follows an almost linear increase - much steadier then the previous tests.

5.1.4 Open Hydro-Brake®
As had been the case with the previous tests, the 121mm orifice opening resulted in an 'S' shape at around 200mm of head. The reduction of the flow rate at this point of hydraulic efficiency is far more pronounced than that of the orifice tests. The flow rate increase after this point was once again essentially linear.

5.1.5 Analysis
Apart from that of the 104mm orifice opening all the lowest flow rates occurred at an approximate head level of 200mm, whilst the highest flow rate occurred at the highest point of testing for each of the orifice sizes. It can be concluded that the opening size has no impact on the hydraulic performance of the Hydro-Brake®, rather just the actual flow rate. The significance of the hydraulic efficiency point is that is this point where the Hydro-Brake® utilizes the pressure drop through the system to further
choke flow compared to that of a normal orifice. It is therefore at this point where head will increase faster which allows the utilization of the unused storage in upstream drainage pipes and mitigates the need for downstream storage points. Once the head and therefore the pressure increases, the Hydro-Brake® inherently allows the flow to increase through the unit to avoid flooding. This process is shown in figure 34.

![Figure 34 - Hydro-Brake® and Normal Orifice Opening Comparison](image)

5.1.1 Conclusions and Applications
Figure 12 depicts four tested Hydro-Brake® units, with all showing significant "s" shaped hydro-graphs. In order to expand the study, it was necessary to determine which of the four Hydro-Brake® unit’s best represented the hydraulic capacity results. The hydrograph obtained from the open Hydro-Brake® hydraulic testing is best represented by the larger small unit tested by the manufacturer.

Therefore these results were used for the XP modelling, with evidence showing the performance of the Hydro-Brake® is in line with the manufactures claims.

Whilst the differing orifice sizes were investigated to give a range of hydraulic conditions it was determined ineffectual to the investigation as the units head is calculated from the invert level of the pressure drop, not from the orifice. No matter what orifice size the vortex motion throughout the chamber regardless of outlet size. This resulted in the Hydro-Brake and the orifice acting as two separate systems, with the Hydro-Brake® undergoing its pre and post initiation phases and then the orifice reducing the flow.
5.2 Single Catchment Inflows

5.2.1 XPSWMM Modelling
The initial investigation of this research was to test the effect of the Hydro-Brake® flow control device as a means for onsite detention by enabling better use of the seldom used storage in drainage pipes. The primary step of this modelling was undertaken using a single catchment system with three key detention points throughout the drainage with data on flow rates, head levels, peak discharge, and peak inflow. Due to the Hydro-Brake® being a small scale unit, the catchment sizes have been restricted to resemble small areas - such as a car park, to be able to utilise the capacity of the flow control device. This testing was undertaken until the system failed on a 1 year 15 minute ARI storm burst. Appendix 1 shows the raw data for these points.

5.2.1.1 100m² Catchment
The initial catchment size that was modelled was 100m². Peak flow rates resulting from storms over such a small area were insignificant, with the Hydro-Brake® remaining in its pre-initiation phase until a 20 year 15 Minute ARI Storm. It can therefore be concluded that the addition of a flow control device such as the Hydro-Brake® on such a small area is ineffectual due to the minimal need for detention. This was as expected as a 100m² would usually be considered as a sub-catchment as part of an overall catchment design.

5.2.1.2 200m² Catchment.
Unlike the 100m² catchment, storms as low as a 1 year 15 Minute ARI burst caused conditions throughout the system in which the Hydro-Brake® entered its post-initiation phase. As discussed earlier, through testing the hydraulic capacity of the Hydro-Brake® device it was determined that flow control due to the vortex action in the system begins at an approximate head level of 0.2m. Table 10 shows a peak inflow of 5.7l/s caused by a 1 year 15 Minute ARI storm burst, with each flow control point choking the flow to an eventual peak discharge of 3.7l/s. This choking effect caused longer residence times within the drainage pipes, causing the head level to rise and the storage space to be used. This system worked without the need for downstream storage or bigger piping system up until a 20 year 15 Minute ARI Storm.

5.2.1.3 300m² Catchment.
The 300m² catchment, like the 200m² utilized the hydraulic capacity of the Hydro-Brake® at storm bursts as small as that of a 1 year 15 minute ARI, with each detention point reducing the flow through to the next piping section whilst causing longer residence times in the upstream pipes. Due to the
higher peak inflows, more of the empty storage capacity of the drainage piping was needed. This system worked successfully up until a 10 year 15 Minute ARI storm burst.

5.2.1.4 500m$^2$ Catchment
The 500m$^2$ catchment resulted in significantly larger peak inflows to that of previously modelled catchment sizes. In this system at a 1 year 15 Minute ARI event, the peak inflow was modelled to be 13l/s. At this point the significance of using the Hydro-Brake® in conjunction with each other is apparent, with the first two detention points slowing the flow enough for the third detention point to bring the flow down to 6.8l/s whilst not needing any downstream storage showing the advantage of utilising the upstream pipe storage. This system worked successfully up until a 5 year 15 Minute ARI storm burst.

5.2.1.5 700m$^2$ Catchment
The 700m$^2$ catchment resulted in a peak inflow of 17.3 L/s at a 1 year 15 minute ARI storm burst. Much like the 500m$^2$ catchment, the first two detention points slowed the flow enough for the third to significantly decrease peak discharge to 6.7 L/s.

5.2.2 Conclusions
Analysis of the single catchment inflow drainage design have shown that while at low peak inflows, and small catchment areas the potential benefits of using the Hydro-Brake® as a means of on-site detention are probably limited. These single inflow modelling results have shown with larger catchment sizes and peak inflows a significance of the Hydro-Brake® was evident. By using these flow control devices in conjunction with each other at sequential points can reduce the need for downstream storage by using the upstream underground stormwater drainage lines. A further advantage to this was the exiting flow level can be reduced to far more workable levels to travel into the SQID’s downstream.

The literature review chapter shows the tendency for the Hydro-Brake® to utilise flow control as the upstream pipes surpass 200-300mm head. Whilst the head in the pipe surpasses this point, it is this tendency that results in the pipes approaching in capacity before the pressure becomes too great and outflow begins to accelerate.

As there is only one inflow, each point of attenuation results in a decreased flow rate through the system. In the lower storm events this results in the head level through the downstream pipes becoming lower then what is needed for the Hydro-Brake® to move past its pre-initiation phase. It is therefore necessary in these systems to use three detention points, with that advantage of onsite detention only being utilized at upstream points. However for the purpose of this investigation three points were used for each test.
5.3 Multiple Sub-Catchment Inflows

5.3.1 XPSWMM Modelling
The next stage of the investigation followed essentially the same process as section 6.2. The key difference with this system was the fact that each of the detention points were classified as an open manhole, in which more flows could be entered into the system at downstream points. The catchment sizes were restricted due to the small Hydro-Brake® being used for testing purposes. The upstream catchment was considered the largest in all circumstance with the sequential downstream sub-catchments differing in smaller sizes.

5.3.1.1 100m² Main Catchments.
The initial testing configuration began with a 100m² main catchments followed by two smaller sub catchments. The following systems were studied;

- **100m²-50m²-50m²** - Much like the smallest catchment with the single inflow systems, this configuration did not fully utilise the hydraulic performance of the Hydro-Brake® until the 20 year 15 minute ARI storm, in which the added inflow from the last catchment along with the upstream inflow caused the last detention point to successfully choke the outflow.

- **100m²-100m²-50m²** - At low flow rates, the Hydro-Brake® was once again not utilized correctly, however at storms greater than a 2 year 15 minute ARI the attenuation process began, with flow being held back into the upstream piping system. This system successfully handled the inflow up to a 10 year 15 minute ARI storm burst.

- **100m²-100m²-100m²** - Under this configuration the Hydro-Brake® was initialised at the 1 year 15 minute ARI storm, however only at the last detention point. This configuration was successful up to a 5 year 15 minute ARI storm.

5.3.1.2 200m² Main Catchments
The same testing was undertaken with as mentioned above with the initial catchment being set at 200m². The following catchment systems were studied;

- **200 m²-50 m²- 50 m²** - Whilst the initial flow rate was larger than any of the flows in the previous tests, the 50m² catchments resulted in such insignificant peak inflows that the use of onsite detention was ineffectual.

- **200 m²-100 m²-50 m²** - This catchment system used the hydraulic capacity of the Hydro-Brake® from the initial 1 year 15 Minute ARI storm test with head levels through the piping system remaining between 200-400mm. This system was able to support flows from storms up to a 5 year 15 Minute ARI storm before reaching capacity.
• **200 m²-100 m²- 100 m²** - At early storm events it can be seen that this configuration acts in the same way as the previous system, with similar outflows being evident in all areas of systems. It can be seen that even with the multiple inflow points that the system is successfully attenuating the flow to a peak discharge of 6.8L/s. This system was able to support flows from storms up to a 5 year 15 Minute ARI burst.

### 5.3.1.3 300m² Main Catchments

The same method of modelling was used for the 300m² catchment, with each of the sub-catchments being set at smaller sizes to the upstream main catchment. The following catchment systems were investigated:

• **300 m²-50 m²- 50 m²** - Like the previous tests which consist of a 50m² catchment, whilst the initial peak inflow was larger than that of previous tests, due to the small inflows from the 50m² catchments, the only real need for onsite detention was at the first hydro-chamber. This catchment configuration supported flows up until the 5 year 15 minute ARI storm event.

• **300 m²-100 m²- 100 m²** - This system used the hydraulic capacity of the Hydro-Brake® at all detention points throughout the system with head levels starting at 560mm and falling to 490mm for a 1 year 15 minute ARI storm burst.

• **300 m²-200 m²-100 m²** - This configuration followed a similar trend to that of the test above, with slightly higher head levels resulting at each point over the testing apparatus. This system successfully stored inflow from storm events up to and including a 5 year 15 minute ARI.

• **300 m²-200 m²-200 m²** - At this catchment configuration the head levels through all the pipes increase significantly, with the 5 year 15 minute ARI storm approaching failure. However as was with the previous tests this system successfully stored flows up to and including a 5 year 15 min ARI.

• **300 m²-300 m²- 200 m²** - At this catchment configuration the system cannot successfully handle flows from a 5 year 15 minute ARI storm, with the storage capacity of the second and third pipe in the system being overrun.

• **300 m²- 300 m²- 300 m²** - At this catchment configuration the system cannot successfully handle flows from a 2 year 15 minute ARI storm event with the second pipe in the system breaching its capacity.

### 5.3.1.4 400m² Main Catchment

The above analysis has shown that the system is beginning to fail more rapidly. The following tests were undertaken using a main upstream catchment of 400m²;
- **400 m²-200 m²-200 m²** - For the point of this modelling, the downstream catchments were started at larger sizes due to the objective being to find the configuration at which the system fails. For this the system successfully stores flows from storm events up to and including a 2 year 15 minute ARI.
- **400 m²-300 m²-200 m²** - At this configuration the system fails at a 2 year 15 minute ARI storm, failing in the second pipe.
- **400 m²-300 m²-300 m²** - At this point only the 1 year 15 minute ARI storm event was investigated, with the peak outflow of the system being reduced to 7.9L/s even though all of the peak inflows were larger than this.
- **400 m²-400 m²-100 m²** - At this catchment configuration, the system fails under a 1 year 15 minute ARI storm with the second pipe reaching its storage capacity.

### 5.3.2 Conclusions

Analysis of the results show at low flow rates, the system follows a similar trend to that of the single inflow system. A key difference however is how rapidly the head in the system increases due to the multiple inflows. Permutations with low downstream inflows recovered efficiently, but larger systems reached capacity far more rapidly than that of the single catchment system.

It can be seen from the results that even the small Hydro-Brake® unit being studied can result in many permutations of successful flow control, with the upstream pipes approaching capacity whilst the exiting flow decreases throughout the system.

### 5.4 Laboratory Testing

#### 5.4.1 Analysis

Analysis of the in lab results shows that at the lower peak inflows, the Hydro-brake® is less effective with only minimal flow control occurring. It was deemed that at such low flow rates attenuation and flow control is probably not needed as the system easily handles the input flow volumes. The transition between peak inflows of 4L/s and 5L/s show the advantage of flow control throughout the system. The laboratory results (figure 32 and 33) show at 4L/s inflow, that the system experiences a fairly constant head level of around 200mm with little to no change occurring in outflow. However, at 5L/s the Hydro-Brake® automatically activates its post initiation phase, causing the flow control process to happen through the pressure drop and vortex motion through the system. This resulted in significant head increase in both the first and second storage tank with the third tank not passing the Hydro-Brakes pre-initiation phase as can be seen in figure 35 and 36 respectively.
The testing also shows the flow out of the system decreased throughout, with the peak flow dropping by 15.6%. This raises a significant point in that the flow rate has been brought down to a more workable level, with the upstream storage being utilised. As the tanks in the lab testing are a representation of upstream drainage piping, this verifies that by using systematically placed Hydro-Brake®s, the naturally occurring storage space within drainage points can be utilised.
5.4.2 Considerations and Conclusions
By using the Hydro-Brake® at systematically placed points throughout the system the upstream storage of the drainage has been used whilst lowering the peak discharge out of the drainage configuration.

The testing in this investigation has been undertaken using the small scale Hydro-Brake® and relatively small piping. For a real world situation it would be recommended to use the larger models and larger drainage piping to would allow greater catchment sizes and peak flow rates throughout the system.
Chapter 6: Conclusion and Recommendations

This chapter concludes on the points raised in the discussion, with emphasis on the trends in the data, the reduction in OSD and how the Hydro-Brake® could relate to future WSUD principles. Recommendations are given for the future use of the Hydro-Brake® as well as how future study could be undertaken to further investigate the significance of the Hydro-Brake® as an integral part of all WSUD design.

6.1 Conclusion

Hydraulic capacity testing and modelling of the Hydro-Brake® has proven its advantage over a standard orifice plate. The greatest advantage of the Hydro-Brake® is to allow free outflow with no flow control, until the system reaches a head of approximately 200mm. At head levels greater then 200mm the flow is effectively choked, with an 'S' shape curve evident in the resulting outlet hydrograph. With head levels less than 600mm flow control continues with diminishing effect. Above this head level, the Hydro-Brake® starts to function similar to a standard orifice plate which avoids exceeding the capacity of the drainage system. The results of this study verify performance claims made by the manufacturer and clearly demonstrate the feasibility of using the Hydro-Brake® as a flow control device to improve the performance of SQIDs and achieve WSUD objectives.

Stormwater modelling using XPSWMM was undertaken to investigate the unique hydraulic capacity of the Hydro-Brake®, by determining the effect of using systematically placed flow control devices throughout a drainage system. The study investigated a single inflow drainage system and multiple inflow sub-catchment design.

-For this testing the following design assumptions were adopted:

- 100m lengths of pipe between catchments or detention points.
- A Manning’s value of 0.014 for concrete pipes.
- A set slope of 1% for all drainage piping and a minimal 0.01% slope for catchments.
- The use of the Maroochy Shire area for rainfall data.
- All manholes and Hydro-Chambers have been assumed to hold 0 storage.

The results of the single inflow system testing demonstrated that the Hydro-Brake® can be used to effectively enable upstream pipes to be used as a storage medium. At low flow rates generated from small catchment sizes, the system followed a free flow pattern, with little flow control. At higher flow rates generated from larger catchment sizes, the Hydro-Brake's® unique 'S' shaped outflow hydrograph was shown to achieve greater flow attenuation - and control between head levels of
200mm and 600mm. The testing assumed that, no downstream storage was available with the only OSD and retention areas being the existing drainage lines.

Failure point for each test was attained when any of the upstream pipes exceeded its capacity. The following catchment sizes caused failure at the corresponding storms;

- 20 year 15 minute ARI storm burst. - 200m$^2$
- 10 year 15 minute ARI storm burst. - 300m$^2$
- 5 year 15 minute ARI storm burst - 500m$^2$
- 1 year 15 minute ARI storm - 700m$^2$

Following the same testing conditions, a multiple inflow sub-catchment system was investigated. The following catchment sizes caused failure at the corresponding storm;

- 20 year 15 minute ARI storm burst. - 100 m$^2$-100 m$^2$-50 m$^2$
- 10 year 15 minute ARI storm burst. - 100 m$^2$-100 m$^2$100 m$^2$ and 200 m$^3$-100 m$^3$-50 m$^2$
- 5 year 15 minute ARI storm burst. - 300 m$^2$-300 m$^2$-200 m$^2$
- 2 year 15 minute ARI storm burst. - 300 m$^2$-300 m$^2$- 300 m$^2$
- 1 year 15 minute ARI storm burst. - 400 m$^2$-400 m$^2$-100 m$^2$

To validate this investigation, a single inflow system was set up in the stormwater lab to investigate the validity of the XPSWMM modelling results.

This laboratory based investigation used water tanks to represent the potential storage volume in upstream drainage pipes. It was found that at low flow rates, little to no flow control occurred. However, at higher flow rates the inherent flow control ability of the Hydro-Brake® was utilised, causing the upstream tanks to approach their storage capacity. This also caused decreasing downstream flow volumes, while reducing the outflow rates by up to 15%.

The Hydro-Brake® was shown to successfully control flow and attenuate stormwater runoff. By systematically placing these units throughout a drainage system, on site detention may mitigate the need for downstream storage measures whilst lowering exiting flow volumes, allowing the application to minimise size requirements of successful WSUD SQIDs.

6.2 Recommendations
This section gives recommendations for the Hydro-Brake® based on the results obtained throughout the study. Recommendations for further testing and analysis are given.
6.2.1 Use of the Hydro-Brake®
This study has concluded that the use of the Hydro-Brake® successfully mitigated the need for downstream storage and reduce peak discharge from a drainage system. It is recommended that the Hydro-Brake® be used in drainage systems in order to decrease the need for downstream storage and size restraints of WSUD potentially leading to huge economic benefits. The following design recommendations have been generated in order to assist with a real world application;

- Use the larger scale Hydro-Brake® to allow for larger flow rates and therefore catchment sizes.
- Adapt the pipe size to suit the drainage need - the larger the pipe, the more storage capacity.
- Systematically place the Hydro-Brake® in order to maximise the attenuation process.
- Place before BMP’s such as an infiltration trench to increase the success rate of natural, eco-system friendly WSUD.

The significance of these points is that by successfully using the Hydro-Brakes in an already existing drainage line at systematic points can significantly reduce the need for downstream OSD systems resulting in huge benefits for the developer whilst still meeting council’s regulations for stormwater management and WSUD principles.

6.2.2 Further Testing and Analysis
To fully analyse the Hydro-Brake® it is recommended that further in lab testing be undertaken. To do this a fully functional storm modelling pump would be needed to recreate model rainfall events. Larger space requirements would be needed to test an actual drainage system to get a comprehensive understanding of the benefits of using the Hydro-Brake® in WSUD.
Chapter 7: References

This section gives due credit to all sources of this research thesis. Without the relevant input from these sources the research could not have been undertaken successfully.


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Chapter 8: Appendix

8.1 Single Catchment Inflow XPSWMM Raw Data
This section of the Appendix gives all the direct results from the XPSWMM modelling for the single catchment inflows.

8.1.1 100m² Catchment

8.1.1.1 1 Year 15 Min ARI Storm

Figure 37 - Peak Inflow

Figure 38 - Head Pipe 1
Figure 39 - Head Pipe 2

Figure 40 - Flow Rates Through Hydro-Brakes

Figure 41 - Pipe 1 Long Section

Figure 42 - Pipe 2 Long Section
8.1.1.2 2 Year 15 Min ARI Storm

Figure 43 - Peak Inflow

Figure 44 - Head Pipe 1

Figure 45 - Head Pipe 2
Figure 46 - Flow Rates through Hydro-Brakes

Figure 47 - Pipe 1 Long Section

Figure 48 - Pipe 2 Long Section
8.1.1.3 5 Year 15 Min ARI Storm

Figure 49 - Peak Inflow

Figure 50 - Head Pipe 1

Figure 51 - Head pipe 2
8.1.1.4 10 Year 15 Min ARI Storm

Figure 52 - Flow Rate through Hydro-Brakes

Figure 53 - Pipe 1 Long Section

Figure 54 - Pipe 2 Long Section

Figure 55 - Peak Inflow
Figure 56 - Head Pipe 1

Figure 57 - Head Pipe 2

Figure 58 - Flow Rates through Hydro-Brakes
8.1.1.5 20 Year 15 Minute ARI Storm

Figure 61 - Peak Inflow

Figure 62 - Head Pipe 1
8.1.2 200m² Catchment

8.1.2.1 1 Year 15 Minute ARI Storm

Figure 67 - Peak Inflow

Figure 68 - Head Pipe 1

Figure 69 - Head Pipe 2
Figure 70 - Head Pipe 3

Figure 71 - Flow Rate Through Hydro-Brakes

Figure 72 - Pipe 1 Long Section

Figure 73 - Pipe 2 Long Section

Figure 74 - Pipe 3 Long Section
8.1.2.2 5 Year 15 Minute ARI Storm

Figure 75 - Peak Inflow

Figure 76 - Head Pipe 1

Figure 77 - Head Pipe 2
8.1.2.3 10 Year 15 Minute ARI Storm

**Figure 83 - Peak Inflow**

**Figure 84 - Head Pipe 1**

**Figure 85 - Head Pipe 2**
Figure 86 - Head Pipe 3

Figure 87 - Flow Rate Through Hydro-Brakes

Figure 88 - Pipe 1 Long Section

Figure 89 - Pipe 2 Long Section

Figure 90 - Pipe 3 Long Section
8.1.2.4 20 Year 15 Minute ARI Storm

Figure 91 - Peak Inflow

Figure 92 - Head Pipe 1

Figure 93 - Head Pipe 2
Figure 94 - Head Pipe 3

Figure 95 - Flow Rates Through Hydro-Brakes

Figure 96 - Pipe 1 Long Section

Figure 97 - Pipe 2 Long Section

Figure 98 - Pipe 3 Long Section
8.1.3 300m² Catchment

8.1.3.1 1 Year 15 Minute ARI Storm

Figure 99 - Peak Inflow

Figure 100 - Head Pipe 1

Figure 101 - Head Pipe 2
Figure 102 - Head Pipe 3

Figure 103 - Flow Rate Through Hydro-Brakes

Figure 104 - Pipe 1 Long Section

Figure 105 - Pipe 2 Long Section

Figure 106 - Pipe 3 Long Section
8.1.3.2 5 Year 15 Minute ARI Storm

Figure 107 - Peak Inflow

Figure 108 - Head Pipe 1

Figure 109 - Head Pipe 2
8.1.3.3 10 Year 15 Minute ARI Storm

Figure 115 - Peak Inflow

Figure 116 - Head Pipe 1

Figure 117 - Head Pipe 2
Figure 118 - Head Pipe 3

Figure 119 - Flow Rate Through Hydro-Brakes

Figure 120 - Pipe 1 Long Section

Figure 121 - Pipe 2 Long Section

Figure 122 - Pipe 3 Long Section
8.1.4 500m² Catchment

8.1.4.1 1 Year 15 Minute ARI Storm

Figure 123 - Peak Inflow

Figure 124 - Head Pipe 1

Figure 125 - Head Pipe 2
8.1.4.2 5 Year 15 Minute ARI Storm

Figure 131 - Peak Inflow

Figure 132 - Head Pipe 1

Figure 133 - Head Pipe 2
Figure 134 - Head Pipe 3

Figure 135 - Flow Rate Through Hydro-Brakes

Figure 136 - Pipe 1 Long Section

Figure 137 - Pipe 2 Long Section

Figure 138 - Pipe 3 Long Section
8.1.5 700m² Catchment

8.1.5.1 1 Year 15 Minute ARI Storm

Figure 139 - Peak Inflow

Figure 140 - Head Pipe 1

Figure 141 - Head Pipe 2
Figure 142 - Head Pipe 3

Figure 143 - Flow Rate Through Hydro-Brakes

Figure 144 - Pipe 1 Long Section

Figure 145 - Pipe 2 Long Section

Figure 146 - Pipe 3 Long Section
8.2 Multiple Catchment Inflows XP Raw Data

8.2.1 100m²-50 m²-50 m² Catchment

8.2.1.1 1 Year 15 Minute ARI

Figure 147 - Peak Inflows

Figure 148 - Effective Head Pipe 1

Figure 149 - Effective Head Pipe 2
Figure 150 - Effective Head Pipe 3

Figure 151 - Flow Rates through Hydro-Brakes

Figure 152 - Long Section Pipe 1

Figure 153 - Long Section Pipe 2

Figure 154 - Long Section Pipe 3
8.2.1.2 2 Year 15 Minute ARI

![Image of charts](image1.png)

Figure 155 - Peak Inflows

![Image of charts](image2.png)

Figure 156 - Effective Head Pipe 1

![Image of charts](image3.png)

Figure 157 - Effective Head Pipe 2
Figure 158 - Effective Head Pipe 3

Figure 159 - Flow Rate through Hydro-Brakes

Figure 160 - Long Section Pipe 1

Figure 161 - Long Section Pipe 2

Figure 162 - Long Section Pipe 3
8.2.1.3 5 Year 15 Minute ARI

- Figure 163 - Peak Inflows
- Figure 164 - Effective Head Pipe 1
- Figure 165 - Effective Head Pipe 2
Figure 166 - Effective Head Pipe 3

Figure 167 - Flow Rate through Hydro-Brakes

Figure 168 - Long Section Pipe 1

Figure 169 - Long Section Pipe 2

Figure 170 - Long Section Pipe 3
8.2.1.4 10 Year 15 Minute ARI

Figure 171 - Peak Inflows

Figure 172 - Effective Head Pipe 1

Figure 173 - Effective Head Pipe 2
8.2.1.5 20 Year 15 Minute ARI

Figure 179 - Peak Inflows

Figure 180 - Effective Head Pipe 1

Figure 181 - Effective Head Pipe 2
8.2.2 100m²-100 m²-50 m² Catchment

8.2.2.1 1 Year 15 Minute ARI

Figure 187 - Peak Inflows

Figure 188 - Effective Head Pipe 1

Figure 189 - Effective Head Pipe 2
Figure 190 - Effective Head Pipe 3

Figure 191 - Flow Rates through Hydro-Brakes

Figure 192 - Long Section Pipe 1

Figure 193 - Long Section Pipe 2

Figure 194 - Long Section Pipe 3
8.2.2.2 2 Year 15 Minute ARI

Figure 195 - Peak Inflows
Pipe 1

Figure 196 - Effective Head Pipe 1
Pipe 2

Figure 197 - Effective Head Pipe 2
Figure 198 - Effective Head Pipe 3

Figure 199 - Flow Rates through Hydro-Brakes

Figure 200 - Long Section Pipe 1

Figure 201 - Long Section Pipe 2

Figure 202 - Long Section Pipe 3
8.2.2.3 5 Year 15 Minute ARI

Figure 203 - Peak Inflows

Figure 204 - Effective Head Pipe 1

Figure 205 - Effective Head Pipe 2
Figure 206 - Effective Head Pipe 3

Figure 207 - Flow Rates through Hydro-Brakes

Figure 208 - Long Section Pipe 1

Figure 209 - Long Section Pipe 2

Figure 210 - Long Section Pipe 3
8.2.2.4 10 Year 15 Minute ARI

Figure 211 - Peak Inflow

Figure 212 - Effective Head Pipe 1

Figure 213 - Effective Head Pipe 2
Figure 214 - Effective Head Pipe 3

Figure 215 - Flow Rates through Hydro-Brakes

Figure 216 - Long Section Pipe 1

Figure 217 - Long Section Pipe 2

Figure 218 - Long Section Pipe 3
8.2.3 100m²-100 m²-100 m² Catchment

8.2.3.1 1 Year 15 Minute ARI

Figure 219 - Peak Inflows

Figure 220 - Effective Head Pipe 1

Figure 221 - Effective Head Pipe
Figure 222 - Effective Head Pipe 3

Figure 223 - Flow Rates through Hydro-Brakes

Figure 224 - Long Section Pipe 1

Figure 225 - Long Section Pipe 2

Figure 226 - Long Section Pipe 3
8.2.3.2 2 Year 15 Minute ARI

Figure 227 - Peak Inflows

Figure 228 - Effective Head Pipe 1

Figure 229 - Effective Head Pipe 2
Figure 230 - Effective Head Pipe 3

Figure 231 - Flow Rates through Hydro-Brakes

Figure 232 - Long Section Pipe 1

Figure 233 - Long Section Pipe 2

Figure 234 - Long Section Pipe 3
8.2.3.3 5 Year 15 Minute ARI

Figure 235 - Peak Inflows

Figure 236 - Effective Head Pipe 1

Figure 237 - Effective Head Pipe 2
Figure 238 - Effective Head Pipe 3

Figure 239 - Flow Rates through Hydro-Brakes

Figure 240 - Long Section Pipe 1

Figure 241 - Long Section Pipe 2

Figure 242 - Long Section Pipe 3
8.2.4 200m²-50 m²-50 m² Catchment

8.2.4.1 1 Year 15 Minute ARI

Figure 243 - Peak Inflows

Figure 244 - Effective Head Pipe 1

Figure 245 - Effective Head Pipe 2
8.2.4.2 2 Year 15 Minute ARI

Figure 251 - Peak Inflows

Figure 252 - Effective Head Pipe 1

Figure 253 - Effective Head Pipe 2
Figure 254 - Effective Head Pipe 3

Figure 255 - Flow Rates through Hydro-Brakes

Figure 256 - Long Section Pipe 1

Figure 257 - Long Section Pipe 2

Figure 258 - Long Section Pipe 3
8.2.4.3 5Year 15 Minute ARI

Figure 259 - Peak Inflows

Figure 260 - Effective Head Pipe 1

Figure 261 - Effective Head Pipe 2
Figure 262 - Effective Head Pipe 3

Figure 263 - Flow Rates through Hydro-Brakes

Figure 264 - Long Section Pipe 1

Figure 265 - Long Section Pipe 2

Figure 266 - Long Section Pipe 3
8.2.4.4 10Year 15 Minute ARI

Figure 267 - Peak Inflows

Figure 268 - Effective Head Pipe 1

Figure 269 - Effective Head Pipe 2
Figure 270 - Effective Head Pipe 3

Figure 271 - Flow Rates through Hydro-Brakes

Figure 272 - Long Section Pipe 1

Figure 273 - Long Section Pipe 2

Figure 274 - Long Section Pipe 3
8.2.5 200m$^2$-100 m$^2$-50 m$^2$ Catchment

8.2.5.1 1 Year 15 Minute ARI

Figure 275 - Peak Inflows

Figure 276 - Effective Head Pipe 1

Figure 277 - Effective Head Pipe 2
Figure 278 - Effective Head Pipe 3

Figure 279 - Flow Rates through Hydro-Brakes

Figure 280 - Long Section Pipe 1

Figure 281 - Long Section Pipe 2

Figure 282 - Long Section Pipe 3
8.2.5.2 2 Year 15 Minute ARI

Figure 283 - Peak Inflow

Figure 284 - Effective Head Pipe 1

Figure 285 - Effective Head Pipe 2
Figure 286 - Effective Head Pipe 3

Figure 287 - Flow Rates through Hydro-Brakes

Figure 288 - Long Section Pipe 1

Figure 289 - Long Section Pipe 2

Figure 290 - Long Section Pipe 3
8.2.5.3 5 Year 15 Minute ARI

Figure 291 - Peak Inflows

Figure 292 - Effective Head Pipe 1

Figure 293 - Effective Head Pipe 2
Figure 294 - Effective Head Pipe 3

Figure 295 - Hydro-Brakes through Hydro-Brakes

Figure 296 - Long Section Pipe 1

Figure 297 - Long Section Pipe 2

Figure 298 - Long Section Pipe 3
8.2.6 200m²-100 m²-100 m² Catchment

8.2.6.1 1 Year 15 Minute ARI

Figure 299 - Peak Inflows

Figure 300 - Effective Head Pipe 1

Figure 301 - Effective Head Pipe 2
Figure 302 - Effective Head Pipe 3

Figure 303 - Flow Rates through Hydro-Brakes

Figure 304 - Long Section Pipe 1

Figure 305 - Long Section Pipe 2

Figure 306 - Long Section Pipe 3
8.2.6.2 2 Year 15 Minute ARI

Figure 307 - Peak Inflows

Figure 308 - Effective Head Pipe 1

Figure 309 - Effective Head Pipe 2
8.2.6.3 5 Year 15 Minute ARI

Figure 315 - Peak Inflows

Figure 316 - Effective Head Pipe 1

Figure 317 - Effective Head Pipe 2
Figure 318 - Effective Head Pipe 3

Figure 319 - Flow Rates through Hydro-Brakes

Figure 320 - Long Section Pipe 1

Figure 321 - Long Section Pipe 2

Figure 322 - Long Section Pipe 3
8.2.7 300m²-50 m²-50 m² Catchment

8.2.7.1 1 Year 15 Minute ARI

Figure 323 - Peak Inflows

Figure 324 - Effective Head Pipe 1

Figure 325 - Effective Head Pipe 2
Figure 326 - Effective Head Pipe 3

Figure 327 - Flow Rates through Hydro-Brakes

Figure 328 - Long Section Pipe 1

Figure 329 - Long Section Pipe 2

Figure 330 - Long Section Pipe 3
10.2.7.2 2 Year 15 Minute ARI

Figure 331 - Peak Inflows

Figure 332 - Effective Head Pipe 1

Figure 333 - Effective Head Pipe 2
Figure 334 - Effective Head Pipe 3

Figure 335 - Flow Rates through Hydro-Brakes

Figure 336 - Long Section Pipe 1

Figure 337 - Long Section Pipe 2

Figure 338 - Long Section Pipe 3
8.2.7.3 5 Year 15 Minute ARI

Figure 339 - Peak Inflows

Figure 340 - Effective Head Pipe 1

Figure 341 - Effective Head Pipe 2
Figure 342 - Effective Head Pipe 3

Figure 343 - Flow Rates through Hydro-Brakes

Figure 344 - Long Section Pipe 1

Figure 345 - Long Section Pipe 2

Figure 346 - Long Section Pipe 3
8.2.8 300m²-100 m²-100 m² Catchment

8.2.8.1 1 Year 15 Minute ARI

Figure 347 - Peak Inflows

Figure 348 - Effective Head Pipe 1

Figure 349 - Effective Head Pipe 2
Figure 350 - Effective Head Pipe 3

Figure 351 - Flow Rates through Hydro-Brakes

Figure 352 - Long Section Pipe 1

Figure 353 - Long Section Pipe 2

Figure 354 - Long Section Pipe 3
8.2.8.2 2 Year 15 Minute ARI

Figure 355 - Peak Inflows

Figure 356 - Effective Head Pipe 1

Figure 357 - Effective Head Pipe 2
Figure 358 - Effective Head Pipe 3

Figure 359 - Flow Rates through Hydro-Brakes

Figure 360 - Long Section Pipe 1

Figure 361 - Long Section Pipe 2

Figure 362 - Long Section Pipe 3
8.2.8.3. 5 Year 15 Minute ARI

Figure 363 - Peak Inflows

Figure 364 - Effective Head Pipe 1

Figure 365 - Effective Head Pipe 2
Figure 366 - Effective Head Pipe 3

Figure 367 - Flow Rates through Hydro-Brakes

Figure 368 - Long Section Pipe 1

Figure 369 - Long Section Pipe 2

Figure 370 - Long Section Pipe 3
8.2.9 300m²-200 m²-100 m² Catchment

8.2.9.1 1 Year 15 Minute ARI

Figure 371 - Peak Inflows

Figure 372 - Effective Head Pipe 1

Figure 373 - Effective Head Pipe 2
Figure 374 - Effective Head Pipe 3

Figure 375 - Flow Rates through Hydro-Brakes

Figure 376 - Long Section Pipe 1

Figure 377 - Long Section Pipe 2

Figure 378 - Long Section Pipe 3
8.2.9.2 2 Year 15 Minute ARI

Figure 379 - Peak Inflows

Figure 380 - Effective Head Pipe 1

Figure 381 - Effective Head Pipe 2
Figure 382 - Effective Head Pipe 3

Figure 383 - Flow Rates through Hydro-Brakes

Figure 384 - Long Section Pipe 1

Figure 385 - Long Section Pipe 2

Figure 386 - Long Section Pipe 3
8.2.9.3 5 Year 15 Minute ARI

Figure 387 - Peak Inflows

Figure 388 - Effective Head Pipe 1

Figure 389 - Effective Head Pipe 2
Figure 390 - Effective Head Pipe 3

Figure 391 - Flow Rates through Hydro-Brakes

Figure 392 - Long Section Pipe 1

Figure 393 - Long Section Pipe 2

Figure 394 - Long Section Pipe 3
8.2.10 300m²-200 m² Catchment

8.2.10.1 1 Year 15 Minute ARI

Figure 395 - Peak Inflows

Figure 396 - Effective Head Pipe 1

Figure 397 - Effective Head Pipe 2
Figure 398 - Effective Head Pipe 3

Figure 399 - Flow Rates through Hydro-Brakes

Figure 400 - Long Section Pipe 1

Figure 401 - Long Section Pipe 2

Figure 402 - Long Section Pipe 3
8.2.10.2 2 Year 15 Minute ARI

Figure 403 - Peak Inflows

Figure 404 - Effective Head Pipe 1

Figure 405 - Effective Head Pipe 2
Figure 406 - Effective Head Pipe 3

Figure 407 - Flow Rates through Hydro-Brakes

Figure 408 - Long Section Pipe 1

Figure 409 - Long Section Pipe 2

Figure 410 - Long Section Pipe 3
8.2.10.3 5 Year 15 Minute ARI

Figure 411 - Peak Inflows

Figure 412 - Effective Head Pipe 1

Figure 413 - Effective Head Pipe 2
Figure 414 - Effective Head Pipe 3

Figure 415 - Flow Rates through Hydro-Brakes

Figure 416 - Long Section Pipe 1

Figure 417 - Long Section Pipe 2

Figure 418 - Long Section Pipe 3
8.2.11 300m$^2$-300 m$^2$-200 m$^2$ Catchment

8.2.11.1 1 Year 15 Minute ARI

Figure 419 - Peak Inflows

Figure 420 - Effective Head Pipe 1

Figure 421 - Effective Head Pipe 2
8.2.11.2 2 Year 15 Minute ARI

Figure 427 - Peak Inflows

Figure 428 - Effective Head Pipe 1

Figure 429 - Effective Head Pipe 2
8.2.12 300m²-300 m²-300 m² Catchment

8.2.12.1 1 Year 15 Minute ARI

Figure 435 - Peak Inflows

Figure 436 - Effective Head Pipe 1

Figure 437 - Effective Head Pipe 2
Figure 438 - Effective Head Pipe 3

Figure 439 - Flow Rates through Hydro-Brakes

Figure 440 - Long Section Pipe 1

Figure 441 - Long Section Pipe 2

Figure 442 - Long Section Pipe 3
8.2.12.2 2 Year 15 Minute ARI

Figure 443 - Peak Inflows

Figure 444 - Effective Head Pipe 1

Figure 445 - Effective Head Pipe 2
Figure 446 - Effective Head Pipe 3

Figure 447 - Flow Rates through Hydro-Brakes

Figure 448 - Long Section Pipe 1

Figure 449 - Long Section Pipe 2

Figure 450 - Long Section Pipe 3
8.2.13 400m²-200 m²-200 m² Catchment

8.2.13.1 1 Year 15 Minute ARI

Figure 451 - Peak Inflows

Figure 452 - Effective Head Pipe 1

Figure 453 - Effective Head Pipe 2
Figure 454 - Effective Head Pipe 3

Figure 455 - Flow Rates through Hydro-Brakes

Figure 456 - Long Section Pipe 1

Figure 457 - Long Section Pipe 2

Figure 458 - Long Section Pipe 3
8.2.13.2 2 Year 15 Minute ARI

Figure 459 - Peak Inflows

Figure 460 - Effective Head Pipe 1

Figure 461 - Effective Head Pipe 2
Figure 462 - Effective Head Pipe 3

Figure 463 - Flow Rates through Hydro-Brakes

Figure 464 - Long Section Pipe 1

Figure 465 - Long Section Pipe 2

Figure 466 - Long Section Pipe 3
8.2.14 400m²-300 m²-200 m² Catchment

8.2.14.1 1 Year 15 Minute ARI

Figure 467 - Peak Inflows

Figure 468 - Effective Head Pipe 1

Figure 469 - Effective Head Pipe 2
Figure 470 - Effective Head Pipe 3

Figure 471 - Flow Rate through Hydro-Brakes

Figure 472 - Long Section Pipe 1

Figure 473 - Long Section Pipe 2

Figure 474 - Long Section Pipe 3
8.2.14.2 2 Year 15 Minute ARI

Figure 475 - Peak Inflows

Figure 476 - Effective Head Pipe 1

Figure 477 - Effective Head Pipe 2
Figure 478 - Effective Head Pipe 3

Figure 479 - Flow Rates through Hydro-Brakes

Figure 480 - Long Section Pipe 1
8.2.15 400m²-300 m²-300 m² Catchment

8.2.15.1 1 Year 15 Minute ARI

Figure 481 - Peak Inflow

Figure 482 - Effective Head Pipe 1

Figure 483 - Effective Head Pipe 2
8.2.16 400 m²-400 m²-300 m² Catchment

8.2.16.1 1 Year 15 Minute ARI

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**Figure 489 - Peak Inflows**

![Peak Inflows](image1)

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**Figure 490 - Effective Head Pipe 1**

![Effective Head Pipe 1](image2)

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**Figure 491 - Effective Head Pipe 2**

![Effective Head Pipe 2](image3)