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Effects of local on-site stormwater detention systems on regional catchment hydrology

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Abstract

Stormwater detention is an essential component in conventional urban drainage systems. Its function is to attenuate the increase in peak discharge of stormwater runoff that inevitably results from the urbanisation of land. Its purpose is to manage adverse impacts to downstream hydrology, including flood, erosion and degradation of water quality.

Theoretically, detention should be a stormwater management technique that is adopted at carefully selected locations using comprehensive catchment wide analysis. Installing detention at inappropriate locations within the regional catchment is known to have potentially adverse impacts on the regional hydrology. The size and performance of detention can also be optimised when there is freedom to select its location in the regional catchment. In practice however, stormwater detention is commonly mandated for all new urban development projects, regardless of their location within the regional catchment. Where stormwater detention is mandated for all urban development sites, the allowable outlet flow rate should also ideally be established using catchment wide analysis. In practice however, the allowable outlet flow rate is typically specified as the maximum pre-development flow rate from the development site for design storms of equal return intervals. This research builds upon many previous studies that have highlighted this practice as a serious concern.

The usage of variable rainfall distribution probabilities for the creation of hydrographs in the engineering design of infrastructure like stormwater detention is not common, largely due to the complexities of model set-up, the analyses of results required to arrive upon a deterministic design outcome, and a lack of prescriptive guidelines for practitioners. As part of this research this limitation in current practice has been shown to be the cause of a significant extent of failure to meet objectives in peak flow management of development site discharge. This research has also shown conclusively that the selection of the number of rainfall patterns used for the design of a detention system is proportional to its success in achieving peak flow reduction objectives.

This thesis provides research into the development of a practical solution to these issues, whereby stormwater detention can continue to be mandated for all new development projects, with regional hydrologic impacts able to be considered without full-scale regional catchment assessment. At the core of this is the development of a numerical model for hydrologic assessment of regional catchments with urbanisation and detention installed at

varying hypothetical locations throughout. Under the dominating influence of rainfall pattern variability, recurring trends are revealed that describe the impact of urbanisation and detention on the regional catchment peak discharge flow rate. The trends are shown to be respective of two key factors, being the ratio of development site area to the regional catchment area and the location of the development within the regional catchment. With these two input parameters, it is shown to be possible to estimate the mean impact of urbanisation and detention on the regional catchments peak outflow at a specific downstream location.

A new system of equations is presented within this thesis that is considered to be a significant tool for better ensuring that detention is designed with due consideration given to its location and impacts on the hydrology of its regional catchment. This outcome is expected to provide a means for decision making regarding the installation or avoidance of stormwater detention in favour of infrastructure upgrades.

The development of the new equations has used a regression analysis of numerical modelling results taken from a number of case study catchments located along the eastern coastline of Australia. This geographical area has been selected due to the intensity of urban development, availability of recorded rainfall data, and the prevalence of local government policies that mandate the usage of stormwater detention for urban development projects in these regions. Whilst the significance of the spatial variance of rainfall patterns is recognised, the usage of a Monte Carlo technique to account for variable rainfall probability has been adopted as a means to universalise the results.

In a world of future climate change and uncertainty, the variability and unpredictability of the temporal patterns of rainfall are a major concern. This research serves to better understand these issues and provides a direction forward for the design and assessment of stormwater detention with due consideration given to its potential impact on the hydrology of its regional catchment.

Foreword

This research project started from a simple question: what about the downstream impact of all these detention systems we are building everywhere?

As a practicing civil engineer of several years it has been my role to design and oversee the construction of countless detention systems around Australia. In most cases and in compliance with mandatory regulations (and what the computer programs say), stormwater runoff from each detention system was designed to release outflow at a rate that was equal or less than the pre-development conditions.

What happens when the elongated hydrograph peak of the stormwater detention system converges with the peak of another hydrograph at a downstream confluence location is not my problem, I have been told. The fact that rain may not actually fall in the exact timing of the temporal pattern that was used to generate the design hydrographs is also not my problem, I have been told.

At the start of this research project I was simply wondering if we, as practitioners, were doing the right thing and what the actual impact of some of these potential 'faux pas' may be on regional hydrology and the environments in which we live. During the research project I learnt that there is, in fact, a real problem in the way that things were being done. At the conclusion of this research project I am hoping to have contributed to the solution and a better way forward for protecting our regional waterways from unpredicted flood changes and water quality impacts or wasted construction resources.

This project would not have been possible without the support of Professor Hong Zhang of Griffith University. Hong's patience in translating my thoughts into academic process has been invaluable. Hong is truly an expert in the field of water research and I have been extremely lucky to have her supervision, help and mentoring throughout this project. Many other affiliates of Griffith University have also assisted in this project, including my co-supervisor Professor Dong-Sheng Jeng and the now graduate civil engineer, Kartik Mehta.

Acknowledgement is also given to the team of professional engineers at Michael Bale and Associates Pty Ltd, particularly Alex Rowlands. Many of the case studies used in this research are real engineering projects that have been expertly delivered by the team at Michael Bale and Associates Pty Ltd, and it has been my pleasure to work with them.

My wife Sandy and daughter Mackenzie also deserve prizes for putting up with me.

Statement of Originality

This work has not previously been submitted for a degree or diploma in any university. To the best of my knowledge and belief, the thesis contains no material previously published or written by another person except where due reference is made in the thesis itself.

Rodney Ronalds

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Acknowledgement of Papers included in this Thesis

Included in this thesis are journal papers in Chapters 4, 5, and 6 which are co-authored with other researchers. My contribution to each co-authored paper is outlined at the front of the relevant chapter. The bibliographic details for these papers including all authors, are:

Chapter 4:

Ronalds, R. and Zhang, H., 2017. *An alternative method for on-site stormwater detention design*. Journal of Hydrology (New Zealand), 56(2), pp.137-153

Chapter 5:

Ronalds, R., Rowlands, A and Zhang, H., 2019. *On-site stormwater detention for Australian development projects: Does it meet frequent flow management objectives?*. Water Science and Engineering, 12(1), pp.1-10

Chapter 6:

Ronalds, R. and Zhang, H., 2019. *Assessing the Impact of Urban Development and On-Site Stormwater Detention on Regional Hydrology Using Monte Carlo Simulated Rainfall.* Water Resources Management, pp.1-20.

(Signed) _____ (Date) 28/2/19

Rodney Ronalds

(Countersigned) _____ (Date) 06/03/19

Supervisor: Professor Hong Zhang

Additional Publications not within this Thesis

In addition to the published journal papers that constitute Chapters 4, 5, and 6 of this thesis, the following peer reviewed publications have been produced during the period of candidature:

Ronalds, R. and Zhang, H., 2016. *Fundamentals for on-site stormwater detention design: Optimising design outcomes and reducing risk of regional effect.* In 37th Hydrology and Water Resources Symposium 2016: Water, Infrastructure and the Environment (p. 479). Engineers Australia.

Ronalds, R., Rowlands, A. and Zhang, H., 2017. *The performance of on-site stormwater detention systems in response to recent advances in hydrologic theory*. In 13th Hydraulics in Water Engineering Conference (p. 354). Engineers Australia.

1. Introduction

1.1 An Overview of Stormwater Detention and the Problem Definition

Urban development projects involving buildings, roads and infrastructure inevitably result in the removal of natural vegetation, increased impervious areas, reduced infiltration of rainfall to groundwater, and increased drainage efficiency of flow paths. These factors all contribute to adverse hydrologic conditions in the regional catchment that include increased flow volumes, increased flood frequency, increased magnitude of flooding, increased stream erosion, increased runoff frequency, faster flood peaks, and decreased baseflow.

Stormwater detention is a common solution to these issues, which acts to collect runoff from an urbanised area and provide temporary storage of water with a controlled release to the downstream outlet. A commonly mandated design objective for detention is to achieve a maximum allowable release rate that is equal to that of the pre-developed site, which is often referred to as the "micro-management" method. The intention of this objective is to limit peak discharge in downstream waterways and the resulting adverse hydrologic conditions that are associated with higher peak flow rates and velocities in regional waterways.

Since the early 1970's and extending to today, various studies have shown that the management of site-based stormwater runoff to pre-developed conditions at an individual development site scale is not always effective in managing regional catchment peak runoff. The elongation of hydrographs that result from detention can cause coincident peaks with other sub catchments in the regional catchment, exacerbating peak flow, velocity, and all the associated adverse impacts that result.

There are some studies that have proposed exclusion zones that prohibit detention in the lower potions of the regional catchment (Flores *et al.* 1982, Goff and Gentry 2006, Leise 1991, McCuen 1979, Saunders 2008). Another widely researched solution to the problem is the strategic distribution of detention throughout the catchment, with algorithms for ideally locating and sizing detention for the benefit of the total catchment hydrology (Bellu *et al.* 2016, Duan *et al.* 2016, Kaini, *et al.* 2007, Ravazzani *et al.* 2014, Shuster and Rhea 2013, Su *et al.* 2010, Tao *et al.* 2012, Wang *et al.* 2017). Regional catchment studies can also provide allowable land parcel storage or release rates that consider the capacity of

flow paths and allowable flooding of the regional catchment (Beecham *et al.* 2005, Lees and Lynch 1992, Phillips 1989, Silveri and Rigby 2006).

These approaches are however limited in application and practicality. Property ownership issues are one of the major obstacles that prevent the distribution of detention to ideal locations within a regional catchment. Another obstacle is the engineering resources required to undertake regional assessments that assign either detention storage requirements or allowable site discharge rates that consider the regional catchments hydrology. Mandates therefore prevail around the world that require detention to be installed at all new urban development projects, using the micro-management design criteria (Akan *et al.* 1994, Olenik 1999, Pezzaniti 2003, Schueler and Claytor 2000).

This research provides a focussed investigation into the potential impact that the currently dominant trends in practice and policy for detention design can have on regional hydrology.

All of the analyses included in this thesis consider case studies that involve catchments along the eastern coastline of Australia. This geographic region is rapidly growing with intensifying urbanisation. The use of stormwater detention for the mitigation of increases to peak runoff from urbanised land parcels is also routinely mandated by local governments in this region.

1.2 Assessment Criteria for Detention Design Objectives

The process of determining the size and configuration of a stormwater detention system has followed a linear and deterministic approach for many years, involving the modelling of hydrographs to assess the changes occurring to peak runoff in pre- and post-development conditions. This traditional process, and the resulting impacts the occur to regional hydrology, are challenged by this research.

The traditional detention design process starts with the creation of a hydrograph as an input condition, which is a key topic of this research. The incoming hydrograph is required to be modified to account for the delayed outlet and reduced peak discharge that result from the storage within the detention system. There are several methods for creating a hydrograph, ranging from simple analytical approximations to detailed routing equations. Similarly, the methods of simulating the performance of detention and modifying the incoming hydrograph vary from the simplistic to the complex. In all cases however, the incoming

and outgoing hydrographs must be determined to the satisfaction of the designer in order to make an assessment of the achievement of the detention design objectives.

The second major hurdle in the design of a detention system is the establishment of an allowable outlet flow rate. The extent to which a detention system is capable of reducing peak flow is unlimited, providing adequate space and construction materials are available. As with any element of engineering infrastructure, a target objective is required, and the optimised solution will achieve this objective with the best use of space and material resources. Where micro-management policies are in place the traditional design methodology is demonstrated using the flow chart in Figure 1.1. In this process the first step is to calculate the existing case hydrology to determine an allowable existing, or "pre-development" scenario for each design storm return interval. The hydrograph is generated as a plot of the land parcel stormwater runoff (Q) over time (t). The peak of the existing case hydrograph ($Q_{existing\ (max)}$) is then extracted and used as the criteria to set the allowable maximum height of the hydrograph discharging from the development case scenario with detention.

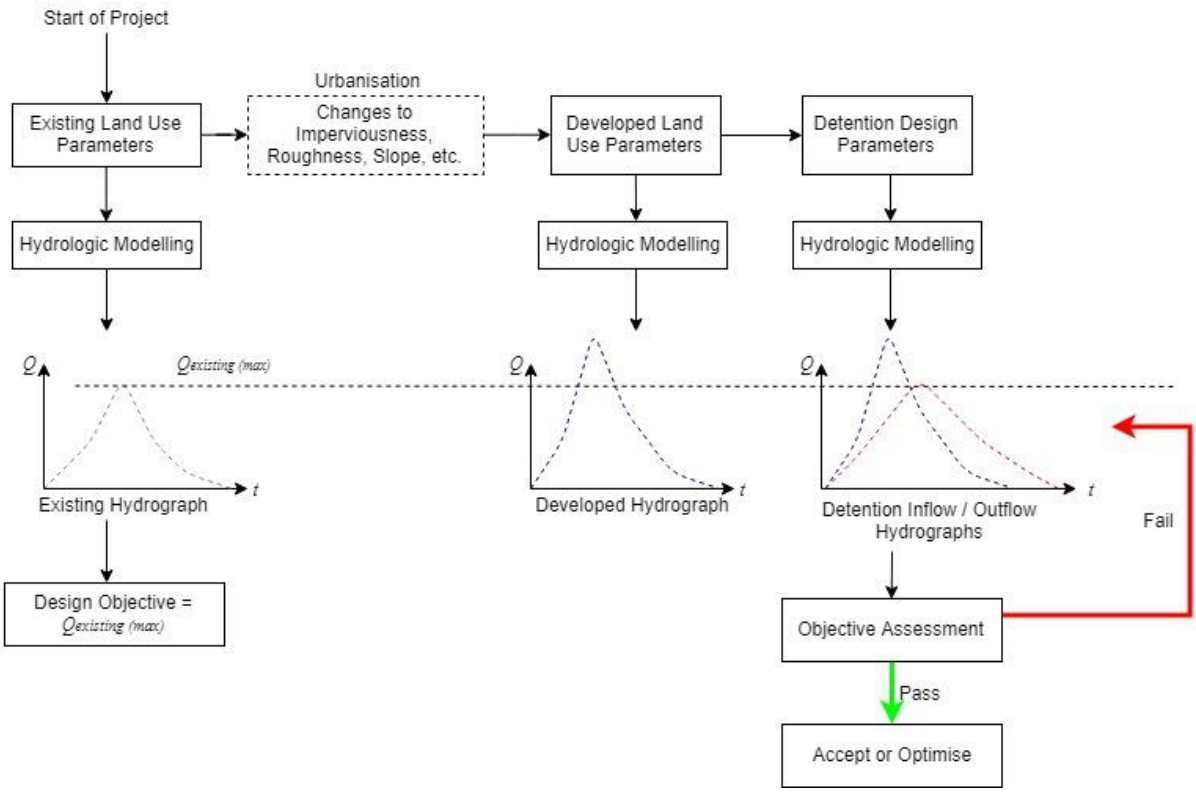

Figure 1-1 – Traditional Flow Chart for Assessment of Detention Design Objectives

This research challenges this traditional design approach and aims to provide a modification to this process that can account for the potential impact that an urban development project can have on the regional catchment.

1.3 Variable Rainfall Probability and Prediction Uncertainty

Many of the most significant recent advances in hydrology have focused on the natural variability of rainfall patterns and the inability for dependence upon a singular temporal pattern for the prediction of a storm event to provide a reliable result (Babister *et al.* 2016c, Charalambous *et al.* 2013, Hill and Mein 1996, Kuczera *et al.* 2006, Nathan *et al.* 2016, Nathan and Weinmann 2013, Rahman *et al.* 2002b).

Ensemble analysis is a developing practice that involves the usage of a number of temporal patterns (typically 10) that are taken from recorded events to model a particular storm of a specific recurrence interval and duration. Monte Carlo simulation allows for a much wider range of temporal patterns to be generated by using random variables to generate an unlimited array of potential patterns.

These developing practices reveal a significant issue of prediction uncertainty. To state deterministically that any hydraulic structure is designed to account for the probability of a design event is no longer possible. When several outcomes are derived for the same event, the achievement of design objectives must relate to probabilistic confidence limits.

Whatever the process for making a selection of design event (i.e. taking the mean, maximum, minimum, or closest to median), there will be several discarded results that still accurately represent the probability of the design event. It is these discarded events that cause concern of prediction uncertainty.

This research aims to assess the impacts that the emerging awareness of variable rainfall probability may have on the design of detention systems, as well the performance of detention systems that have been constructed using outdated single design event approaches. Variable rainfall probability is also incorporated into the process of considering the regional impact of micro-management detention objectives.

The consideration of variable rainfall probability also acts to universalise the results and findings of the analyses that are presented in this research. Considering randomly distributed rainfall patterns aims to remove the ability for recurring past trends of rainfall in specific areas to skew the findings or limit applicability to a single geographic area.

2. Literature Review

2.1 The Hydrologic Impact of Urban Development

The adverse hydrologic conditions that result from urban development have prompted numerous and evolving studies, focussing on increased flow volumes (Harris and Rantz 1964), increased flood frequency and magnitude (Leopold 1968), increased stream erosion (Hammer 1972), increased runoff frequency (ASCE 1975), faster flood peaks (Mein and Goyen 1988), decreased baseflow (Paul and Meyer 2001), as well as adverse impacts on flood levels, extents, distribution of flood waters, timing and duration of flooding (McLuckie *et al.* 2016).

The removal of natural vegetation and increased amount of impervious area that inevitably results from urbanisation is largely considered to be the primary contributor to hydrologic problems (McCuen and Moglen 1988, Arnold and Gibbons 1996, Snyder *et al.* 2005). Other factors include straighter than natural channels and overland flow paths with reduced flow distance before water reaches a stream (Ladson *et al.* 2016), reduced interception and infiltration of precipitation (Berland *et al.* 2017) and the resulting changes to groundwater behaviour that are consequence from reduced infiltration (Konrad and Booth 2005).

Around the world, many countries are experiencing hydrologic issues as a result of urbanisation, typified by expansive areas of medium- to low-density housing that sprawled in the mid twentieth century (Brown *et al.* 2009). As urban development continues to spread, so to do the hydrologic concerns. Klein (1979) provides details of an early study indicating that stream quality impairment is first evidenced when watershed imperviousness reaches 12%, but does not become severe until imperviousness reaches 30%. Burns *et al.* (2012) found that in catchments with as little as 5–10% total imperviousness, conventional stormwater drainage without treatment measures can contribute to increases in frequency and magnitude of storm flows with resulting flood issues and water quality degradation. Booth *et al.* (2002) found that at a 10% level of effective impervious surface, runoff production increased to the extent that the post-development 2-year storm was found to yield the same amount of discharge as a 10-year pre-development storm.

The ‘traditional’ view of the role of stormwater drainage has been to manage the ‘nuisance’ caused by ponding stormwater and to curb community complaint by making the drainage systems capable of efficiently routing stormwater runoff to receiving waters (Walsh *et al.*

2005a). In traditional drainage systems the underground pipes are used to quickly remove runoff, effectively taking away both groundwater infiltration and overland flow (Loukas and Quick 1996, Elsenbeer and Vertessy 2000). This results in an increased volume of water that is being transported at a greater velocity, resulting in an abbreviated hydrograph that peaks sooner than natural conditions (Hood *et al.* 2007).

2.1.1 Water Quality Issues

Stormwater runoff from urbanised areas with impervious surfaces has contributed to the collapse of healthy freshwater ecosystems in urban environments around the world (Ladson *et al.* 2006, Roy *et al.* 2008). Referred to as the "urban syndrome" (Walsh *et al.* 2005b, Meyer *et al.* 2005), the degradation of water quality as a result of urban development is a known concern for authorities and engineering practitioners.

Water quality issues can be broadly lumped into two categories with separate treatment mechanisms:

2.1.2 Nutrients and Suspended Sediments

Sartor and Boyd (1972) describe the fact that runoff from urban areas is not clean rainwater and has a significantly adverse impact on receiving waterways. Among the sources of pollution in urban runoff water are debris and contaminants from streets, contaminants from open land areas, publicly used chemicals, air-deposited substances, ice control chemicals, and dirt or contaminants washed from vehicles. Specifically, nutrients such nitrogen and phosphorus have been identified as key concerns. These nutrients remain concerns today, due to their role in eutrophication and harmful algal blooms (Brooks *et al.* 2016, Lusk and Toor 2016). Suspended sediments have also evolved as a major issue, not only due to the physical impact on water clarity but also due to the known ability to capture particulate matter such as nitrogen and phosphorus and mobilize these pollutants with the sediment (Vase and Chiew 2004, Liu and Davis 2014, Landsman and Davis 2018).

Typically, the management of nutrients and suspended sediments is carried out by organic treatment systems such as bio-retention (Davis *et al.* 2009), which includes physical and biological processes that mimic ecological processes similar to those that occur in nature (Liu *et al.* 2014). Alternatively, infiltration systems are common to remove nutrients and suspended solids along with the bulk of the stormwater runoff, by restoring pre-development water fluxes to groundwater and limiting the impacts of urbanisation on the water cycle (Petrucci *et al.* 2017).

2.1.2.1 Erosion and Sedimentation

Even when nutrients and sediments are managed at source and “clean” water is delivered to a natural stream, erosion and sedimentation can result from changes to the flow regime including increased flow, water level, velocity or increased time periods of higher runoff rates. Urbanised streams experience increased sediment supply, incision, enlargement, and homogenization of channel morphology (Vietz *et al.*, 2014).

Vegetated treatment systems designed for load reduction can still degrade receiving streams by lengthening the duration of flows large enough to cause habitat disturbance and channel erosion downstream, through their storage and routing. Additionally, unless they are designed with the ability to retain flows and reduce volume, the resulting outflow rates can exceed channel erosion thresholds (typically flows exceeded once in 0.5–1.5 years) for long periods, with resulting geomorphic and ecological damage (Burns *et al.* 2012).

2.1.3 Flood Issues

Urban development increases impervious areas and reduces the amount of rainfall that would naturally be lost to infiltration and baseflow. In simple terms, more stormwater runs off urbanised areas compared to natural areas and this results in flood issues. On average, urban catchments covert 90% of the storm rainfall to runoff, whereas the non-urban forested catchments retain 25% of the rainfall and only convert 75% to runoff (Shang and Wilson, 2009). Urban development systems also utilise drainage systems that are smoother, faster, straighter and more efficient than natural systems, resulting in the earlier arrival of the hydrograph centroid with reduced lag time (Ogden *et al.* 2011). These factors contribute to quantifiable increases to flood volumes, peak discharge and frequency of floods (Dougherty *et al.* 2007).

The effects of urbanisation on peak flow and flooding have been discussed since the 1960s (Brater and Sangal 1968). Figure 2-1 provides a graphical description of the increased flow volume and decreased lag time that is the well-established effect of urbanisation (Leopold 1968).

As urbanisation continues to become larger and denser around the world, floods are becoming more frequent and more devastating than ever before (Salvadore *et al.* 2015, Mark *et al.* 2004, Schmitt *et al.* 2004).

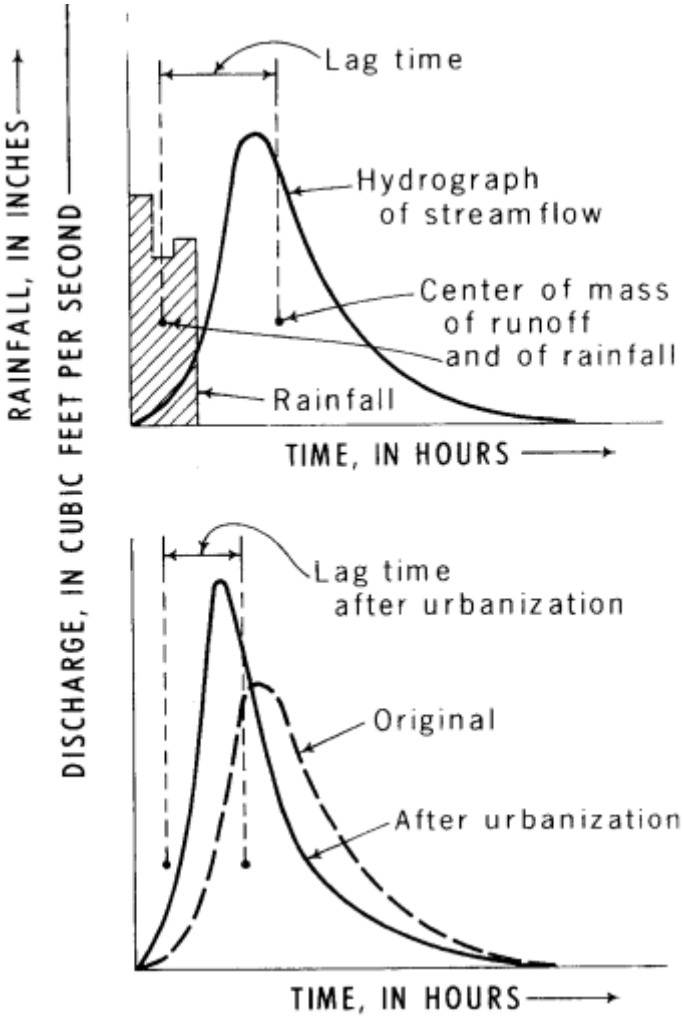

Figure 2-1 – Hypothetical hydrograph showing reduced lag and increased peak flow as a result of urbanisation (Leopold 1968)

Hollis (1974) found that floods with a return period of 100 years can be expected to double in size by the urbanisation of a catchment resulting in 30% imperviousness. Konrad (2003) provided a study of two catchments over the span of a 40-year period and found that the catchment that experienced urban development also experienced dramatic increases to peak discharge when compared to the catchment that remained natural, see Table 2-1:

Table 2-1 – Increase in Peak Discharge as a Result of Urban Development (Konrad 2003).

Flood Frequency	**Increase in flood peak discharge because of urban development**
2-year	100 to 600 percent
10-year	20 to 300 percent
100-year	10 to 250 percent

For both water quality issues and flood mitigation reasons, the concept of stormwater volume management has succeeded as an important consideration to manage the effect of urbanisation on catchments (Phillips *et al.* 2016).

2.2 The Function of Stormwater Detention

For the context of this research, stormwater detention refers to any storage of runoff that aims to attenuate peak discharge of stormwater via a controlled structural release system.

Stormwater detention works by holding back incoming stormwater runoff and forcing lag upon the outflow hydrograph that results in an elongated discharge time with a lower discharge peak. In Figure 2-2, the detention system has reduced the hypothetical site's runoff hydrograph (Q_{IN}) to a peak discharge (Q_{OUT}) that is equal to a pre-determined maximum allowable release rate. The volume between Q_{IN} and Q_{OUT} is temporarily stored (or detained) in the detention system during the storm event. In most applications, the maximum allowable release rate is equal to the pre-development peak flow rate for the design return period(s) (Walesh 1989).

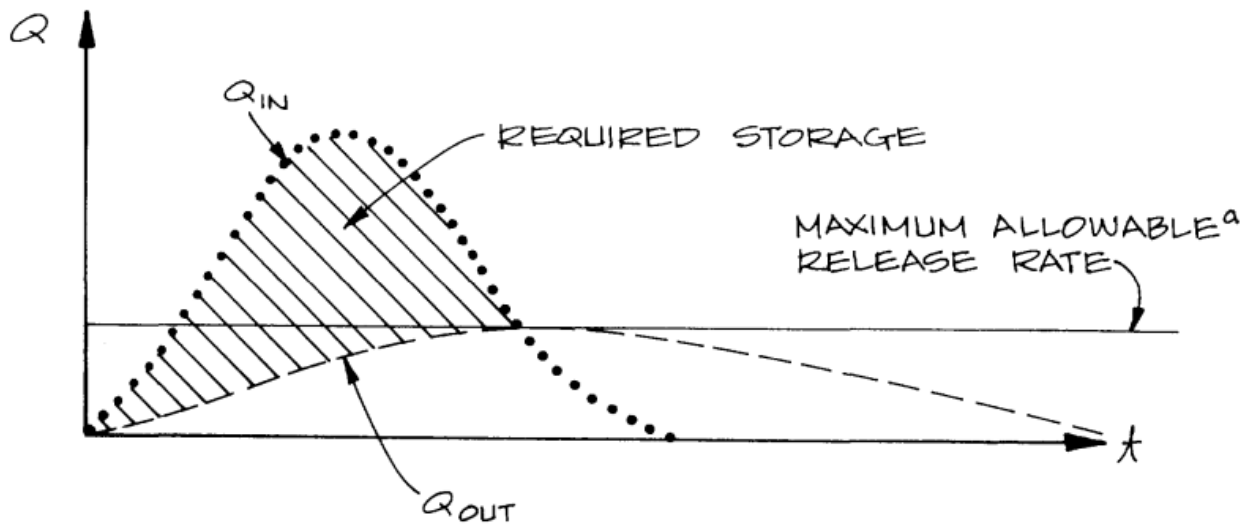

Figure 2-2 – The Effect of Detention on a Hydrograph (Walesh 1989)

In practice, traditional detention systems are typically large regional basins that provide attenuation to a development such as a residential subdivision including roads and allotments, created by excavation into the ground surface and partially formed by an embankment on the downslope side. Alternatively, On-Site Detention (OSD) is a system provided at an allotment scale, designed for small developments such as a high-rise tower and constructed from blockwork or concrete (Ladson and Nathan 2016).

2.3 Other Urban Water Management Approaches

It is important to note that stormwater detention is not the only approach to mitigating the adverse impacts of urbanisation. There are several other approaches that are either alternative or complementary to stormwater detention in an urban drainage system.

2.3.1 Infiltration

Infiltration is a key component of the Low Impact Development (LID) concept that has been a developing trend in many countries such as the United States, Canada and China. The primary aim of LID is toward detaining, storing and infiltrating urban runoff to return to groundwater (Wong *et al.* 2002).

For LID to be successful, it is generally appreciated that highly permeable soils are required and the practice is limited to frequent storm events (Holman-Dodds *et al.* 2003). Methods of designing infiltration systems range from simplistic monographs using single design storms events (Jia *et al* 2016) to complex models that use spatial editors, GIS, and other graphical interface features (Elliot and Trowsdale 2007).

The effectiveness of infiltration has been shown to be limited when simplistic design techniques are utilised, with complex groundwater modelling and the use of continuous simulation of rainfall required to provide accurate deign outcomes (Zimmer *et al.* 2007). Infiltration is also limited in application to management of frequent events. Even in areas where infiltration systems are applied, there often still needs to be measures in place for flood management during the larger, more intense, and rarer storm events (Woznicki *et al.* 2018).

2.3.2 Bio-Retention

Often referred to as ‘rain gardens’, bio-retention systems are vegetated infiltration systems that are usually disguised in landscaped areas of new developments. The typical composition includes 0.7– 1 m of a sand/soil/organic media for treating infiltrating storm-water runoff, a surface mulch layer, various forms of vegetation, orientation to allow 15–30 cm of runoff pooling and associated appurtenances for inlet, outlet, and overflow (Li *et al.* 2009).

Bio-retention systems are used successfully as components of LID for their infiltration ability (Emerson and Traver 2008), particularly when used in highly permeable soils and when there is no under-drainage present.

The primary purpose of bio-retention systems is water quality treatment, specifically in the removal of total nitrogen, total phosphorus, suspended solids and heavy metals (Coffman *et al.* 1994). The efficacy of infiltrating runoff is limited to small and midsized storm events and bio-retention is not considered suitable for peak flow mitigation of significant events. Hunt *et al.* (2006) limit the ability for bio-retention to attenuate peak flow to storm events of 40mm rainfall.

2.3.3 Rainwater Harvesting

Whilst rainwater harvesting has arguably been in practice for thousands of years as man has tried to survive in desert regions and has skilfully learnt to manage the vital but scarce resource of water, it has only been in recent decades that the link to regional catchment runoff quality and quantity has been discussed (Boers and Ben-Asher 1982).

Rainwater harvesting tanks contribute to regional catchment management by storing roofwater runoff and removing it from the contributing catchments total flow discharge, taking with it the volume of water and concentration of nutrients. In practice, rainwater harvesting has been proven to be effective in protecting regional catchment hydrology from adverse impacts to water quality and quantity (Coombes *et al.* 2000).

Recent advanced developments have shown that it is possible for large numbers of rainwater tanks in a regional catchment to be connected as a network and responsive to flood threats. Moriyama (2016) describes the development and efficacy of an intelligent network of 'Smart Rainwater Tanks' that empty upon warning and prior to oncoming rainfall to provide a storage capacity capable of mitigating regional flooding.

In the traditional application however, rainwater tanks alone are not generally considered capable of restoring post-development flood peaks to their pre-development level and are not able to mitigate the impact of urban development. Burns *et al.* (2010) estimate the limited ability for rainwater harvesting tanks to achieve 10 to 20% reduction in peak discharge, which is typically less attenuation than required.

2.3.4 Constructed Wetland or Ponds

Wetlands are popular water quality and quantity treatment features, largely due to their aesthetic value and ability to create habitat for wildlife. Wetlands can provide important benefits to water quality by retaining or transforming pollutants such as nutrients, sediments, pathogens, pesticides, and trace metals (Blahnik and Day 2000, Fisher and

Acreman 2004). Wetlands work in reducing flood impacts by interrupting the organization and energy of floodwaters, spreading out excess water in floodplains and detaining it in shallow impoundments, with excess water reclaiming abandoned streambeds adjacent to the main channel (DeLaney 1995).

In large flow events, wetlands can be subject to damage from erosion and have been shown to be less effective in the reduction of pollutants as the incoming flow rate increases (Knox *et al.* 2008). A form of detention elsewhere is the regional catchment is therefore commonly used in addition to wetlands.

2.3.5 Managing Effective Imperviousness

Increased imperviousness of a catchment is well known to result in enhanced hydraulic efficiency that can cause substantially decreased capacity for rainwater infiltration leading to increased excess runoff and urban flooding (Mejía and Moglen 2010).

It is not realistic to expect urban development practices to be modified to completely exclude imperviousness surfaces. There are however methods of managing the effective imperviousness of catchments to improve regional water quantity and quality aspects, most of which focus on the difference between Total Impervious Area (TIA) and Directly Connected Impervious Area (DCIA).

Effective impervious area is defined as all impervious surface area that is hydraulically connected (i.e. piped) to a drainage system so as to enhance conveyance of water away from a source area, such as a city street or residential neighbourhood. Some examples of effective impervious areas would include streets with curbs or gutters that are drained to an outfall (Shuster *et al.* 2005). Conversely, an ineffective or disconnected impervious surface routes runoff to pervious surfaces in pursuit of infiltration prior to being received by piped systems or discharging to watercourses (Booth and Jackson 1997, Horner e*t al.* 1999, Hatt *et al* 2004, Taylor 2005a, Walsh *et al.* 2004).

In a Korean study, Hwang *et al.* (2017) found that DCIA reduced peak runoff from a regional catchment by up to 12%. Lee and Heaney (2003) provide a study of American catchments with varying amounts of DCIA and found that DCIA is a key factor of urbanisation's effect on storm water quantity and quality, with DCIA contributing 72% of the total runoff volume during the 52 years study, despite only covering 44% of the catchment area.

However, removing the required degree of DCIA that can fully mitigate the adverse impacts of urbanisation on regional hydrology is not considered practicable. Furthermore, not all studies show that the removal of DCIA is the perfect solution to urbanisation. In a Chinese study by Yao *et al.* (2016) that considered heavy rainfall events, it was TIA rather than DICA that was found to be the dominant factor affecting total runoff. In the comparison of calculated runoff to recorded events in an Indonesian study, Farika (2018) found no significant difference between and DCIA and TIA.

2.3.6 Land Use Planning

The argument exists that structural controls, no matter how well they are designed, cannot mitigate any but the most egregious consequences of urbanisation and minimum percentages of forest coverage area should be assigned to catchments for their protection (Booth *et al.* 2002).

The ability for structural controls to be effective in flood attenuation is also limited in application to rivers with low residence times. The longer the residence time, the larger the buffering capacity to attenuate peak flood events. Nearly 2 billion people around the world live in areas of high flood risk with low attenuation potential. Most of these people live in northern South America, highly populated regions of northern India and South East Asia, Central Europe, and the Southwest coast of Africa (Lead *et al.* 2005).

Flooding is also a naturally occurring phenomena that benefits ecosystem health (Mirza *et al.* 2005). Effective land use planning can ideally assign suitable areas of a city to allow for flooding, avoiding the risks associated with flood damage and simultaneously assisting in hydrological and ecological connectivity (Schuch *et al.* 2017).

Yang and Li (2011) provide a novel approach to the management of urban development by suggesting the actual location of new urban areas are planned for with consideration to the regional hydrology, making the important note that the largest impacts of urbanisation are not necessarily seen immediately downstream of the development and a regional approach is required.

This argument is however limited in application and not always going to be possible with existing urbanised areas. Whilst re-settlement of people who live in flood affected areas is potentially a theoretical possibility, communities tend to provide resistance to relocation strategies and they must be very carefully managed and include adequate socioeconomic support (Correa 2011).

2.4 Objectives for Stormwater Detention

Attenuating peak flows is a fundamental principle of flood control and water quality improvement. Effectively designed detention systems can achieve both objectives and are known as an economic and efficient structural practice (Bellu *et al.* 2016).

The design of a detention system involves establishing a combination of volume and outlet arrangement that is aimed at achieving a specific attenuation of flow to comply with the design objectives. There are however two significantly different ways of establishing the objectives, focussing either on the specific outlet of the detention only, or with a much wider view of the regional catchment.

2.4.1 Micro-Management Objectives

A common criterion used in detention basin design is to reduce the post-development peak discharge of the contributing catchment to the pre-development magnitude corresponding to a design rainfall of a specified return period (Akan and Antoun 1994). Often termed the 'micro-management' method (Olennik 1999), the designer relies on the assumption that any adverse impacts to the downstream hydrology are mitigated via the assurance that the peak discharge at the outlet from the detention is not increased beyond the pre-development peak.

The micro-management practice is common around the world. A number of councils in Australia have adopted a blanket policy whereby all new developments or re-developments must incorporate on-site stormwater detention facilities that achieve pre-development discharge peak (Pezzaniti *et al.* 2003). In the United States, The Stormwater Management Act calls for local ordinances on stormwater management to be implemented, focused primarily on "peak management" at the development site scale (Schueler and Claytor 2000). In China, there is a growing push for public policy to enforce detention on all new developments, with micro-management techniques referred to as the "zero-increase policy" (Fang *et al.* 2017).

Figure 2-3 provides a summary of the micro-management design objectives, in which the example shows the detention outflow hydrograph "*C*" achieving a reduced peak discharge from the post-development case hydrograph "*B*" to a rate matching the pre-development hydrograph "*A*".

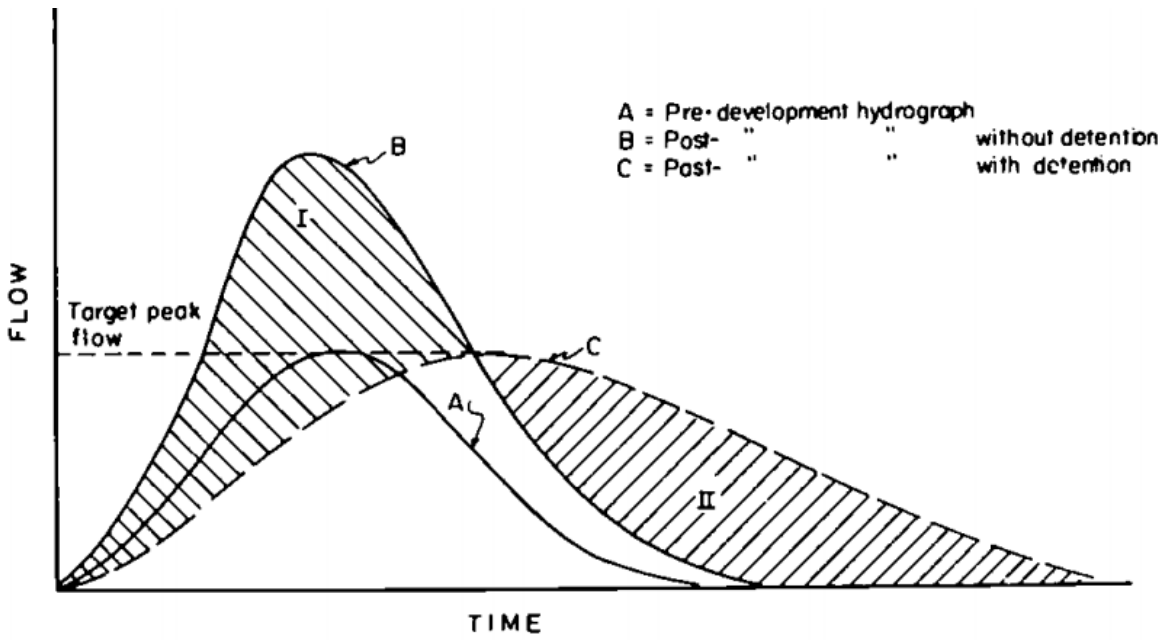

Figure 2-3 – Typical Hydrographs for Pre- and Post-Development Conditions (Nix and Tsay 1988)

Micro-management objectives are a popular option for authorities, largely due to the simplicity of design and assessment required. It is also "fair" insofar as the required volume of detention by ratio of area is typically equal for a development site, regardless of where it is located in the regional catchment.

2.4.2 Total Catchment Objectives

Whilst the micro-management technique is popular and widely practiced, hydrological modelling has shown that application of detention in some parts of the catchment may not have the desired impact on the total catchments hydrograph. In particular, it has been shown via modelling that distributed detention installations in an urban catchment can cause increased peak flows downstream when not carefully located (Debo 2002). As a result of the alterations that detention causes to the timing of the hydrographs, outlet hydrographs from detention systems can combine with flow from other sub-catchments to produce higher flow rates than under previous conditions in the regional catchment (Ravazzani *et al.* 2014).

In response to this, the more complex and exhaustive alternative to the micro-management approach is the total catchment approach. Using this method of assessment, the objective becomes a necessity to ensure that adverse hydrologic impacts are mitigated at all potential locations within the catchment.

In the model of a regional catchment presented in Figure 2-4 by Del Giudice *et al.* (2014), the effect of a detention system in a downstream river section is described. The hydrograph discharging from the regional catchment (*WSf*) at its outlet point (*f*) is dependent upon the shape and timing of the hydrograph discharging from the local catchment (*SWi*) at its outlet point (*i*). In this example, the total catchment management objectives require that the detention system ensures attenuation of flow at both point (*i*) and point (*f*).

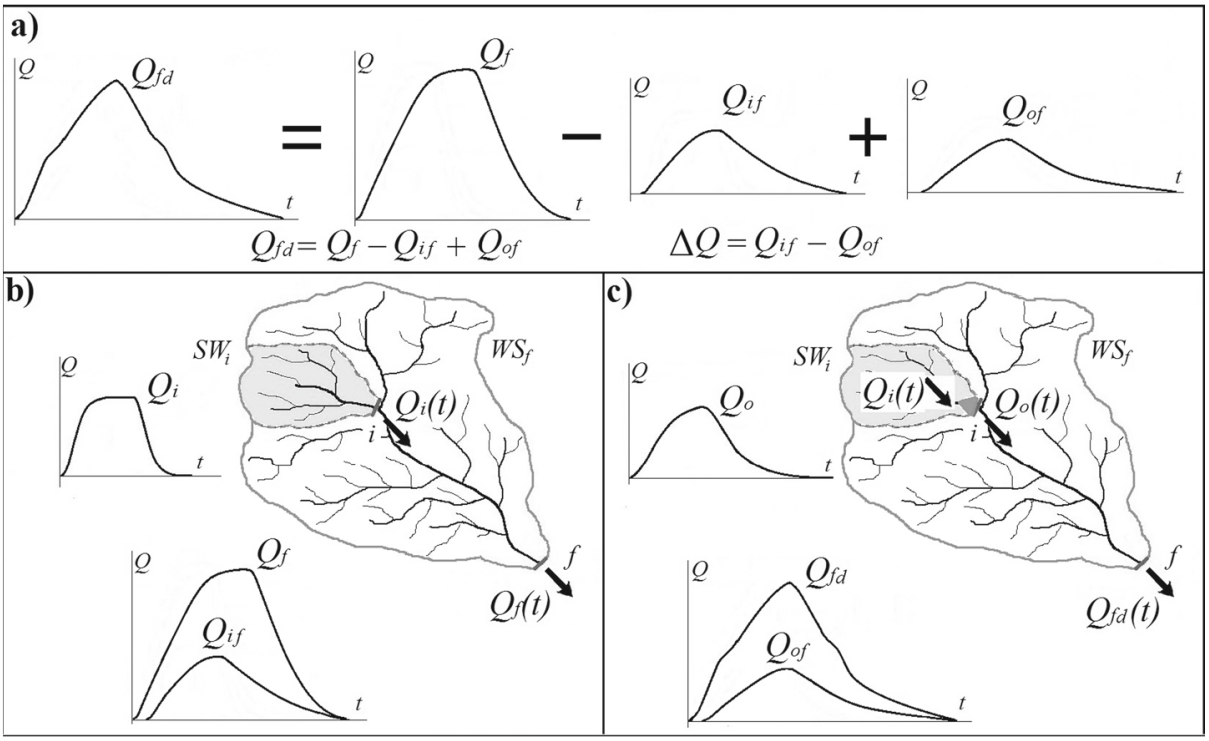

Figure 2-4 – Effects of Detention in a Regional Catchment (Del Giudice et al. 2014)

This dynamic interaction between local and regional hydrographs and the impact that detention can have on regional catchment runoff is the reason for total catchment objectives for designing stormwater detention. Within the total catchment approach to selecting design objectives for detention systems, there are two further sub-categories:

2.4.2.1 At-Source Detention Strategies

Whilst similar to micro-management strategies in the way that all new developments are responsible for installing their own detention system to comply with regulations, at-source strategies for total catchment management have regional strategies in place that govern the release of stormwater from the site. At-source strategies for total catchment management typically involve the specification of a permissible site storage or site discharge rate, obtained from the integration of storage models and theoretical drainage system behaviour of the catchment (Phillips 1989). At-source detention strategies are attractive to both

Councils and developers, because inadequate existing stormwater drainage systems are commonly a strong ground for objecting to developments. At-source detention completely eliminates flooding as a ground for objections, as the development has a strict and prescriptive way to demonstrate compliance without any subjectivity in the calculation process (Lees and Lynch 1992).

Australian examples of catchments that include at-source detention include the Shea Creek Catchment, which is managed by the South Sydney City Council and has used at-source detention strategies for total catchment management since the mid 1980's. A policy is enforced to attenuate all runoff up to the 100 year event to allow for a discharge of no greater than 178 l/s/ha, which is a calculated release rate with allowable regional flooding consequences (Beecham *et al.* 2005). In the City of Wollongong, different catchments have different Permissible Site Discharge (PSD) and Site Storage Rates (SSR) that are based upon a regional catchment assessment, aimed at preventing any increase in peak discharges occurring at any downstream location across a range of events varying in magnitude from a 5 to 100 year average recurrence interval (Silveri and Rigby 2006). In the Upper Paramatta River catchment, policies exist to limit PSD to 80 l/s/ha or provide a SSD 470 m^3/ha, which are based upon extensive calculations and catchment modelling (Lees and Lynch 1992).

In Singapore, at-source strategies for total catchment management require detention to ensure that the maximum peak runoff to be discharged to the public drains will be calculated based on a runoff coefficient of 0.55 (Goh *et al.* 2017).

Mandates for at-source detention are also often coupled with mandates for rainwater harvesting to be included in new development projects. van der Sterren and Rahman (2015) provide an example that proposes modified allowable site discharge rates that account for the existence of storages and re-use of rainwater throughout the catchment.

Whilst at-source stormwater detention strategies are known to have issues in application, they remain attractive stormwater management policy for councils, largely because they are easy to implement, enforce and assess. There is a sense of fairness among land developers when all new development is required to provide detention to account for its own development intensity (Debo and Reese 2002). It is also arguably flexible for unplanned development in expanding communities to require stormwater detention to be installed in parallel with the progress of an evolving city (Shea 1996).

For at-source objectives to be successful in the achievement of total catchment assessment criteria, the designer of the at-source detention basin needs information about an allowable discharge pattern that is suitable for the total catchment. Phillips *et al* (2016) make assumptions that this information is available in local planning schemes or is provided by local authorities, which is not usually the case. For local authorities to be able to provide allowable discharge patterns for each development upon application, the local authorities would be required to hold large, complex regional hydrologic and hydraulic models of the total catchment with allowable levels of development and flooding risk throughout. A practice that is not commonly observed.

Fundamental flaws exist in the implementation of at-source objectives that result when there is uncertainty regarding the allowable discharge pattern for the outlet of detention systems in new developments. Confident design of at-source detention requires a full total catchment assessment that is usually impractical for each small-scale development. The outcome is an alarmingly common adoption of micro-management objectives for stormwater detention design being implemented as misguided attempts to achieve total catchment objectives via at-source control techniques.

2.4.3 Distributed Detention Strategies

Distributed detention strategies require the identification of specific and optimised locations for the installation of detention throughout the regional catchment, designed as a network (Travis and Mays 2008).

Ideally, for successful flood control, it is widely appreciated that detention systems should be considered as a network, rather than individually. Studies in regions including Asia (Duan *et al.* 2016, Tao *et al* 2014, Wang *et al.* 2017), Europe (Bellu *et al.* 2016, Ravazzani *et al.* 2014) and the Americas (Kaini *et al.* 2007, Shuster and Rhea 2013, Su *et al.* 2010) have developed effective models and algorithms for optimized distributed detention locations within the regional catchment.

Whilst distributed detention is strongly recommended and not debated by this research, major constraints exist with implementation of the theory. Firstly, distributed detention needs to be carefully planned for at a regional scale, often requiring participation by multiple municipalities or even nations. Secondly, distributed detention requires significant land dedication, often at very specific locations that will not achieve the desired

outcomes if spatially negotiated. These factors make distributed detention design and implementation a complicated act that is infrequently adopted.

Poelsma *et al* (2013) describe the benefits and efficiencies of wider scale assessment, however acknowledge that the implementation of distributed detention is limited in practice, replaced largely by planning policies that prescribe at-source assessment and design of detention.

2.5 The Regional Effect and "The One Third Rule"

Researchers have long known that the location of stormwater detention has a critical role in its performance. McCuen (1974) was one of the first to express concerns regarding the location of stormwater detention, highlighting the fact that the inappropriate location of detention structures can have outcomes that conflict with their intentions. McCuen (1979) went further to state that the timing change caused by a detention basin can accidently result in increased downstream flooding in some applications.

Watson and Adams (2011) provide a critical review of detention as a solution to flooding and water quality issues, noting that any increase in the total amount of water sent downstream should obviously result in increased flow and flooding and some downstream location. A case study of the Valley Creek in the United States is provided by Watson and Adams (2011), which has experienced recent and rapid urbanisation with the construction of several hundred detention basins. It is noted that the detention systems have been ineffective in the mitigation of increased rates of runoff and erosion as well as decreased rates of recharge and baseflow.

Figure 2-5 – Erosion in the Valley Creek (USA) Resulting from Recent Urbanisation Regardless of the Installation of Several Hundred Detention Basins (using Micro-Management Techniques) (Watson and Adams 2011)

Emerson *et al.* (2005) provide a detailed analysis of an American catchment with an area of 62 km^2 and a public policy of installing stormwater detention for all new developments. With the micro-management principles in place, the objective for each detention in the catchment is to limit a site's post-construction peak flow rate to or below its pre-development level for 2- through 100-year storms. In many of the results, including the example presented in Figure 2.6 below, the installation of the detention around the catchment has resulted in increased regional catchment outflow.

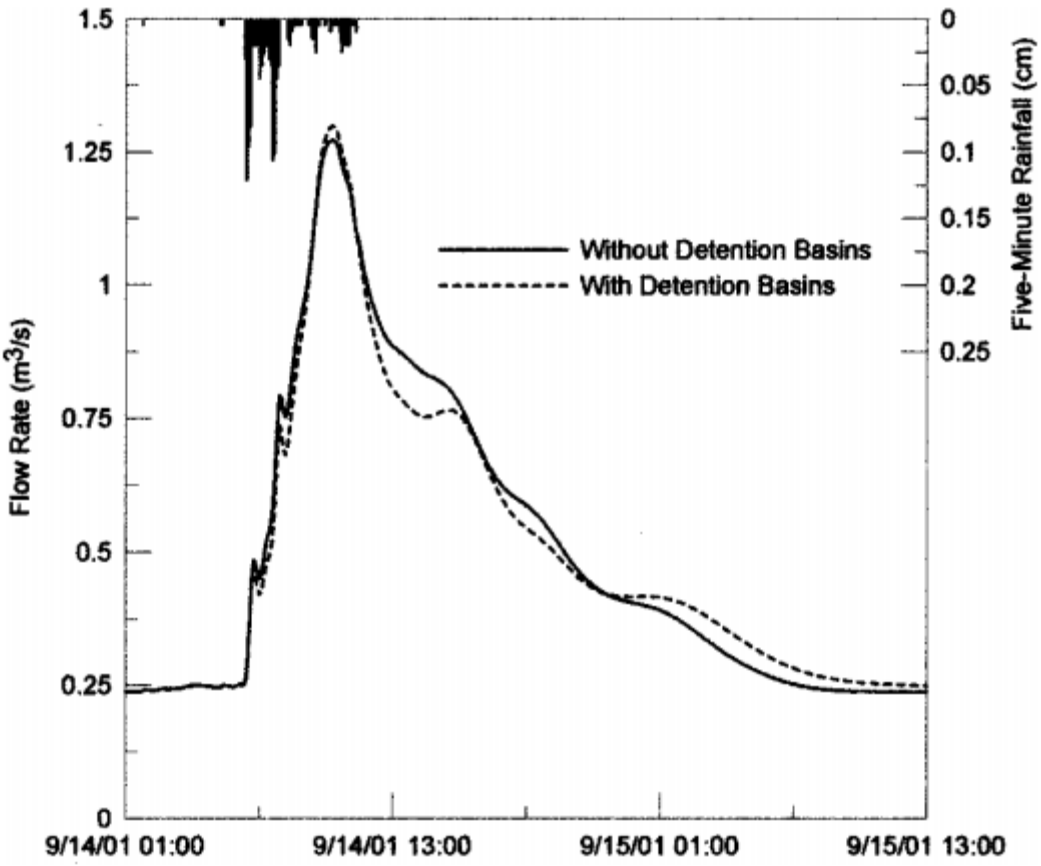

Figure 2-6 – The Regional Effect (Emerson et al. 2005)

The Queensland Urban Drainage Manual (QUDM) (Institute of Public Works Engineering Australia 2017) identifies the fact that coincident flood peaks are a problem that is likely caused by the existence of several basins within a drainage catchment, or basins located within the lower reaches of a waterway. QUDM also notes that it is inappropriate to consider the impact of a single development in isolation from the cumulative effects of full catchment development.

Known as "the regional effect" (Bennett and Mays 1985, McCuen 1979, Ferguson and Deak 1994), the notion that poorly-designed detention system(s) can cause one hydrograph peak to lag or be extended to such a time that it causes a coincidence with another hydrograph peak is generally well acknowledged at this point in time.

Many authors have described the regional effect, proving its importance but usually providing limited recommendations to design engineers, apart from recommendations for complete assessment of the downstream waterway (Saybert 2006).

Some efforts to provide practical solutions have been presented. For example, Flores *et al* (1982) suggested that detention is not suitable for the lowest 20% of a catchment. Leise

(1991) suggested that detention should only be provided within the upper two-thirds of a catchment. Studies have also provided specific geographical analysis, for example Saunders (2008) confirmed by modelling that the regional impact can in certain catchments in the southern regions of the United States be mitigated if a development discharge comes and goes before the arrival of the primary watershed peak flow, with a secondary provision that detention basin outflow should be reduced to 50 to 80% of the pre-development discharge as an alternative.

On the complete opposite site of the spectrum, hydrologic models by Ogawa and Male (1983) indicate that the efficiency of detention systems at attenuating flood episodes increases with their distance downstream. They suggest that a single downstream detention may reduce floodwater velocity and volume better than multiple upstream systems in extreme conditions. They also note that the farther downstream a detention is placed, the larger its size will need to be to effectively detain floodwaters.

An axiom has emerged in current practice that stormwater detention is generally not applicable to the lower third portion of a catchment, often referred to as “The One Third Rule”. The general theory is supported by developing research (McEwan 1974, McEwan 1979, Flores, Badient and Mays 1982, Leise 1991, Goff and Gentry 2006, Saunders 2008), however, due to its subjective nature and in favour of equality for all development sites, the theory is often disregarded in policy and the detailed design of development sites. Micro-management objectives unfortunately continue to prevail in public policy despite their known shortcomings.

2.6 Hydrograph Production in Practice and Progress

The design and assessment of a stormwater detention system requires an appreciation for the creation and manipulation of runoff hydrographs. There are a number of ways in which hydrographs can be generated with varying levels of complexity and accuracy.

2.6.1 The Rational Method and Graphical Approximations

Simple techniques to estimate the peak flow of a hydrograph, such as the Rational Method (Mulvaney 1851, Kuichling 1889) have been in practice for more than 165 years. The calculation inputs include catchment area, rainfall intensity and a single coefficient that accounts for aerial and groundwater infiltration losses.

Using the rational method or an alternative simplistic means of estimating peak flow, basic linear hydrographs can be approximated using either triangular geometry (Abt and Grigg 1978, Donahue *et al.* 1981, Chow 1988, Hong *et al.* 2006, Hong 2008), or trapezoidal geometry (Burton 1980, Guo 1999, Hong *et al.* 2006).

Graphical hydrograph approximations are inherently basic and do not account for any variance in important parameters such as rainfall distribution over time, losses, or the complexities of the catchments response to rainfall and the movement of water through the catchment. They are however suitable for many small-scale engineering applications, such as the sizing of a stormwater drainage pipe.

2.6.2 Unit Hydrograph Methods

A unit hydrograph is a direct runoff hydrograph resulting from one unit of constant intensity uniform rainfall occurring over the entire catchment. The principal concept underlying the application of a unit hydrograph is that each catchment has one unit hydrograph that does not change (in terms of its shape) unless the catchment characteristics change (Weaver 2003). It is a simplified system to create a single hydrograph shape that can be scaled to account for differing events within the catchment or even differing sizes of sub catchments.

Figure 2.7 below provides a summary of the critical parameters, including the discharge (q), peak discharge (q_p), time (t), time to peak (T_p), time of recession (T_r), and time of concentration (T_c). All of which can be determined using analytic equations.

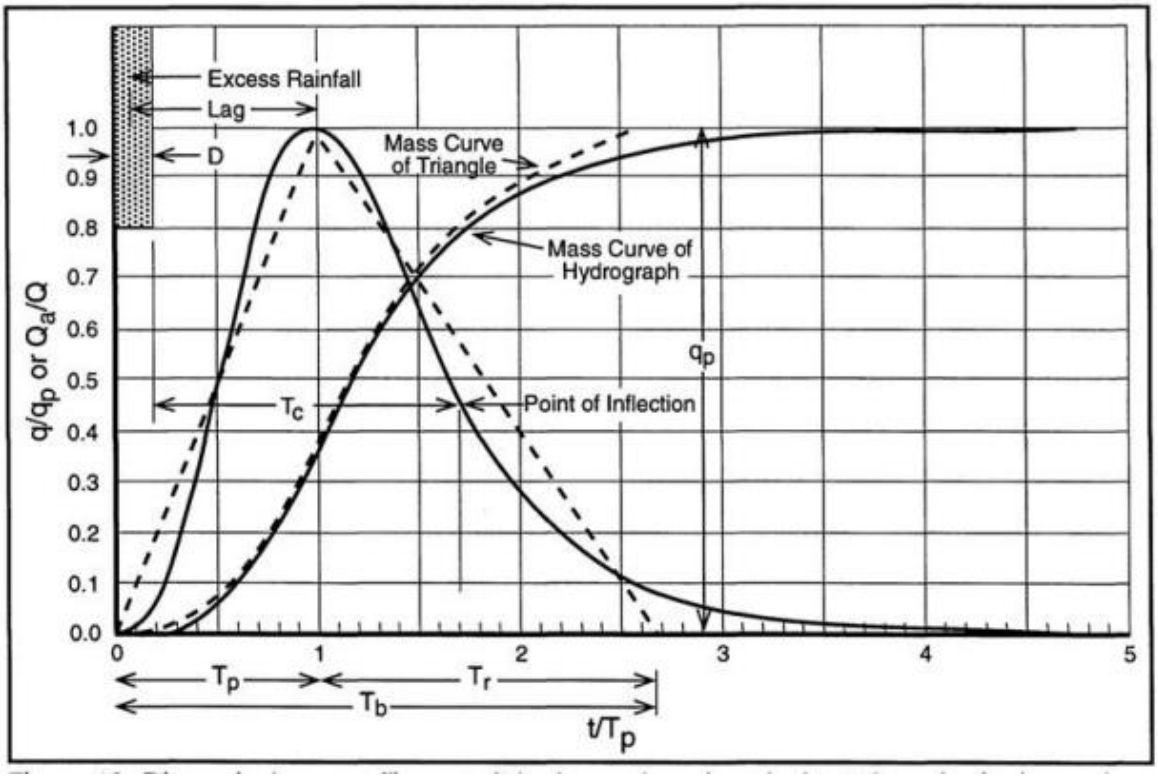

Figure 2-7 – Dimensionless Unit Hydrograph Parameters (United States Natural Resources Conservation Service)

Synthetic unit hydrographs relating to design rainfall events have been in development since the 1930's. Sherman (1932) developed the unit hydrograph technique, describing it as the linear system of surface runoff on a given basin resulting from effective rain falling for a unit period of time. Bernard (1935) used a graph to plot the conversion of rainfall into runoff with catchment characteristics incorporated into a distribution function. Snyder (1938) and McCarthy (1938) developed empirical equations to estimate critical points of the hydrograph, such as peak discharge, time to peak, base period, and total length. Clark (1945) developed methods of isochrones to develop a time–area histogram with curve numbers to define the catchment paraments. Maidment *et al.* (1996) made advances which account for both translation and storage effects in the watershed and the upstream drainage area for local velocity.

The unit hydrograph method is still in practice and development, with continuing modern studies aiming to calibrate the unit hydrograph equations to better suit real recorded rainfall events (Bhuyan *et al.* 2015, Cho *et al.* 2018). However, the method is largely unchanged since the 1970's and not commonly used for complex time dependent applications such as detention design, due to its simplicity and limited accuracy (Hoffmeister and Weisman 1977).

The obvious limitation of the unit hydrograph method is the assumption of constant intensity uniform rainfall over the entire catchment, which does not allow for any appreciation for temporal patterns or variable rainfall pattern probability. The importance of the temporal distribution of rainfall and the need for improvements upon the previously adopted simplistic unit hydrograph methods with constant rainfall assumption were described as early as Cordery (1971), with a strong push since this time for a departure from the unit hydrograph technique and a movement toward runoff routing.

2.6.3 Runoff-Routing Methods

The modelling of rainfall and runoff processes using computers to perform runoff routing is presently the standard practice for high quality and accurate results. In this process, temporal and spatial rainfall distributions of rainfall are converted to runoff hydrographs by applying hydrodynamic laws and using various linear and nonlinear numerical schemes to route hydrographs over land (Akram *et al.* 2014).

The continuity equation (Equation 2-1) and its general solution (Equation 2-2 and Equation 2-3) are used in the process of all runoff-routing methods, where the inflow (*I*) is routed through a storage (*S*) to produce an outflow (*Q*) during a period of time (*t*) divided into (*i*) time steps.

$$\frac{dS}{dt} = I(t) - Q(t) \qquad \textit{2-1}$$

$$\int \frac{dS}{dt} dt = \int I(t)\ dt - \int Q(t)\ dt \qquad \textit{2-2}$$

$$\frac{2(S_{i+1} - S_i)}{\Delta t} = (I_{i+1} + I_i) - (Q_{i+1} + Q_i) \qquad \textit{2-3}$$

In the early development of methods for creating unit hydrographs, the continuity equation and basic versions of runoff routing theory were used to provide the experimental basis for the simplified graphical representation of the unit hydrograph. The evolution of more accurate methods of determining unit hydrograph parameters have all relied upon studies that include first-principles runoff routing, including Clark (1945), Nash (1959a), James *et al.* (1987) and Meadows *et al.* (1991). In the early studies the computation required to perform runoff-routing for each design storm application was considered to be infeasible and the unit hydrograph was a reasonable approach for practitioners. Nowadays computers are easily capable of performing runoff routing, making the unit hydrograph method somewhat redundant.

2.6.3.1 Linear routing

The most common form of linear routing is the Muskingham method, as originally described by Nash (1959b):

$$Q_{i+1} = C_1 I_{i+1} + C_2 I_i + C_3 Q_i \qquad 2\text{-}4$$

Where:

$$C_1 = 1 - \frac{K(1-c)}{\Delta t} \qquad 2\text{-}5$$

$$C_2 = 1 - \frac{K(1-c)}{\Delta t} - c \qquad 2\text{-}6$$

$$C_3 = c \qquad 2\text{-}7$$

$$c = \frac{-\Delta t}{e^{K(1-X)}} \qquad 2\text{-}8$$

In Muskingham routing the value of *X* is a physical parameter that reflects the flood peak attenuation and hydrograph shape flattening of a diffusion wave in motion. The value for *K* is a variable dependent upon the catchment Imperviousness (*Imp*) and area (*A*) which has a commonly adopted solution by Boyd *et al.* (1996) that considers imperviousness (*Imp*) and the catchment area (*A*):

$$K = 3600(1 + Imp)^{1.9}.1.3(A)^{0.38} \qquad 2\text{-}9$$

In practice, linear routing is typically limited in application to stream flow movement where a known hydrograph is within a stream and calculations are required to appreciate the modification to the hydrograph as it moves through the channel as a kinematic wave.

2.6.3.2 Non-Linear Routing

For routing a rainfall pattern through a catchment to determine an overland flow hydrograph, non-linear routing is generally the preferred method. The modern method of non-linear runoff-routing is focussed on the storage parameter of the continuity equation and sometimes referred to as Laurenson routing, after Laurenson (1962). In the original studies, storage was identified as a key input to hydrograph generation, with its correlating effect of delaying and attenuating runoff. Laurenson's original equation to solve for catchment storage is provided as Equation 2-10:

$$S = BQ^{n-1} \qquad 2\text{-}10$$

Where *n* is a factor of the catchments nonlinearity. A solution to *B* was developed by Goyan and Aitken (1976) and is currently used in the software program RAFTS, given as Equation 2-11:

$$B = 0.285A^{0.52}(1+U)^{-1.97}{S_c}^{0.5} \qquad 2\text{-}11$$

Where *A* is the catchment area, *U* is the factor of urbanisation and *Sc* is the slope of the catchment.

Laurenson's model was further developed with the release of the RORB runoff routing software package (Laurenson and Mein 1990). The revised storage discharge relationship was provided by the Equations 2-12 and 2-13:

$$S = 3600kQ^m \qquad 2\text{-}12$$

$$k = k_c k_r \qquad 2\text{-}13$$

Where k_c is an empirical coefficient applicable to the entire catchment and k_r is a dimensionless ratio describing 'relative decay' and given by Equation 2-14:

$$k_r = F_i \frac{L_i}{d_{av}} \qquad 2\text{-}14$$

The factor F_i describes whether the reach is natural or lined. L_i is the length of the channel represented by storage *i*, and d_{av} is the average flow distance in the channel of the sub-area.

Another popular method that deviates from the original Laurenson equations that has been developed by Boyd *et al.* (1993, 1996, 2006 and 2012) and adopted by the WBNM software program to solve for catchment storage, given as Equations 2-15 to 2-17:

$$S = kQ^m \qquad 2\text{-}15$$

$$k_{pervious} = \left(C_{lag}\, A^{0.57}Q^{-0.23}\right)Q^{1-m} \qquad 2\text{-}16$$

$$k_{impervious} = \left(0.1\, C_{lag}\, A^{0.25}\right)Q^{1-m} \qquad 2\text{-}17$$

In Boyd's method, the catchment is separated into pervious and impervious portions and the ultimate hydrograph is the summation of each. The non-linearity parameter *m* is typically a constant value of 0.77 for potentially saturated conditions (Rezaei-Sadr *et al.* 2012). The C_{lag} coefficient is a dimensionless parameter that is used to calibrate the equations to gauged flow or other methods of estimation.

With the introduction of improved computation speed of hydrologic software, runoff routing for hydrograph generation is now more feasible, and due the improved level of

accuracy and ability consider varied shapes and patters more commonly applied than the simple unit hydrograph for complex applications. RORB, RAFTS and WBNM are all standard software programs utilised by practitioners and recommended by industry guidelines.

2.6.4 Known Issues with Current Best Practice

The generation of a hydrograph using runoff routing requires three primary processes:

1. Rainfall Model
2. Runoff Production
3. Hydrograph Production

The production of a reliable hydrograph involves the careful selection of inputs including the critical storm duration, areal reduction factor, spatial pattern, temporal pattern, runoff routing model, model parameters, treatment of baseflow and account for losses (Hill and Thompson 2016).

Figure 2-8 provides a traditional conceptualisation of hydrograph formulation using runoff routing that has been generally unchanged in Australia since the first release of the Australian Rainfall and Runoff Guideline in 1958. A number of the components of this conceptual model are in the process of undergoing significant change as a result of developments in academic literature and hydrologic theory. In addition, a number of additional considerations have been added to the traditional model.

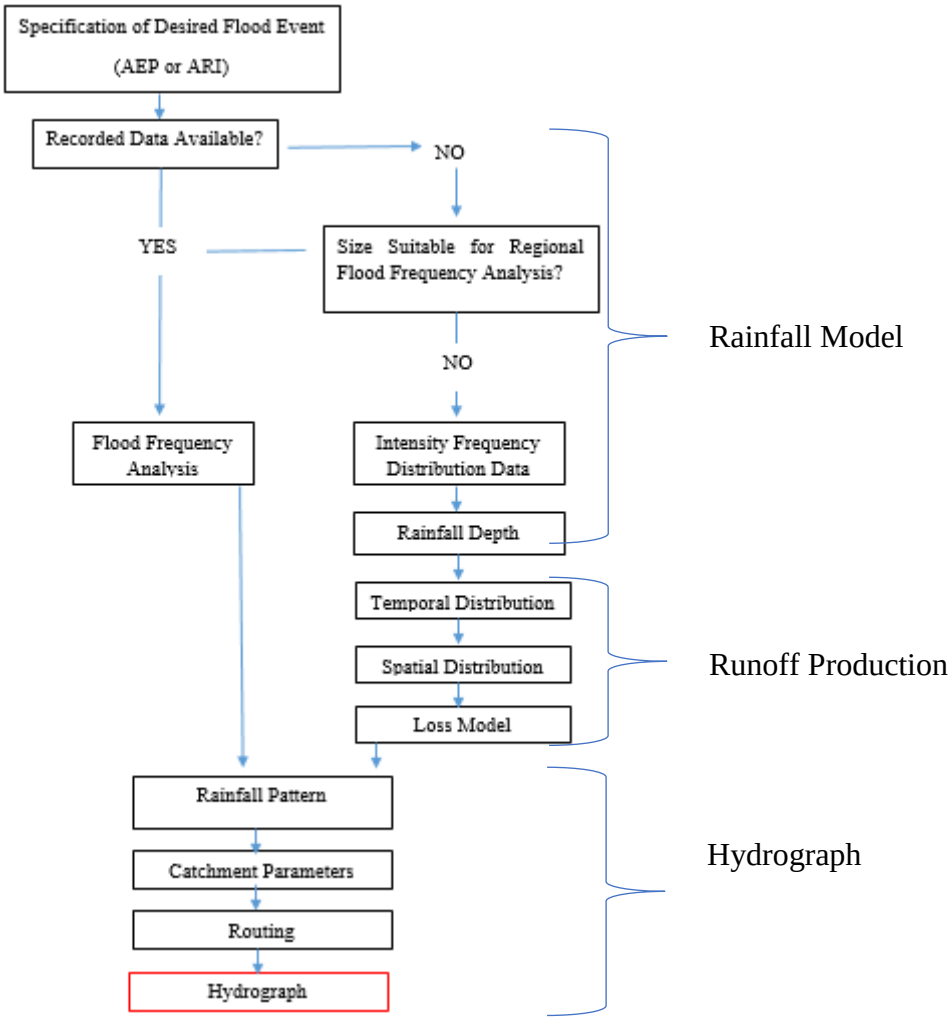

Figure 2-8 – Current Practice for Hydrograph Formation

2.6.4.1 Rainfall: Recorded or Design?

In an ideal world, all hydrologic engineering projects would be located immediately adjacent to a gauging station with many years of recorded flow, depth and rainfall data. The catchment would also be unaltered in imperviousness or drainage efficiency throughout the recording era. In this utopia, at-site flood frequency analysis methods can be used to fit recorded flood peaks to probability distributions and predict the peak of events of certain return periods, using methods such as Gumbel distribution (Gumbel 1958), Generalized Extreme Value (Morrison and Smith 2002) or Log Pearson's III, which has been the recommended method for practice since the 1970's (Srikanthan and McMahon 1981). A thorough overview of the development of at site Flood Frequency Analysis techniques is provided in Jin and Stedinger (1989) and Kuczera (1999).

In practice, particularly in the development of urban engineering projects, this is rarely the case. Small urban development sites requiring rainfall analysis for engineering design are

seldom located close to reliable gauging stations suitable for the application of at-site flood frequency analysis. The estimation of flood frequency statistics for ungauged catchments is a continuing problem of great practical interest.

One alternative practice when gauging is not available is regional transformation, or regional flood frequency analysis, which involves taking recorded data from other catchments and making alterations to allow for application to an ungauged site. Haddad and Rahman (2011) provide an overview of regional flood frequency analysis techniques currently in practice and development in Australia. Recent advancement in the research and development of regional flood frequency analysis models has provided improved means for the utilisation of recorded rainfall away from at-site gauging stations to estimate the design rainfall at-site for a project. These models include L-moments (Hosking and Wallis 1997) and extended work by the International Association of Hydrological Sciences (IAHS), who launched the Predictions in Ungauged Basins initiative for the decade of 2003–2012 (Sivapalan *et al.* 2003), resulting in a large number of studies adopting fuzzy logic, soft computing and neural computing techniques to distribute recorded rainfall (Kumar *et al.* 2015). Whilst proven effective on larger catchments where resources permit the level of analysis required, these methods are rarely suitable for small scale urban design.

Furthermore, the usage of flood frequency analysis presents limitations and requires considerable judgement on behalf of the practitioner. As described by Kuczera and Franks (2016), the usage of recorded flood peaks for flood frequency analysis includes a product of complex joint probabilities, including the interaction of many random variables associated with the rainfall event, antecedent conditions and the rainfall-runoff response transformation. Urbanisation of a catchment during the gauge recording period is also a major factor that can skew the results of a flood frequency analysis.

The development of regional intensity-frequency-duration tables at a national scale has therefore been prioritised in Australia to provide prescriptive and unified design rainfall estimation.

In 2016 an updated Australia wide set of intensity-frequency-duration data was released by the Australian Bureau of Meteorology that was the result of an 8-year project involving gridding techniques and regionalisation of statistical data producing scientifically rigorous and defensible data that can be used across the whole of Australia (Green *et al.* 2016). This

rainfall data provides a means of reliably determining the probable rainfall for design events at a development site, without the gathering of recorded data by a practitioner or any on-site or regional flood frequency analysis.

2.6.4.2 Considering Rainfall Outside of the Design Burst

The possibility for a catchment to be wet prior to the onset of a storm event and the effect that this has on flood prediction has been the topic of major debate for several years (Cordery 1970).

In January 2011 there was a flood event in Queensland, Australia that killed 33 people and was the most expensive in Australia's history (van den Honert and McAneney 2011). Dissection of the flood event has directed attention toward the significant amount of rainfall that occurred prior to the actual event, including the wettest Spring and December on records (Bohensky and Leitch 2014). The existing wetness of the catchment at the time of the major rainfall burst resulted in flood volumes that far exceeded previous predictions for runoff from the associated depth of rainfall during the event.

Excluding pre- and post-burst rainfall periods when modelling flood events can result in inaccuracies of the peak flow calculation, largely due to the amount of design burst rainfall that disappears to losses and attenuation of the hydrograph peak (Rigby and Bannigan, 1996). Modelling of the pre-burst rainfall is therefore an important way to account for the initial moisture state of a catchment, and particularly necessary in coastal catchments (Loveridge *et al.* 2015b).

Referred to as antecedent conditions, there is currently developing awareness that rainfall in the days and hours prior to a design storm can have a significant impact on hydrograph shape and peak. Research into antecedent conditions in Australia by Blakie and Ball (2005) have considered rainfall patterns during a five-day period prior to major recorded events and determined that the majority of antecedent rainfall occurs in the single day before a major event, however without predictable patterns. Phillips *et al.* (2014) undertook a similar one, three and seven-day analysis also in Australia. Both of these studies did not reveal definitive patterns that could be adopted deterministically, indicating that antecedent conditions should ideally be measured individually for each catchment based on recorded data.

In the United States antecedent conditions are measured manually using extended time periods, typically three months prior (Minnesota Board of Water and Soil Resources 2015).

Currently in Australia, methods to account for antecedent conditions are generally factored into loss models based on individual catchment analysis. The approach given by Cordery (1971) is still generally accepted, whereby the antecedent precipitation index (API) is calculated by discounting the time series of daily rainfall prior to the event using an empirical decay factor. The API is then related to the initial loss of the catchment, removing the initial amount of rainfall that lands upon the catchment during the design event. Design guidelines in Australia will regularly provide recommendations for antecedent conditions, alleviating the need for API calculation, commonly providing conservative parameters.

There is debate regarding whether pre-burst rainfall varies with the probability of the storm event. Pathiraja *et al.* (2012) found that the antecedent moisture does varies with AEP. An alternative pilot study into pre-burst rainfalls by Hill *et al.* (2014) found no significant trend with AEP.

The Australian Bureau of Meteorology currently provides values for median, 10%, 25%, 50%, 75% and 90% preburst depths that can be applied to models as antecedent rainfall conditions based upon the design rainfall intensity.

2.6.4.3 Rainfall Loss Models

Rainfall loss (or infiltration) is an essential component in the prediction of overland flow, which must be considered accurately to achieve optimum runoff rates. Loss is defined as the precipitation that does not appear as direct runoff, and is attributed to the following key processes (Hill and Thompson 2016):

- Interception by vegetation;
- Infiltration into the soil;
- Retention on the surface (depression storage); and
- Transmission loss through the stream bed and banks

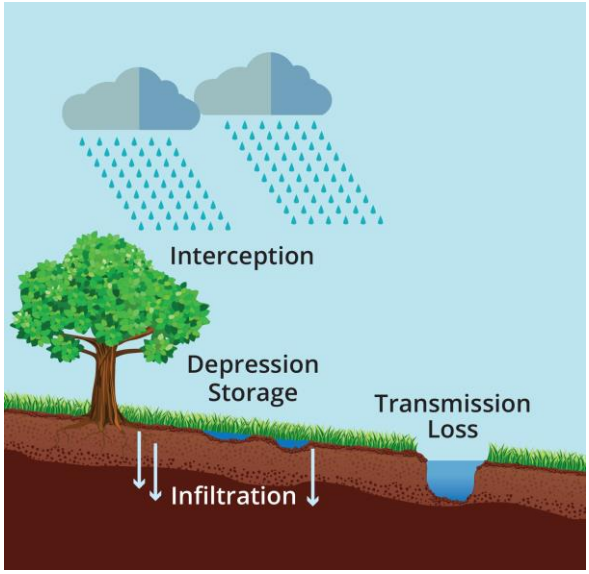

Figure 2-9 – Losses (Hill and Thompson 2016)

The most commonly adopted loss model is the initial loss–continuing loss (*IL–CL*) model, where runoff begins when the rainfall intensity exceeds the infiltration capacity of the soil. In Australia, the *IL-CL* model is currently recommended for application in hydrologic processes contributing to floods (Ladson and Nathan 2016).

Initial loss has an important role in managing antecedent rainfall conditions. As described in Figure 2-10, the initial loss can be separated into losses for a complete storm (IL_s), being the average catchment rainfall that occurs prior to the commencement of significant surface runoff at the catchment outlet, and loss from the storm core (IL_c). After the total assigned value of *IL* is reached, only *CL* is taken away from each bar of the rainfall hyetograph and the remaining rain in excess forming the runoff hydrograph.

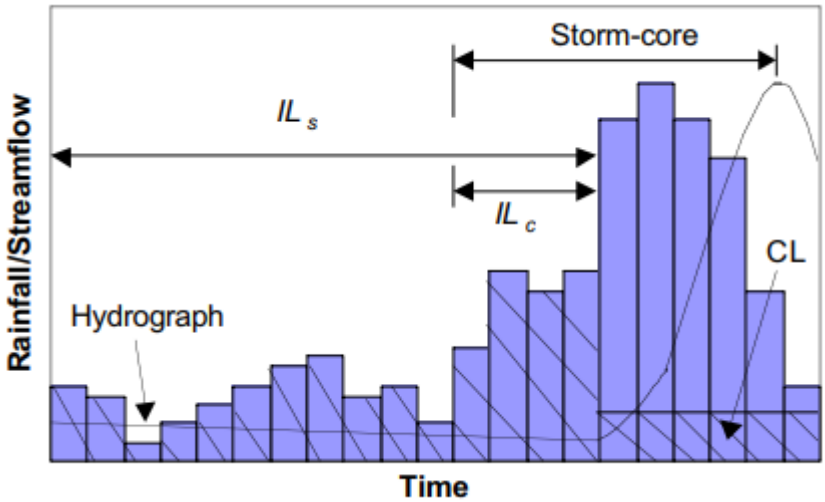

Figure 2-10 – IL-CL *Model (Rahman et al. 2002)*

The initial loss–proportional loss (*IL–PL*) model is an alternative although similar concept, based on the saturated overland flow theory, where runoff is generated from the saturated portions of the catchment. The difference between the two models is described in Figure 2-11.

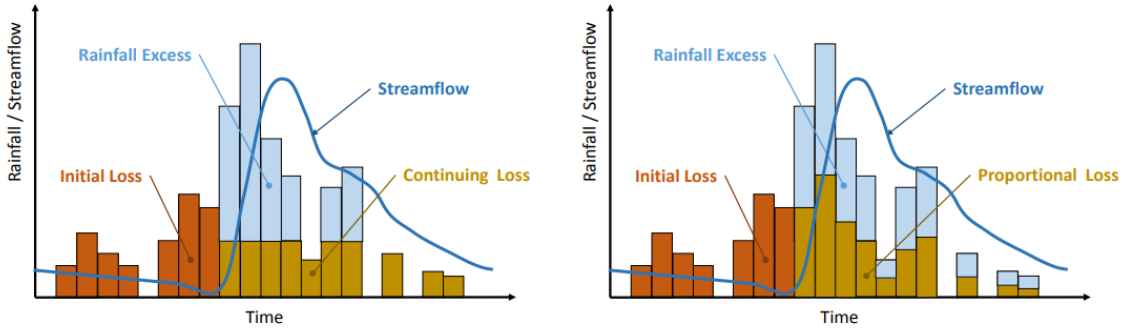

Figure 2-11 – IL-CL and IL-PL Models (Lang et al. 2015)

Another Australian alternative is SWMOD, which is a probability distributed model. SWMOD was developed for use in the south west of Western Australia where saturation excess overland flow is held to be the dominant runoff mechanism for storm events. Its application to other parts of Australia has been shown to be limited (Loveridge *et al.* 2017). Probability distributed storage capacity models offer potential for design flood estimation as they represent flood runoff from saturated source areas and can be used to account for the temporal and spatial variability in runoff generation across a catchment (Hill *et al.* 2014).

The usage of fixed values for losses, which is currently recommended practice in Australia, has received criticism and the use of stochastic losses in design flood estimation rather than fixed values of losses has been recommended (El-Kafagee and Rahman 2011). In an Australian case by Loveridge *et al.* (2013) the *IL-CL* model was found to reproduce the shape of the flood frequency curve better than the *IL-PL* model, with cautions provided surrounding the limitation of fixed variable inputs and recommendations that all design inputs parameters and inputs should be considered as stochastic variables through a full Monte Carlo simulation. Loveridge and Rahman (2014) provide a study using Monte Carlo simulation to compare the performance of different loss models. The loss model that tended to produce the most accurate peak flow estimates when compared to the at-site FFA was found to be the *IL–CL* model, followed by *SWMOD* and the *IL– PL* model. While the *IL–PL* model and *SWMOD* perform quite well in calibration, the two models were shown to have a tendency to consistently overestimate losses when it comes to design.

Further problems with the *IL-CL* method also exist surrounding assumptions of average values. Rahman *et al.* (2002a) found that the use of a mean (or median) value of initial loss (instead of a probability distributed values) reduces the calculated flood magnitudes significantly, particularly at ARIs smaller than 10 years, confirming that it is difficult to find 'representative' design loss values that will produce unbiased design flood estimates. They described the benefits of considering Monte Carlo simulation of initial loss parameters to improve accuracies and reduce sensitivity to errors due to initial loss selections.

Limitations in the *IL-CL* or the *IL-PL* method also include time step issues. Lang *et al.* (2015) provide scaling factors that can be used for high resolution flood studies to account for the errors that occur at the transition point from *IL* to either *CL* or *PL*.

In the United States and in parts of Asia the runoff curve number (CN) method is adopted to account for losses, whereby soils are classified into groups of four types based on their infiltration capacity with diminishing infiltration curves for each. The CN method originates from the United States National Engineering Handbook (Mockus 1964).

More complex models have also been developed recently to account for water movement within the soil and consider soil layers with exponential infiltration and layer horizons, including models that use Horton's infiltration theory (Davidsen *et al.* 2018, Yoo 2012) and Green-Ampt (Stewart 2018). Based on a comparative study by Millari *et al*, (2015), Green-Ampt infiltration model is considered to be superior over the Horton's model due to its ability to account for the antecedent moisture conditions of the soil, which is a critical factor in determining infiltration rate. Horton's model expresses infiltration as a function of time and considers only surface conditions rather than soil moisture content and other essential soil properties.

2.6.5 Continuous Rainfall Simulation

Continuous simulation is an option for modelling stormwater runoff and assessing stormwater detention by considering the results of a complete (continuous) rainfall time series incorporating flood producing bursts of rainfall, low intensity bursts of rainfall and the dry periods between bursts of rainfall (Ball *et al.* 2016). The idea was originally promoted by Beven (1987) as a way to provide more physically based techniques for prediction of flood frequency characteristics as opposed to the design unit hydrograph.

Continuous simulation rainfall modelling is currently receiving significant research attention. The continuous simulation method of estimating a design flood involves running a conceptual runoff-routing model for a long period of time such that all important interactions (covering the dry and wet periods) between the storm (intensity, duration, temporal pattern) and the catchment characteristics are adequately sampled to derive the flood frequency distribution. In general, pluviograph data of hourly resolution (or less) is used to drive the runoff-routing models (Nathan and Ling 2016).

Continuous simulation can assist by providing accurate examples of real storm events that can correspond to a required return interval. By selecting the maximum annual hydrographs obtained from a synthetic runoff time series calculated using continuous simulation of recorded rainfall, it is possible to determine the design hydrograph with an assigned return time through flood frequency analysis (Grimaldi *et al.* 2012).

2.7 Variable Rainfall Probability

2.7.1 Joint Probability Analysis

One of the most significant of the recent changes in the development of design storm hydrographs is the movement away from a deterministic view of the design storm and toward an appreciation of joint probability and the potential for variance of almost all hydrologic inputs to design hydrograph estimation (Babister *et al.* 2016a, Rahman *et al.* 2002b, Nathan *et al.* 2016). The joint probability approach mimics mother nature in that the influence of all probability distributed inputs are explicitly considered, thereby providing a more realistic representation of the flood generation processes (Nathan and Weinmann 2013).

The joint probability approach has been heavily promoted in recent years due to its ‘holistic’ ability to consider the probabilistic nature of all flood-producing input variables (Charalambous *et al.* 2013, Hill and Mein 1996, Kuczera *et al.* 2006). By considering a probability distribution of variables as inputs to the hydrograph generation process the results yield probabilistic outcomes with the ability to calculate statistical results for mean, median, confidence limits, etc.

In the pivotal study conducted by Rahman *et al.* 2002b, joint probability Monte Carlo modelling was carried out to produce high quality flood frequency curves and the process suggested as a replacement the traditional design event approach. Probability distributed values were used for storm core duration and intensity, losses and temporal patterns. For

temporal patterns, a selection of historically recorded patterns was taken from pluviograph records, with added suggestions that further production of additional design temporal patterns could be generated using the multiplicative cascade model.

Monte Carlo simulation can also go far further than just testing various potentialities for probabilistic analysis of results. The traditional approach to hydrologic modelling assumes that the uncertainty in the input-output representation of the model is attributable primarily to uncertainty associated with the parameter values. This is not realistic for real-world applications and Monte Carlo modelling of all inputs as variables with potential errors can help in identifying critical catchment parameters and sensitivities (Vrugt *et al.* 2008).

2.7.1 Temporal Pattern Uncertainty

Hydrographs developed by using the same rainfall depth but with different temporal patterns can result in design flood variances of as much as 250%, making temporal patterns one of the key design parameters that can have a major impact on the design of hydraulic and water control structures (Bhuiyan *et al.* 2010).

Temporal patterns are one of the variables shown to exhibit a wide variability in their observed values and the use of mean or median values of these variables has been shown to be insufficient to represent real-world hydrograph predictions in modelling (Caballero and Rahman 2014a, 2014b). A movement toward the consideration of multiple temporal patterns for the hydrologic modelling of each design storm event has therefore emerged in recent years. Developing methods include sampling of an ensemble of historic temporal patterns (Loveridge and Rahman 2014) as well as artificial disaggregation of rainfall totals using multiplicative random cascade model (Müller and Haberlandt 2018).

Monte Carlo simulation provides a useful and practical method of replicating hyetograph properties and expanding the database of high intensity rainfalls and resulting hyetographs (Kottegoda *et al.* 2014).

2.7.2 Australian Rainfall and Runoff

The recently replaced version of the Australian Rainfall and Runoff Guideline (Pilgrim 1987) has been the guiding force behind the determination of design rainfall event hydrographs for the last three decades, including the shape of temporal patterns. The temporal patterns provided in Pilgrim (1987) were developed using the Average Variability Method of Pilgrim and Cordery (1975), which analyses the variability of

temporal patterns in each zone in order to produce a single representative pattern for the whole zone.

In Pilgrim (1987), the prescribed methodologies result in the usage of one temporal pattern for each design storm. For each design storm duration there are only two temporal patterns to choose from, one for events with a recurrence of less than thirty years and another for events that exceed thirty years. The whole of Australia is divided into nine zones with a set of temporal patterns provided for each. This simplistic approach has been widely criticized for its inability to manage parameter uncertainly or account for the probabilistic nature of key variables except for the rainfall depth (Wood 1976, Rahmen *et al.* 2002b, Kuczera *et al.* 2006, Nathan *et al.* 2003).

In the latest Australian Rainfall and Runoff (Ball *et al.* 2016), the consideration of a probabilistic array of multiple hydrographs is recommended as opposed to the "simple event" with only one hydrograph result. The recommendation includes considering either a full Monte Carlo sampling process or a simplified approach with an ensemble of ten temporal patterns, which are made available to practitioners. The ensembles of temporal patterns are contained in a national database, including suitable samples of data from real recorded rainfall events throughout Australia, created using the Bureau of Meteorology's pluviograph data, totalling 2290 pluviographs across Australia (Loveridge *et al.* 2015a). Figure 2-12 provides a comparative description of the three techniques.

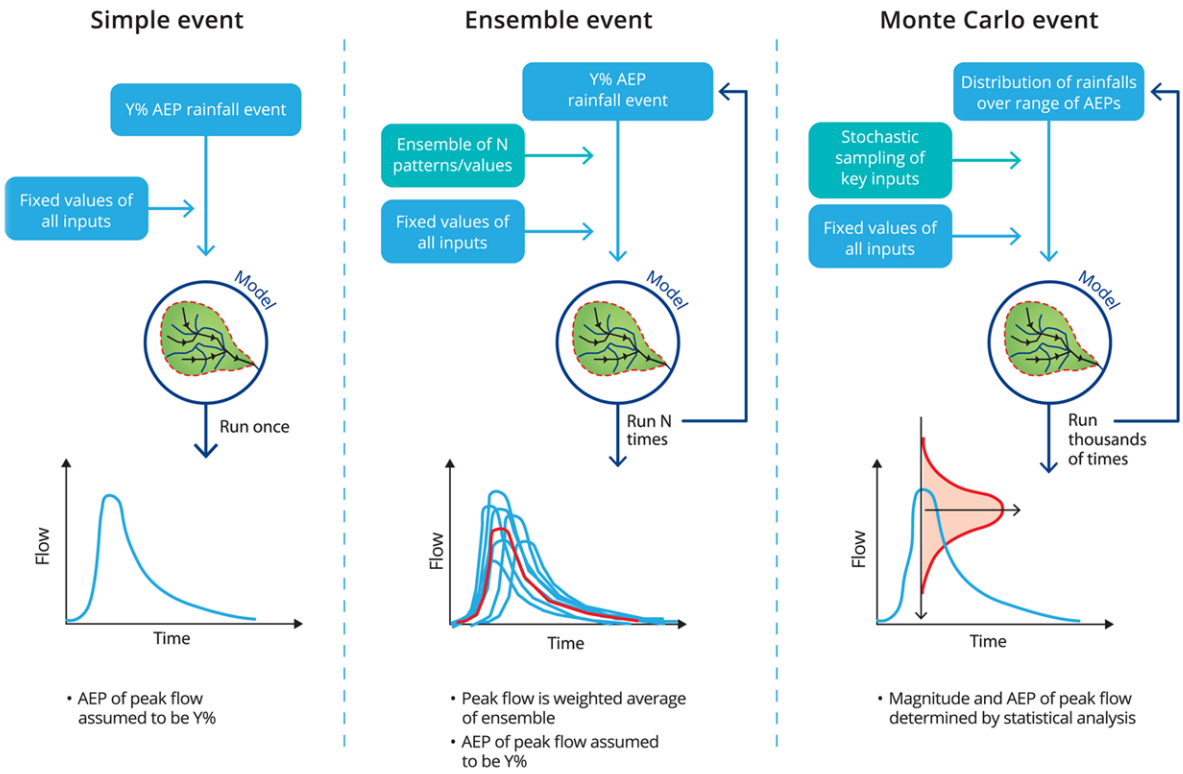

Figure 2-12 – Description of Variable Rainfall Probability (Ball et al. 2016)

3. Research Objectives and Methodology

3.1 Knowledge Gap and Research Questions

Three research questions stand out from the literature that highlight a gap in current knowledge and the need for further investigation:

1. What are the real impacts of micro-management style stormwater detention on regional catchment peak discharge?
2. Can we practically predict the peak flow impact of an urban development and its detention system at a specific downstream location, without conducting a full catchment analysis?
3. Can the answer to these questions suitably account for variable rainfall probability?

A modification to the traditional design procedure for a development site scale stormwater detention system shown in Figure 1-1 is the target outcome of the research.

If an answer to the research questions could be reliably developed, the calculated impact of an urbanised land parcel with and without detention could be determined at important downstream locations. This information could be used to make decisions regarding the installation or avoidance of detention using micro-management design objectives.

A modified design process that accounts for this calculated regional impact is described by Figure 3-1. A new step is added at the commencement of the design procedure, requiring a high-level check of the land parcel's location within the regional catchment and identification of critical downstream locations that could be sensitive to increases in runoff. Using measurements of stream lengths and catchment areas only, the potential impact of the urbanisation of the land parcel on the peak flow of runoff can be determined before conducting any hydrologic calculations. In this modified method, the term F_{dev} refers to the factor of impact that a developed land parcel is expected to incur on the downstream peak flow. The term F_{det} refers to the factor of impact that a development with the inclusion of detention is expected to incur on the downstream peak flow. If F_{dev} is greater than F_{det} then the installation of detention is beneficial, and the design process should proceed. If F_{det} is greater than F_{dev} then the detention is likely to cause an adverse impact on regional peak runoff, and the designer should avoid the installation of detention in favour of upgrading downstream infrastructure to have capacity for the increased land parcel runoff.

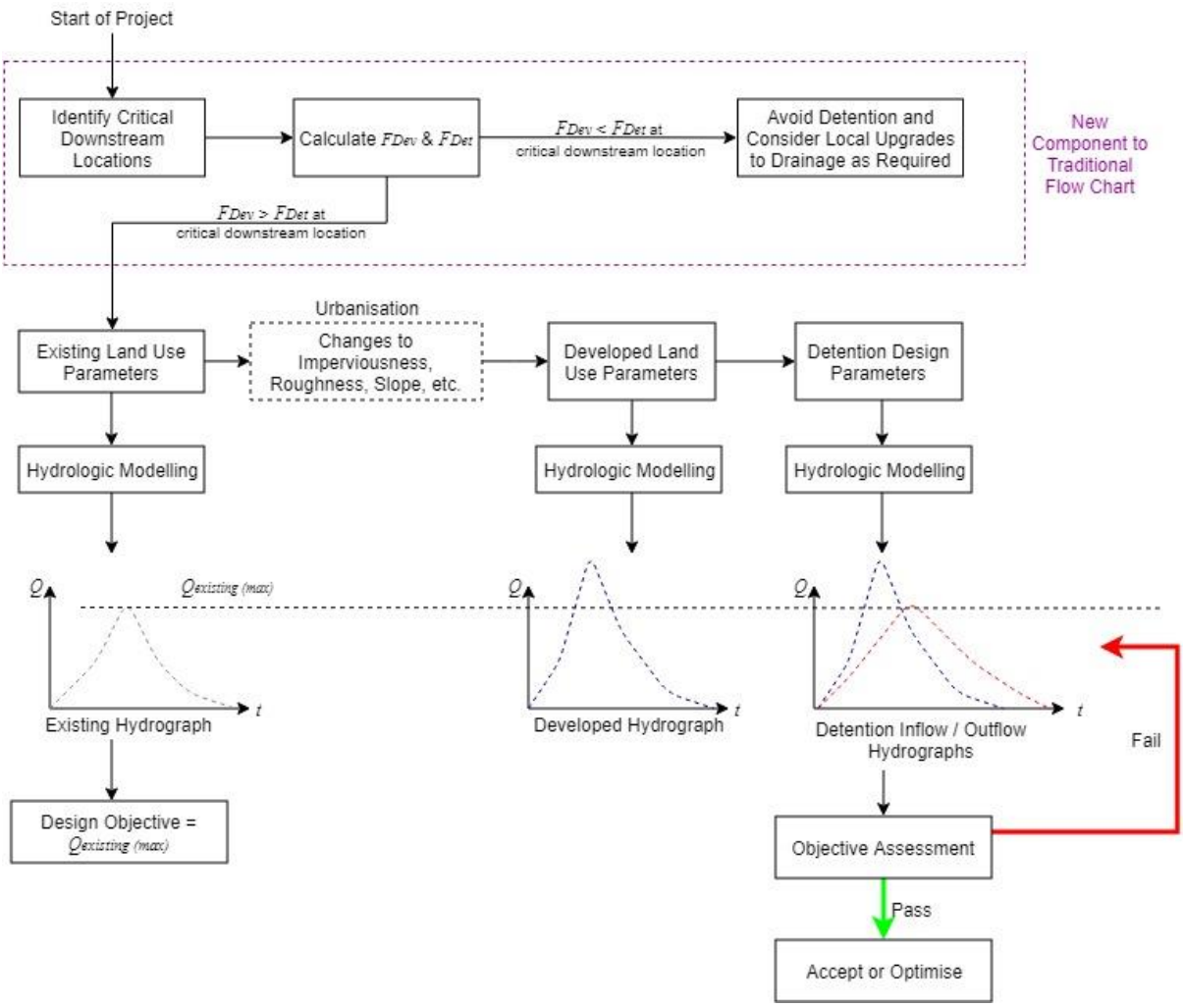

Figure 3-1 – Proposed Alterative Detention Design Assessment Criteria Flow Chart

3.2 Methodology and Overview of Remaining Thesis Chapters

The research methodology has involved the incremental development and verification of a numerical hydrologic model capable of generating hydrographs for regional catchments. The purpose of the model is to ultimately calculate and understand the impact to regional peak runoff that results from urbanisation throughout the catchment, with and without detention.

In parallel to the development of the model, there is a focus on demonstrating the significance and contributing to the advancement of knowledge into variable rainfall probability and its importance as the inflow condition of the detention design process.

In Chapter 4, the foundations are set for the research. A packaged hydrologic software product is used to demonstrate the existence of the regional effect and it is shown that inappropriately sized detention in the lower portions of a catchment can exaggerate increases to regional catchment runoff. An alternative detention design method is presented

that involves simple analytical equations for the hydrograph inflow and a detailed rating curve to describe the function of the detention on hydrograph modification. The programming of the rating curve is the first component of the regional numerical model to be carried forward to further stages of the research.

In Chapter 5, the investigations into variable rainfall probability commence. Ensembles of rainfall patterns are used to replace the singular design event methodology that has been used in the specification of 20 real world detention systems. The results are analysed to understand and quantify the impact of variable rainfall probability on detention design and performance. A numerical model is developed that is capable of performing continuous rainfall simulation using non-linear catchment routing and assessing the performance of the 20 real development sites in response to many years of recorded rainfall. The rating curve from Chapter 4 is incorporated in the numerical model and the added capability for non-linear catchment routing is carried forward to further stages of the research.

In Chapter 6 the numerical model is complete, with the ability to perform non-linear catchment routing and assessment of detention performance using explicitly routed rating curves, developed in stages from the previous chapters. A regional modelling framework is developed that involves sub-catchments and hypothetical urbanisation of varying size land parcels, connected in series with Muskingham channel routing. The exploration into variable rainfall probability also reaches maturity with the programming of a Monte Carlo rainfall generator, capable of creating thousands of randomly generated temporal patters. The model is used to compare the impact on regional peak flow that is in response to urbanisation and detention at varying locations within the regional catchment. Recurring trends are identified in the results and equations are established to predict the results for future applications.

In Chapter 7 a discussion is provided regarding the significance of the research and the applicability of the findings to public policy and practitioners in the engineering industry.

4. An alternative method for on-site stormwater detention design

STATEMENT OF CONTRIBUTION TO CO-AUTHORED PUBLISHED PAPER

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:

Ronalds, R. and Zhang, H., 2017. *An alternative method for on-site stormwater detention design*. Journal of Hydrology (New Zealand), 56(2), p.137.

My contribution to the paper involved the development of the research methodology, the execution of the research and the preparation of the manuscript.

(Signed) _____ (Date) 28/2/19

Rodney Ronalds

(Countersigned) _____ (Date) 06/03/19

Supervisor: Professor Hong Zhang

Overview of Chapter

A widespread approach for the protection of hydrologic conditions in regional catchments is the attenuation of peak discharges through on-site detention. It is common practice and policy for detention to be dimensioned via analysis of the catchment area including or immediately surrounding a development site, with the objective being maintenance of pre-development peak flow conditions at the development site's outlet.

The Regional Effect is a term given to the phenomenon of adverse hydrologic conditions that result from the inappropriate location of stormwater detention systems in a regional catchment. In this chapter, an analysis is presented that highlights the inadequacy of site-focussed stormwater detention design with the definition of a Regional Effect Point, at which detention located in the downstream catchment is not beneficial. It is also reasoned that unnecessarily large detention volumes can exacerbate the Regional Effect and should be avoided.

In Queensland, Australia, current guidelines recommend the use of runoff-routing models for dimensioning of on-site detention. As an alternative, some Queensland local councils provide deemed-to-comply solutions that involve basic inputs of site area and land use to calculate on-site detention. Via observations, the volumes produced by deemed-to-comply solutions greatly exceed those calculated by runoff-routing methods.

To improve material and construction efficiency, limit unnecessary land dedication, and seek to reduce the potential for the Regional Effect whilst complying with current mandates for on-site detention, an alternative detention dimensioning method is presented that does not require packaged computer software. The process can be summarised by three interrelated modules: (*i*) graphical extrapolation of the Rational Method for hydrograph approximation; (*ii*) depth-storage-discharge programming; and (*iii*) numerical runoff routing using an alternative solution to the continuity equation given by the Queensland Urban Drainage Manual (2013).

4.1 Introduction

Recent revisions of the Queensland Urban Drainage Manual (QUDM) (Queensland Government 2013) have omitted reference to four analytic equations for preliminary detention basin volume specification, i.e., Basha (1994), Boyd (1980), Carrol (1990) and Culp (1948). In lieu of these preliminary design equations, the current manual recommends that designers rely on computer models to undertake runoff routing calculations to determine the effects of urbanisation and calculate on-site detention storage requirements. The given runoff routing methods include RAFTS (Aitken 1975), RORB (Laurenson and Mein 2010), WBNM (Boyd *et al.* 1993, Boyd *et al.* 1999, Boyd *et al.* 2012) as well as the time-area runoff routing methods DRAINS/ILSAX (O'Loughlin 2014) and PC-DRAIN (Badini 2008). As the only given alternative to the usage of packaged computer models, QUDM includes one brief reference to an explicit solution to the continuity equation, without direction on its usage or application.

A number of local council governments in Queensland override QUDM by providing simple alternatives referred to as 'deemed-to-comply' solutions (Brisbane City Council 2014, Gold Coast City Council 2015). Deemed-to-comply solutions provide a means of calculating detention volume based on site area and land use only, often without quantification of runoff and consideration of the development site's location in the regional catchment. As an observation that is supported by this study, deemed-to-comply solutions typically result in an on-site detention volume that significantly exceeds that calculated by runoff-routing or the analytical equations given by earlier revisions QUDM. This suggests that deemed-to-comply solutions may contribute to wasted resources in the over allocation of building materials and land dedication for flood detention in new developments. A second concern, with wider-reaching consequences, is that flood detentions designed by deemed-to-comply solutions may be inadvertently compounding regional flooding and environmental issues via the phenomenon known as the Regional Effect.

This chapter addresses two research questions: 1) Could unnecessarily large stormwater detention storage at development sites compound the potential for the Regional Effect? and 2) Could a return to the fundamentals of stormwater detention system design result in optimized outcomes and limit the potential for the Regional Effect?

4.2 Detention assessment and design objectives

The design of a new stormwater detention system requires objectives that direct the system's performance criteria. Two primary schools of thought are used to guide the establishment of design objectives by assessment authorities: *micro-management* and *total catchment*, each with significantly different levels of assessment required.

4.2.1 Micro-management Objectives

The simplest and more historic design objective, termed the 'micro-management' method (Olenik 1999), assesses the performance of a detention system at its outlet to the downstream receiving drainage system. Using this objective, the designer relies on the assumption that any adverse impacts on the downstream hydrology are mitigated by ensuring the peak discharge at the outlet from the detention is not increased beyond the pre-development peak. Figure 4-1 provides a summary of micro-management design objectives, in which the example shows the 'site with detention runoff' curve reaching a maximum discharge that is equal to the existing case maximum discharge.

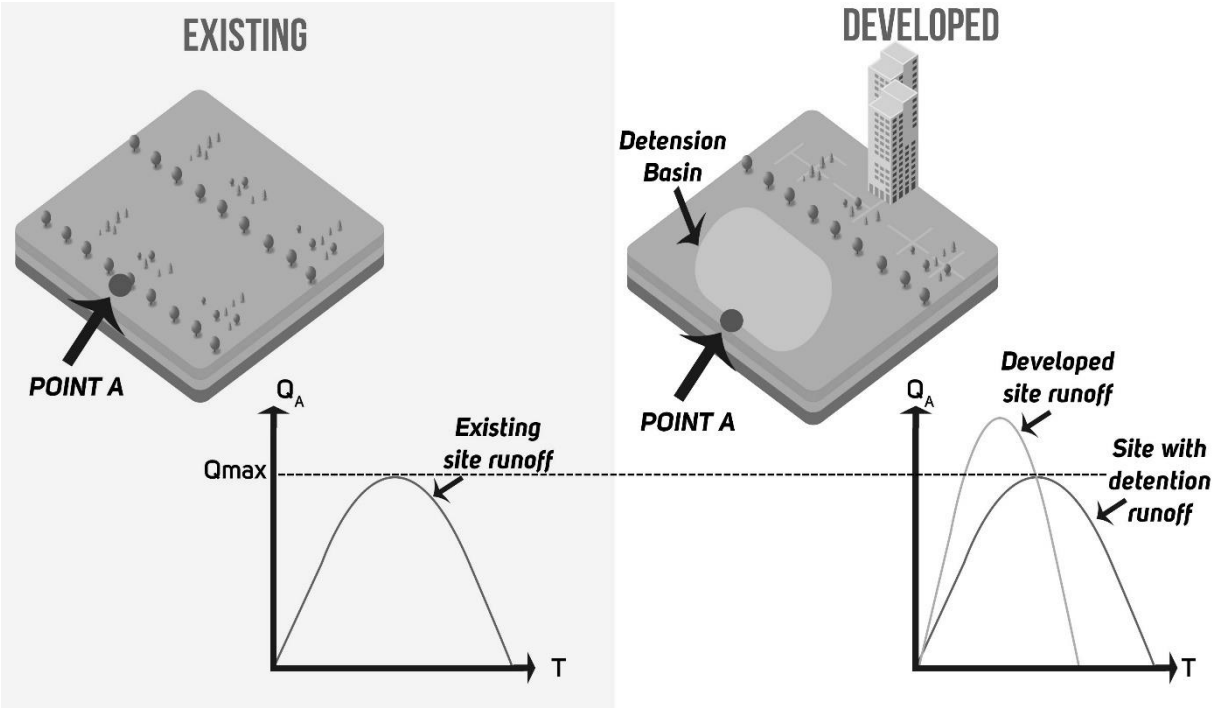

Figure 4-1 – The Micro-Management Approach to Stormwater Detention Design

From the reference point of an immediately downstream neighbour, the micro-management approach to stormwater detention is an effective strategy to avoid the adverse hydrologic impacts of urban development. For assessment authorities, micro-management policies are easy to implement, enforce and assess. There is a sense of fairness among land developers

when all new development is required to provide detention to account for its own development intensity (Debo 1982). It is also flexible for unplanned development in expanding communities, as stormwater detention can be installed in parallel with the progress of an evolving city (Shea 1996).

4.2.2 Total Catchment Objectives

The more complex alternative to micro-management is the total catchment approach. Using this method of assessment, the objective is to ensure that adverse hydrologic impacts are mitigated at all locations within the catchment. Design objectives that adopt this approach recognise the complexity of regional catchment hydrology, specifically the effect of hydrograph peak timing. Figure 4-2 provides a diagram of the total catchment approach, in which the subject site discharge (point A) is detained to achieve the required result at the total catchment outlet (point B).

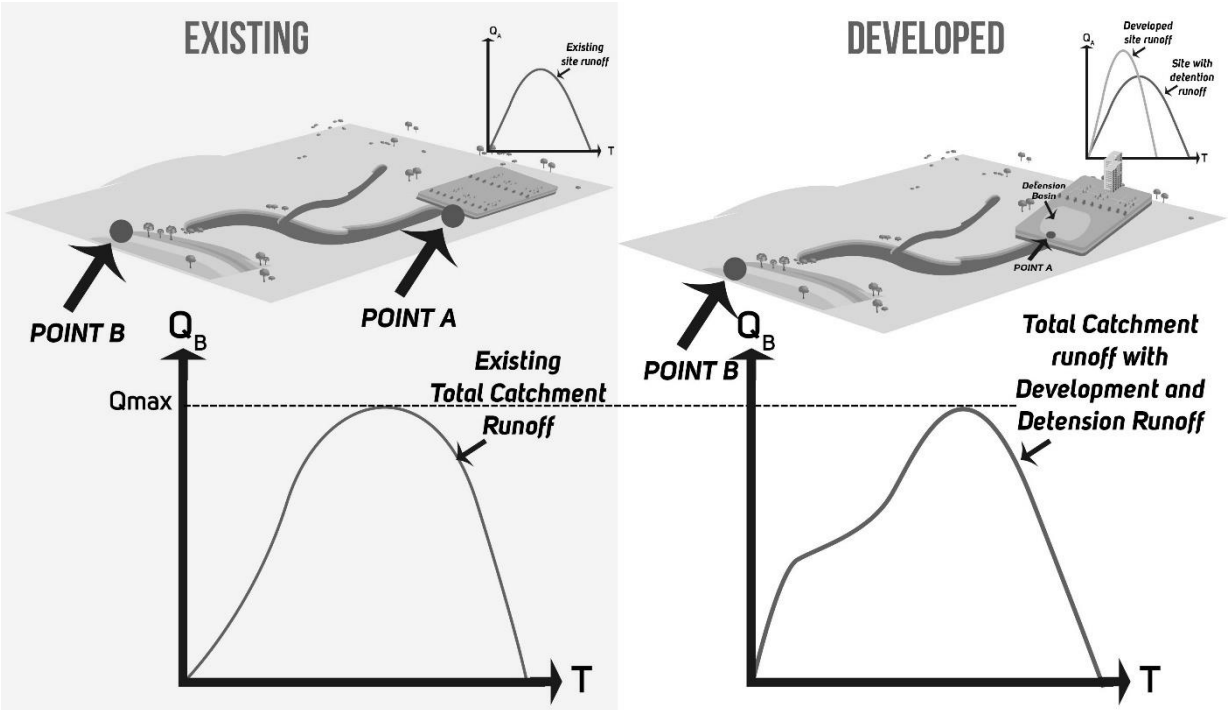

Figure 4-2– The Total Catchment Approach to Stormwater Detention Design

Within the total catchment approach to selecting design objectives for detention systems, there are two sub-categories:

- At-source strategies (Argue 2004), which require the installation of detention systems at each new development site location individually, independently designed to ensure adverse hydrologic impacts resulting from the associated development are mitigated.

- Distributed detention strategies (Travis and Mays 2008), which require the identification of specific and optimised locations for the installation of detention throughout the regional catchment, designed as a network.

At-source detention design methods for total catchment objectives are onerous for individual development projects, and are therefore rarely required.

Ideally, for successful flood control, detention systems should be considered as a network, rather than individually. Studies in Asia (Tao *et al.* 2014, Duan *et al.* 2016), Europe (Bellu *et al.* 2016, Ravazzani *et al.* 2014) and the Americas (Kaini *et al.* 2007, Shuster and Rhea 2013, Su *et al.* 2010) have developed effective models and algorithms for optimized distributed detention locations within the regional catchment. In addition to reduced flood risk, regional detention strategies are known to be more efficient and result in as much as 41% less detention storage over the whole catchment (McEnery and Morris 2012).

Whilst distributed detention is strongly recommended and not debated by this research, major constraints exist with implementation of the theory. Firstly, distributed detention needs to be carefully planned for at a regional scale, often requiring participation by multiple municipalities or even nations. Secondly, distributed detention requires significant land dedication, often at very specific locations that will not achieve the desired outcomes if spatially negotiated. These factors make distributed detention design and implementation a complicated act that is infrequently adopted.

4.3 The Regional Effect

McCuen (1974) raised concerns regarding the location of stormwater detention in a regional context, highlighting the fact that the inappropriate location of detention structures can have outcomes that conflict with their intentions. McCuen (1979) further stated that the flow timing change caused by a detention basin can inadvertently result in increased downstream flooding in some applications.

Known as the “Regional Effect” (Flores *et al.* 1982, Leise 1991, Goff and Gentry 2006, Seybert 2006, Saunders 2008), the notion that inappropriately allocated stormwater detention systems can cause one hydrograph peak to lag or be extended to such a time that it causes a coincidence with another hydrograph peak is generally well acknowledged. Figure 4-3 shows the Regional Effect graphically, with an elongated detention hydrograph contributing to the peak of the regional hydrograph.

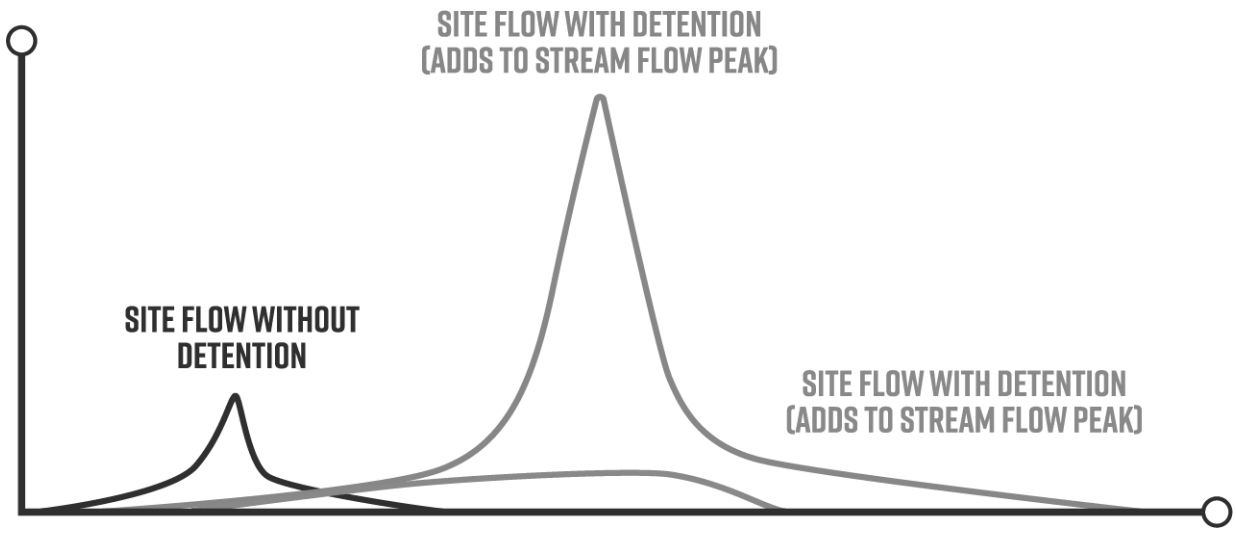

Figure 4-3 – Hydrograph Description of the "Regional Effect"

Where stormwater detention design policies include micro-management objectives, there is the potential for unnecessarily large stormwater detention storage to have adverse effects, as well as being a waste of resources. The Regional Effect theory suggests that an unnecessarily large stormwater detention storage at certain locations in a catchment, can potentially exacerbate flooding and environmental damage in downstream watercourses.

A broad and generalised application of Regional Effect theory is the axiom that stormwater detention is generally not applicable to the lower third of a catchment. This concept can be encountered in some South East Queensland stormwater planning guidelines (Queensland Government 2013, Brisbane City Council 2014); however, it is usually overruled by direct mandates for stormwater detention to achieve micro-management objectives.

4.4 Case study – South East Queensland

South East Queensland, Australia has experienced major regional flooding in recent years, with most flood damage occurring in the lower portions of regional catchments where the major cities are located, including the state's two largest cities, Brisbane and Gold Coast. Brisbane City and the Central Gold Coast are both located within the lowest 10% of the regional catchments (see Figure 4-4).

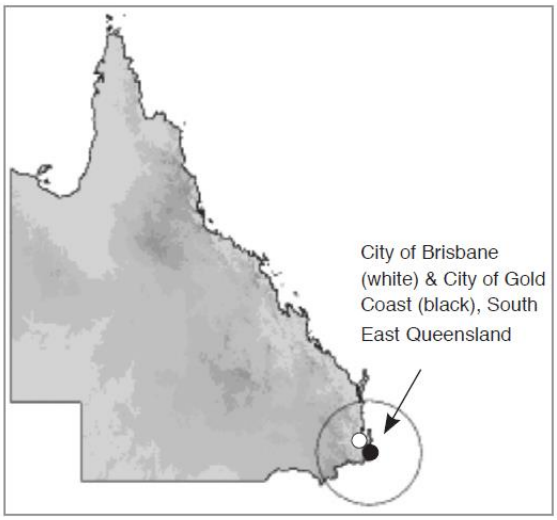

Figure 4-4 – Case Study Locality (South East Queensland)

South East Queensland has a subtropical climate with high rainfall intensities and rainfall concentrated in the summer months. Notable floods resulting in major damage and deaths in Queensland include the January 1974 and January 2011 events. The most recent resulted in more than one million square kilometres of flooding and an approximate account of damages and losses at $15.7 billion (World Bank 2011).

In South East Queensland, the micro-management approach to stormwater detention objectives is mandated by current development codes, exemplified by the Gold Coast Healthy Waters Code (2015): "*on site detention systems are designed to restrict peak outflows for* Q_2 Q_5, Q_{10}, Q_{20}, Q_{50} *and* Q_{100} *to pre-development conditions*" to achieve the performance outcome of "*demonstrat[ing] no adverse impact on stormwater flooding or the drainage of properties external to the subject site".*

4.5 Confirmation of the Regional Effect

In this study, South East Queensland catchments experiencing urban development are used to investigate the Regional Effect in practice and consider the potential impact of over-allocation of stormwater detention.

4.5.1 Regional Effect Model Set-up

A process of numerical experimentation was undertaken in which a development site was conceptualized at ten alternative locations within a regional catchment, each at 10% increments from the outlet of the regional catchment. At each of the ten locations the development site was modelled with three development scenarios: undeveloped, developed without detention, and developed with detention. The detention was dimensioned using

WBNM in-built methods with micro-management objectives, i.e., ensuring that the development site discharge is limited to pre-developed peak runoff. The modelled peak discharges at the outlet of the regional catchment were compared for the different development scenarios. The hydrologic parameters of the model set-up are presented in Table 4-1.

Table 4-1 – Hydrologic Parameters Used in the WBNM Model for Testing the Regional Effect

Australian Rainfall and Runoff 1987 Rainfall Design Rainfall Temporal Pattern	3
Rainfall station location	-27.9667,153.4167
Mean annual rainfall	1200 mm
WBNM lag parameter	1.6
Initial loss	15 mm
Continuing loss	2 mm
Existing case imperviousness	20%
Development site imperviousness	75%

The location of the rainfall station is within the central Gold Coast area, where temporal patterns of rainfall are typical of the Eastern Australian coastal areas. Hypothetical urban development sites with areas of 1ha and 10ha were considered, which are representative of typical commercial projects such as shopping centres, and residential 50-100 allotment subdivisions, respectively.

4.5.2 Finding the Regional Effect Point

The intention of the analysis was to locate the “Regional Effect Point”, defining it by the distance at which the development site is located upstream of the regional catchment outlet, as described by Figure 4-5. Upstream of this location, development with detention designed using micro-management objectives have the desired impact by reducing peak discharge at the regional catchment outlet. Developments with detention downstream of the Regional Effect Point have the opposite effect, resulting in an increase in peak discharge at the regional catchment outlet.

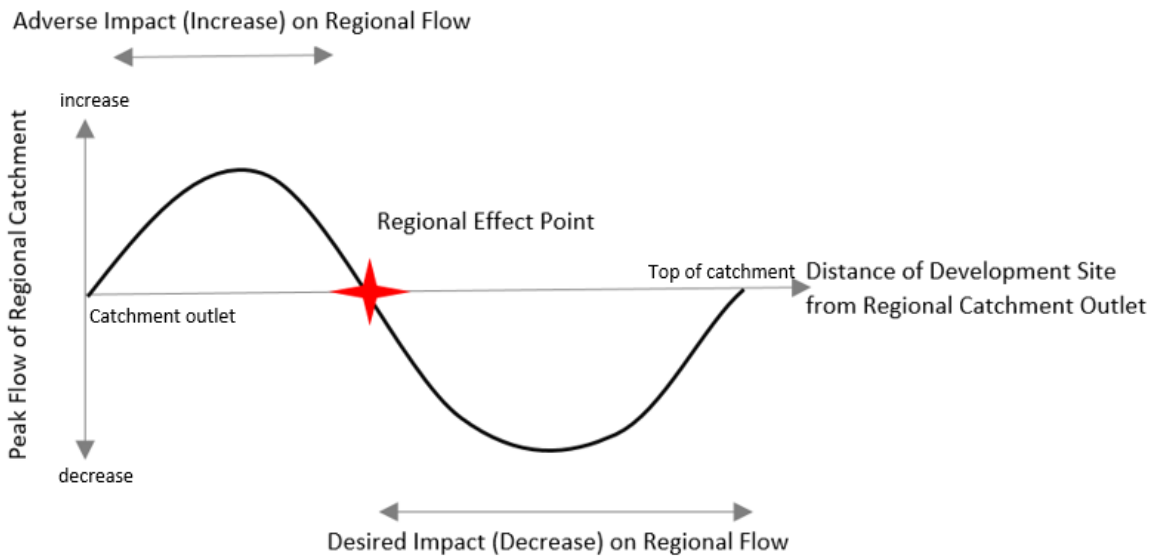

Figure 4-5 – Regional Effect Point

The analysis was performed on six regional catchments. In total, 11,858 runoff routing analyses were performed, accounting for the three scenarios of development, all standard average recurrence interval storm events (1, 2, 5, 10, 20, 50 and 100 years), all standard durations within range of the critical storm (15 minutes up to 1,480 minutes), and ten times for each regional catchment to assess for the shifting locations of the development site. Table 4-2 provides a summary of the catchments and the Regional Effect Point analysis results.

Table 4-2 – Summary of catchments and results of Regional Effect Point analysis

	Regional catchment size	Development site size	Location of Regional Effect Point (% upstream of catchment outlet)
Regional Effect Model Scenario 1	100 ha	1 ha	Does not exist
Regional Effect Model Scenario 2	100 ha	10 ha	Does not exist
Regional Effect Model Scenario 3	1,000 ha	1 ha	26%
Regional Effect Model Scenario 4	1,000 ha	10 ha	21%
Regional Effect Model Scenario 5	10,000 ha	1 ha	46%
Regional Effect Model Scenario 6	10,000 ha	10 ha	32%

The results suggest there should not be a rule for excluding detention from any fixed portion of a regional catchment without further analysis. The results also indicate a pattern regarding the size of the development site relative to the regional catchment. As described graphically in Figure 4-6, for development sites that are large compared to the regional catchment, the

results indicate that there is no Regional Effect Point, meaning that micro-management policies are effective in protecting the hydrology of the regional catchment. As the development site becomes smaller compared to the regional catchment, the Regional Effect Point appears and moves higher up the catchment.

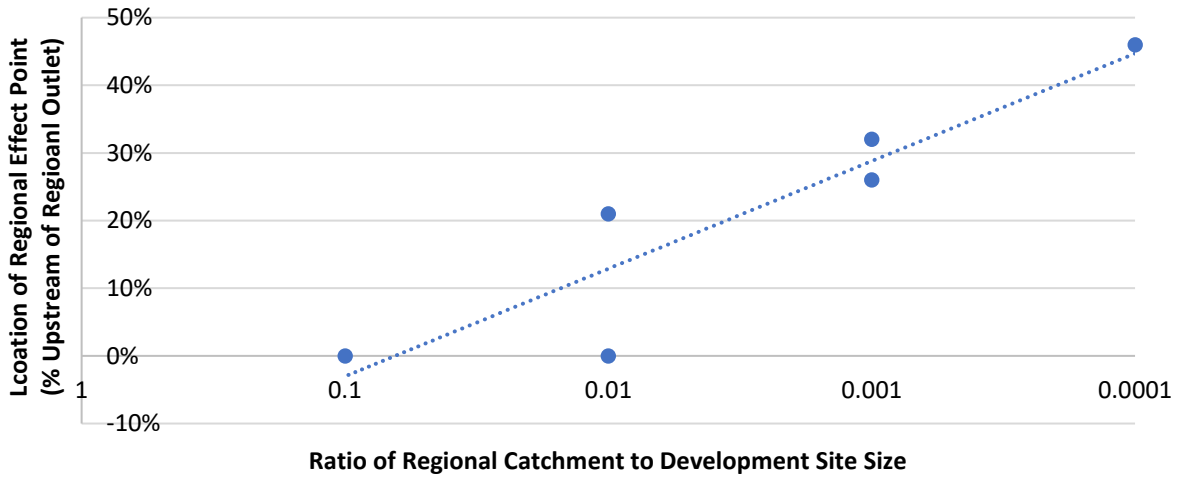

Figure 4-6 – Location of Regional Effect Point Relative to Catchment Size, Based on the Results From Six Catchments Used in This Case Study

4.5.3 Detention Versus No Detention

Table 4-2 and Figure 4-6 suggest that a 1 ha development site within a 10,000 ha catchment (scenario 5) can have the potential to increase regional catchment peak runoff when development with detention occurs anywhere in the lowest 46% of the catchment. Figure 4-7 shows the addition of the results for development occurring without any form of detention and indicates an important finding in the lowest 15% of the catchment. In this zone, development with detention results in increased peak runoff in the regional catchment, not only above the undeveloped scenario, but also above the scenario of development without detention. This is typical of Scenarios 3, 4 and 6 as well.

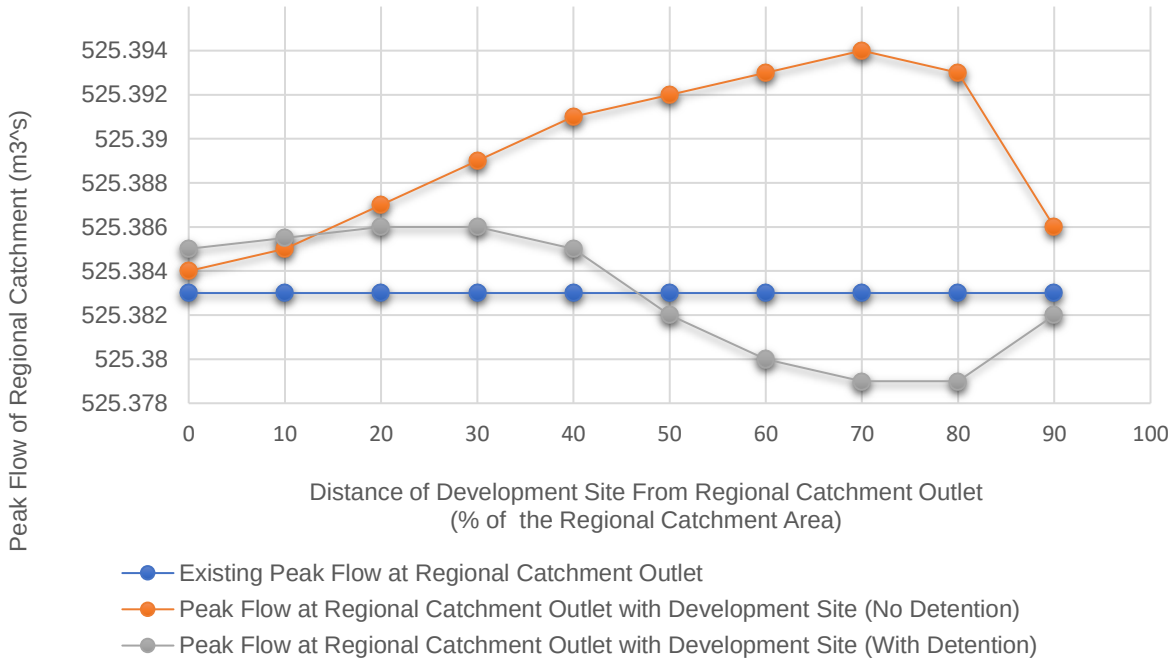

Figure 4-7 – Regional Effect Model Scenario 5 Results

Further investigation was undertaken at the location along the main regional stream line 10% upstream of the regional catchment outlet. The detention volume calculated using WBNM (260 m^3) was replaced with the deemed-to-comply calculation (320 m^3) and the results were exacerbated, showing that at this location detention is not only worse for the regional hydrology than inaction, but also that over-allocation of detention at this location can lead to adverse hydrologic conditions in terms of increased peak flow at the catchment outlet.

4.6 Optimised detention design method development

A method for detention dimensioning was developed that is recommended for consideration by practitioners charged with the responsibility of designing stormwater drainage and detention systems for small development sites, where the Rational Method is considered to be a suitable means of stormwater runoff determination.

4.6.1 Inflow Hydrograph Approximation

Peak flows for design storm events can be estimated simply using the Rational Method (Equation 4-1) (Mulvaney 1851, Kuichling 1889), one of the oldest and most widely-adopted approaches. The inputs include site area (*A*), rainfall intensity (*I*), and a Runoff

Coefficient (*C)* that accounts for aerial and groundwater infiltration losses. The subscripts $_y$ and $_t$ relate to storm event recurrence interval and storm duration, respectively.

$$Q_{y,peak} = \frac{C_y I_{y,t} A}{360} \qquad 4\text{-}1$$

The hydrologic assessment and design of detention systems requires review of both critical and non-critical storm durations (t_d), where a critical storm duration is that which matches the time of concentration of the catchment. To allow for this assessment, graphical extrapolation of the Rational Method results is suggested, adopting the triangular or trapezoidal method as relevant.

Graphical representation of the triangular method (Abt and Grigg 1978, Donahue *et al.* 1981, Chow 1988, Hong *et al.* 2006, Hong 2008) is shown in Figure 4-8. Equation 4-2 is recommended for using the triangular extrapolation of the Rational Method for determining an inflow hydrograph for a storm event.

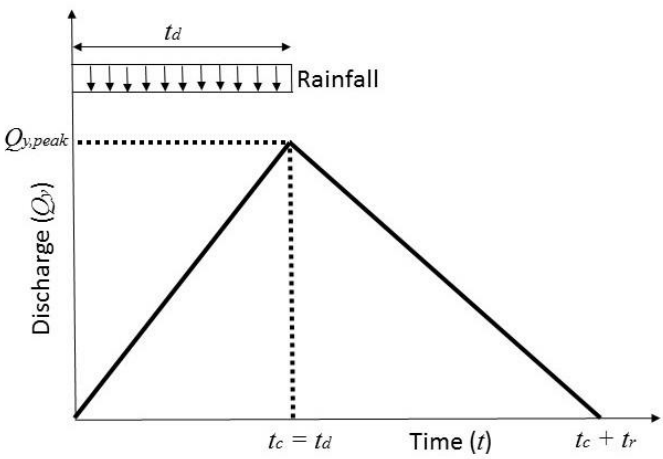

Figure 4-8 – Triangular Method for Storm Hydrograph Estimation

$$Q_y(t) = \begin{cases} C_y I_y A \frac{t}{t_c} \langle for\ t \leq t_c \rangle \\ C_y I_y A \left(\frac{t_r + t_c - t}{t_r}\right) \langle for\ t > t_c \rangle \end{cases} \qquad 4\text{-}2$$

Based on a study of 500 catchments by Hong *et al.* (2006), the time from the storm peak to the end of the stormflow portion of the hydrograph (t_r) is recommended as $1.76t_c$, where t_c is the catchment time of concentration.

An alternative is the trapezoidal method (Burton 1980, Guo 1999, Hong *et al.* 2006), which is recommended for non-critical storm events, where storm duration exceeds the catchment time of concentration. Graphical representation of the trapezoidal inflow hydrograph approximation is provided in Figure 4-9. Equation 4-3 is recommended for using the

trapezoidal extrapolation of the Rational Method for determining an inflow hydrograph for a storm event.

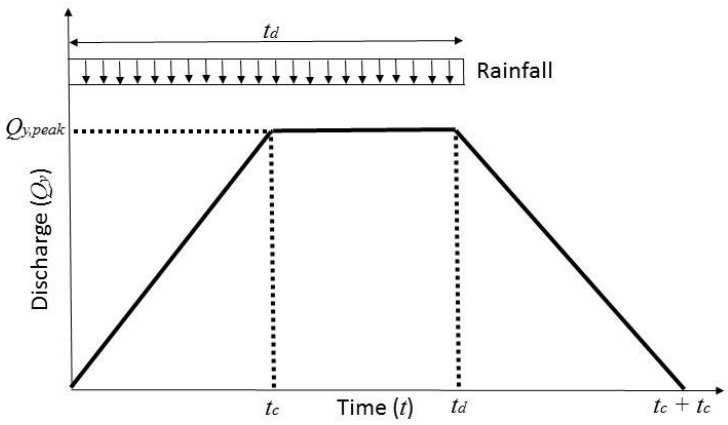

Figure 4-9 – Trapezoidal Method for Hydrograph Estimation

$$Q_y(t) = \begin{cases} C_y I_y A \frac{t}{t_c} \langle for\ t < t_c \rangle \\ C_y I_y A \langle for\ t_c \ \leq t \leq t_d \rangle \\ C_y I_y A - \frac{C_y I_y A}{t_d}(t - t_c) \langle for\ t > t_d \rangle \end{cases} \qquad 4\text{-}3$$

4.6.2 Detention Design Parameters (Height-Storage-Discharge Function)

4.6.2.1 Detention tank volume

An idealized detention volume can be used for typical small scale urban development application that considers rectangular tanks with vertical walls. The detention storage volume follows the form:

$$S = H \times B \qquad 4\text{-}4$$

where *B* is the detention base area, *S* is the detention storage volume, and *H* is the maximum detention height.

4.6.2.2 Piped outflow - low stage piped outlet to atmospheric conditions

In situations where a low flow outlet pipe is not under pressurized conditions (i.e., the detention water depth is lower than the outlet pipe diameter and a tail water condition is not specified for the downstream outlet), the Manning equation (Chow 1959) can be applied to solve for piped flow discharge:

$$Q_{dis} \ = \frac{1}{n} A_w (\frac{A_w}{P})^{\frac{2}{3}} S_g^{\frac{1}{2}} \qquad 4\text{-}5$$

where Q_{dis} is the capacity of the outflow pipe, n is the Manning coefficient, A_w is the wetted area of pipe flow, P is the wetted perimeter of pipe flow and S_g is the slope (gradient) of the outlet pipe.

4.6.2.3 Piped outflow - pressurized outlet flow conditions

For conditions where the detention depth is either higher than the outlet pipe diameter or a tail water condition is specified for the downstream outlet, the outlet pipe arrangement is considered to flow under pressure. Pressurised flow through the pipe can be calculated using the head loss form of the Darcy–Weisbach equation (Brown 2002):

$$\frac{\Delta h}{L} = f_D \frac{1}{2g} \frac{V^2}{D} \qquad 4\text{-}6$$

$$Q_{dis} = A_p \left[\sqrt{\left(\frac{2gD(h_{tank} - h_{DS})}{L f_D} \right)} \right] \qquad 4\text{-}7$$

$$Where: \frac{1}{\sqrt{f_D}} = -2\, log\left(\frac{5.76}{Re^{0.9}}\right) \quad and \quad R_e = \frac{\rho}{\mu} VD \qquad 4\text{-}8$$

where L is the length of the conduit, f_D is the Darcy Friction Factor, V is the pipe velocity, D is the pipe diameter and g is the gravitational constant. The parameter h refers to water surface elevation and the subscripts $_{tank}$ and $_{DS}$ refer to the water level inside the detention system and downstream of the detention outlet, respectively.

As a generalisation, flow through the outlet pipe of a detention system is considered "turbulent", and in the range of the "smooth pipe law" (Colebrook *et al.* 1939) for detention design applications. Fluid dynamic viscosity (μ) may be assumed as 0.9×10^{-3} kg/(m.s) and fluid density (ρ) may be assumed as 1×10^3 kg/m^3. Using these values, experimental results that reflect typical detention outlet conditions produce Reynolds number (R_e) results in the order of 5000-8000.

4.6.2.4 Weir flow

At the higher stages of the detention outlet a weir may be specified to account for high level bypass or overflow. Weir flow capacity is calculated as depth above the weir level as follows (Queensland Government 2013):

$$Q_{weir\ cap} = C_w L_w h_{weir}^{\frac{3}{2}} \qquad 4\text{-}9$$

where C_w is the weir coefficient (1.66 for sharp crested), L_w is the weir length, and h_{weir} is the water depth above the weir overflow.

4.6.2.5 Combined Height-Storage-Discharge function for detention basin (rating curve)

The design dimensions and outlet options can be combined to generate a rating curve for detention routing that follows the overall form:

$$Q_h = \begin{cases} \frac{1}{n} A_w \left(\frac{A_w}{P}\right)^{\frac{2}{3}} S_g^{\frac{1}{2}} + C_w L_w h_{weir}^{\frac{3}{2}} \langle for\ h \leq D \rangle \\ A_p \left[\sqrt{\left(\frac{2gD(h_{tank} - h_{DS})}{Lf_D}\right)}\right] + C_w L_w h_{weir}^{\frac{3}{2}} \langle for\ h > D \rangle \end{cases} \quad 4\text{-}10$$

where Q_h is the staged storage outflow and h is the incremental depth of the detention system. The peak outflow of the basin is limited to the maximum basin storage by the form:

$$\lim_{h \to H} f(h, D, L, f_D, h_{weir}) = Q_h \quad 4\text{-}11$$

4.6.3 Runoff Routing (Continuity Equation)

The final process of the proposed detention design procedure involves the routing of the extrapolated Rational Method inflow hydrograph through the detention system to produce a modified, "detained" hydrograph output. For this application, the inflow hydrograph (*I*) is routed though the design detention storage (*S*) using an explicit solution of the continuity equation to create an outflow hydrograph (*Q*):

$$\frac{dS}{dt} = I(t) - Q(t) \quad 4\text{-}12$$

4.6.3.1 Explicit solution to the continuity equation

Integration of the continuity equation to provide for an iterative numerical solution is performed as follows:

$$\int \frac{dS}{dt} dt = \int I(t)\ dt - \int Q(t)\ dt \quad 4\text{-}13$$

With a known incoming hydrograph and a known height-storage-discharge function for the design detention system, the continuity equation can be rearranged to make all known factors of an incoming time step equal to the unknown factors of the outlet discharge as follows:

$$\frac{2(S_{i+1} - S_i)}{\Delta t} = (I_{i+1} + I_i) - (Q_{i+1} + Q_i) \quad 4\text{-}14$$

$$(I_i + I_{i+1}) + \left(\frac{2S_i}{\Delta t} - Q_i\right) = \left(\frac{2S_{i+1}}{\Delta t} + Q_{i+1}\right) \quad 4\text{-}15$$

where the subscripts $_i$ and $_{i+1}$ represent positively-ascending time step intervals.

A variation of Equation 4-15 is provided in QUDM (2013) and recommended as an acceptable alternative to the use of a computer model for the final sizing of a detention system, but without any practical advice regarding runoff routing or usage of the equation. An alternative, more practical rearrangement of the equation is:

$$\frac{I_i + I_{i+1}}{2} + \left(\frac{S_i}{\Delta t} + \frac{Q_i}{2}\right) - Q_i = \left(\frac{S_{i+1}}{\Delta t} + \frac{Q_{i+1}}{2}\right) \qquad 4\text{-}16$$

The left-hand side of Equation 4-16 can be solved at each time step based on the previous time step results, starting with an initial inflow and storage equal to zero. The solution to the left-hand side of Equation 4-16 is used at each routing time step to explicitly reference the design rating curve developed by Equation 4-10 to extract the storage and corresponding outflow for the following time step. Solving for storage as a whole number, as opposed to Equation 4-15, gives the designer better appreciation of the detention storage volume as well as the ability to more easily iterate and optimize the detention design, ensuring optimal storage usage and limited superfluous volumes. A graphical presentation of the solution to the continuity equation applied to detention basin design is provided in Figure 4-10.

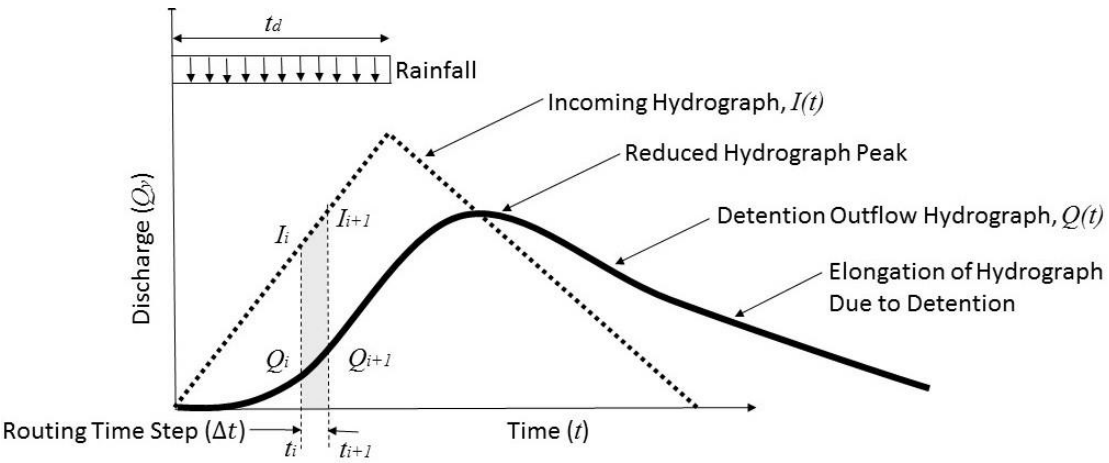

Figure 4-10 – Graphical Summary of Explicit Solution to the Continuity Equation

4.7 Validation and Comparison of Detention Design Method

To validate the hydraulic calculations presented in the proposed design methodology, hydrograph outputs from runoff routing software (WBNM) were used to overwrite the Rational Method extrapolation input. Eight hypothetical South East Queensland locality sites between 0.05 ha and 1 ha in area were considered with the hydrologic parameters provided in Table 4-3. The results indicated precision in hydraulic calculations with a variation of ±5% for detention volume and ±5% for detention water depth, where the

WBNM model outputs are considered as the base and the methodology presented in this study (referred to as the *Ronalds and Zhang method*) is considered to be the variation.

Table 4-3 – Hydrologic parameters used for validation of Ronalds and Zhang method

	Pre-Developed sites	**Developed sites**
Imperviousness	0%	90%
Rational Method C10 value	0.60	0.90
WBNM lag parameter	1.6	
WBNM initial loss	15 mm	
WBNM continuing loss	2 mm	
ARR1987 zone	3	
Rainfall station location	-27.9667,153.4167	
Mean annual rainfall	1200 mm	

4.7.1 Comparison with Other Accepted Methods

Figure 4-11 shows the results of a comparison on detention dimensioning methods using South East Queensland rainfall data and eight hypothetical development sites (catchments), ranging in size from 500 m^2 to 10,000 m^2. For each, the Ronalds and Zhang method was used to size the hypothetical detention systems in accordance with the recommendations of this study. The results are compared to deemed-to-comply solutions for detention volume determination (Brisbane City Council 2014), sizing of detention systems using WBNM, as well as the four analytic equations for preliminary sizing of detention that were published in the 2008 edition QUDM but removed from the provisional 2013 edition, i.e., Basha (1994), Boyd (1989), Carrol (1990) and Culp (1948).

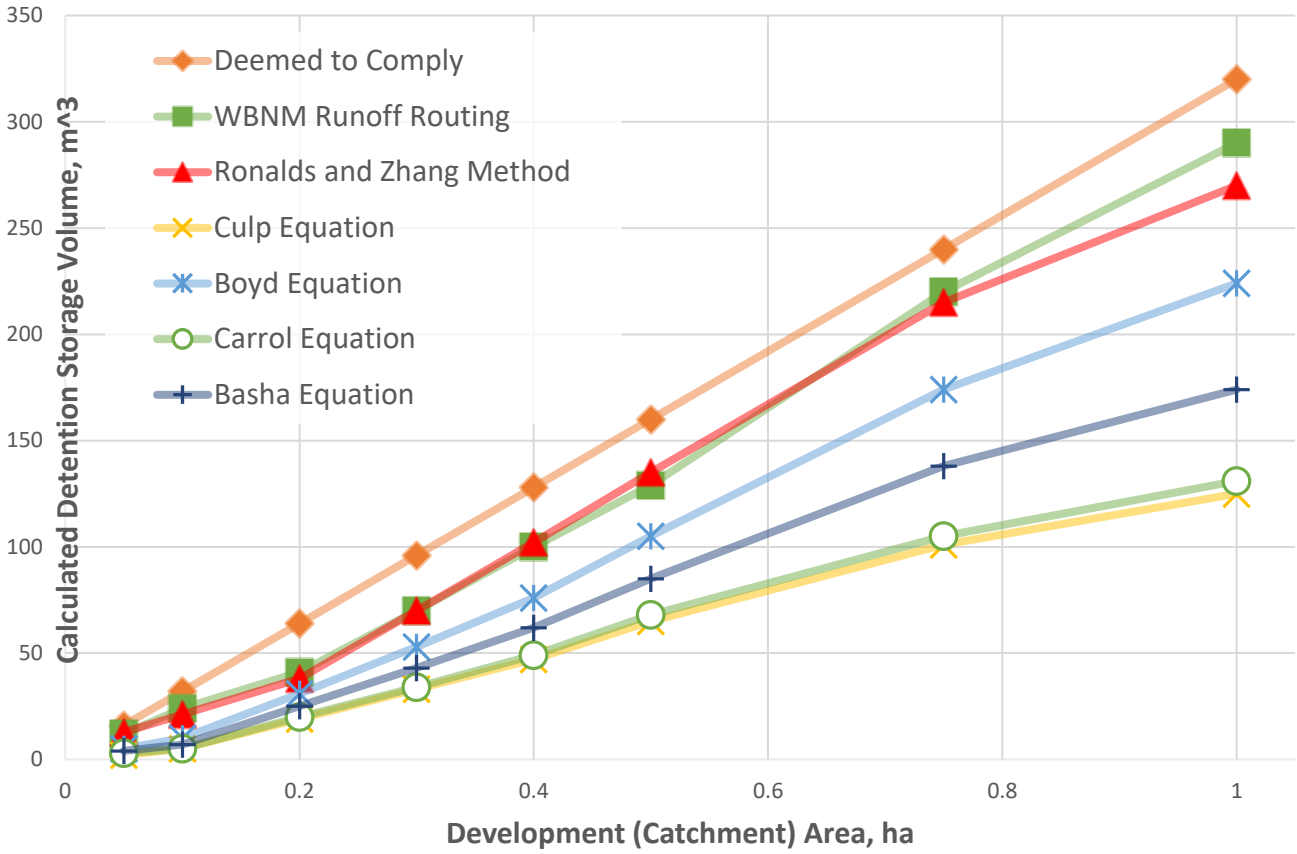

Figure 4-11 – Comparison of Stormwater Detention Storage Volume Results Derived from Various Methods

The results presented in Figure 4-11 demonstrate three key findings: 1) the proposed methodology described by this study achieves detention dimensioning outcomes that closely reflect the WBNM runoff routing model, 2) the preliminary analytical equations significantly underestimate detention requirements, justifying their omission from recent editions of QUDM, and 3) the deemed-to-comply solutions provide results that are larger compared to the other methods, justifying the concerns and recommendations presented in this chapter.

4.8 Discussion

Parameterisation of inflow hydrographs using the triangular or trapezoidal methods is simplistic in application, without regard for temporal patterns. It is known that a rainfall hyetograph does not follow the perfect pattern of a triangle or trapezoid; however, it is also known that rainfall may not consistently follow any one predictable pattern. Australian Rainfall and Runoff (Ball *et al.* 2016) recommends using an ensemble or a Monte Carlo analysis of rainfall patterns. For regional catchment analysis, the concept of multiple

temporal patterns may be justified as it provides a more realistic representation of natural flood generation processes, considering the influence of all probability distributed inputs (Nathan and Weinmann 2013). However, for the design of a development site-scale stormwater detention system, where all surrounding pipework is dimensioned using a static Rational Method calculation, the triangular and trapezoidal methods are considered to be as reasonable as any other single temporal pattern.

The recommendation for triangular or trapezoidal extrapolation is limited to the size of development sites included in this study. The durations of storm events resulting in high peak discharges for the catchments assessed are typically short and their Australian Rainfall and Runoff (ARR1987) temporal patterns result in fast, front- loaded hydrographs that reasonably reflect the triangular extrapolation method shape. For larger catchments, the effect of temporal pattern diversity becomes too significant to recommend triangular or trapezoidal extrapolation.

The analysis of the Regional Effect Point provided is simplistic and undertaken only to justify the need for improvement upon deemed-to-comply solutions. It does not consider a suitable range of catchment variables and alternatives to confirm a method for determining the location of Regional Effect Point for unrelated catchments. Further work to confirm or provide means to calculate the location of the Regional Effect Point should use ensemble or a Monte Carlo analysis for the regional catchment modelling, along with a wider range of catchment variables.

The hydraulic equations for detention outflow provided by the Ronalds and Zhang method are theoretical without experimentation to confirm applicability to local conditions or typical on-site detention dimensions. Minor differences exist in the programming of hydraulic outflow conditions in each of the computer models recommended by QUDM. Recent experimental developments of detention outflow equations by Hong (2010) are not considered by any of the current computer models. An experimental review of hydraulic outflow equations could therefore lead to improved accuracy and confidence in the Ronalds and Zhang method.

4.9 Summary

This chapter has shown that unnecessarily large stormwater detention storage at development sites located within certain portions of a regional catchment can compound the potential for the Regional Effect. The definition of a Regional Effect Point has been used to

describe a location in the regional catchment at which on-site detention in any areas downstream will not achieve the mitigation of regional peak flow increase, and that unnecessarily large stormwater detention can be worse for regional conditions than optimally sized systems.

A simplistic method for estimating the required size of stormwater detention using fundamental methods has been presented. Referred to as the Ronalds and Zhang method, it is suggested that the alternative method can be used to more accurately calculate the required volumes of stormwater detention, in lieu of methods such as deemed-to-comply solutions that have been shown to over-estimate the required volumes.

This research has not provided a means to avoid all potential for Regional Effect. However, it has been shown that unnecessarily large stormwater detention storage detention does not do this either. In an ideal world with unlimited time and resources available to designers, the potential for Regional Effect should be assessed as part of the design process for every on-site stormwater detention system. As a practical and feasible alternative, stormwater detention volumes should be carefully calculated using fundamental hydrologic and hydraulic analysis to avoid unnecessarily large stormwater detention storage, improving construction material efficiency, land use efficiency and reducing the potential for Regional Effect.

5. On-Site stormwater detention for Australian development projects: Does it meet frequent flow management objectives?

STATEMENT OF CONTRIBUTION TO CO-AUTHORED PUBLISHED PAPER

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:

Ronalds, R., Rowlands, A and Zhang, H., 2019. *On-Site stormwater detention for Australian development projects: Does it meet frequent flow management objectives?*. Water Science and Engineering, 12(1), pp.1-10

My contribution to the paper involved the development of the research methodology, the execution of the research and the preparation of the manuscript.

(Signed) _____ (Date) 28/2/19

Rodney Ronalds

(Countersigned) _____ (Date) 06/03/19

Supervisor: Professor Hong Zhang

Overview of Chapter

On-site stormwater detention (OSD) is a conventional component of urban drainage systems, designed with the intention of mitigating increases to peak discharge of stormwater runoff that is the inevitable result of urbanisation. In Australia, singular temporal patterns for design storms have governed the inputs of hydrograph generation and in turn the design process of OSD for the last three decades. This study raises the concern that many existing OSD systems designed using the singular temporal pattern process may not be achieving their stated objectives when they are assessed against a variety of alternative temporal patterns. The performance of twenty real OSD systems is investigated using two methods: 1) new ensembles of design temporal patterns prescribed by the latest version of the Australian Rainfall and Runoff guideline, and 2) real recorded rainfall data taken from pluviograph stations modelled using continuous simulation. It is shown conclusively that usage of singular temporal patterns is ineffective in providing assurance that an OSD will mitigate increases to peak discharge for all possible storm events. Ensemble analysis is shown to provide improved results, however it too falls short of providing any guarantee in the face of real patterns of naturally occurring rainfall.

5.1 Introduction

5.1.1 Stormwater Detention in Australian Catchments

Australia is a country known for extreme weather conditions, including high-intensity rainfall and flash flooding. Due to lifestyle, economic, and logistic factors, Australia is experiencing significant recent and planned urbanisation in the coastal or riverine areas that are subject to a high risk of flood inundation. This presents a concern for engineers and planners, as development within and surrounding the floodplain has the known potential to result in changes to flood behaviour, including flow path locations, flow rates, velocities, flood levels, flood extents, channel erosion, and duration of flooding (Poff et al. 1997).

There have been several attempts by researchers and authorities to mitigate the impact that urbanisation can have on catchment hydrology. Effectively reducing the imperviousness of a developed catchment has been promoted as one of the potential solutions (Booth and Jackson 1997, Horner *at al.* 1999, Hatt *et al.* 2004, Taylor 2005, Walsh *et al.* 2005). However, reducing the imperviousness in high-density urban development to a degree to achieve regional hydrologic neutrality is not always considered to be a feasible and effective solution. The concept of stormwater volume control has prevailed as one of the preferred means to manage the effect of urbanisation on catchments and the downstream impacts associated with urban development (Phillips and Canterford 1987, Phillips *et al.* 2016).

One of the most common forms of stormwater volume management adopted in engineering practice is on-site stormwater detention (OSD), involving the use of tanks or basins at a development-site scale. These systems are designed using hydrograph theory to delay and reduce the peak discharge from a development site to a desired level, typically to match the peak discharge under pre-development conditions.

The use of OSD originates from American studies (Poertner 1976) that were adapted to Australian conditions (Phillips 1987). The first time for local governments in Australian councils to adopt OSD policies was in Sydney in the early 1980s (O'Loughlin *et al*. 1995). In South East Queensland, OSD is currently required to restrict peak discharges outflows for 39.35%, 18.13%, 10%, 5%, and 1% annual exceedance probability (AEP) events to pre-development conditions, in order to achieve the performance outcome of "no adverse impacts on stormwater flooding or the drainage of properties external to the subject site" (City of Gold Coast 2018).

5.1.2 Peak Discharge Management Objectives for Frequent Storm Events

OSD is used for stormwater volume management to achieve two primary objectives: controlling discharge in frequent events and mitigating flood in rare events. Achieving both of these objectives with one OSD system is a challenge and requires the specification of a storage volume and outlet configuration that is effective for all possible storm events within the range of design event probabilities (Phillips 1995).

This study is focused on frequent flow management objectives, which require the use of OSD to reduce the discharge of development-site runoff during storm events with a frequent probability of recurrence, between the AEP range of 39.35% and 10%. The theory supporting these objectives relates to the stability of watercourse embankments and the need for impervious developments to avoid changes to the frequency, depth, and velocity of runoff (Bledsoe 2002, Shuster *et al.* 2005, Hawley *et al.* 2017, Vietz and Hawley 2019).

Increases in peak stormwater discharge are not the only outcome of urbanisation that has detrimental effects. Increases in the volume of runoff will also contribute to elongated hydrographs with extended time periods of bankfull flow and increased probability of coincident hydrograph peak in downstream waterways. OSD with peak flow design objectives does not address these issues, which is the reason for recent developments in distributed detention as part of a catchment-wide network (Kaini *et al.* 2007, Travis and Mays 2008, Ravazzani *et al.* 2014, Tao *et al.* 2014, Duan *et al.* 2016, Bellu *et al.* 2016, Shuster and Rhea 2013, Su *et al.* 2010) and research into the phenomenon known as the "regional effect" that results from multiple detention systems in series (McCuen 1974, Bennett and Mays 1985, Ferguson and Deak 1994, Saunders 2006, Ronalds and Zhang, 2017). This study is limited to peak discharge management objectives for frequent storm events, in accordance with current practice in the case study area.

5.1.3 An Era of Over-Reliance on Determinism

For the last three decades, the development of design event hydrographs in Australia has followed a prescriptive procedure, largely directed by the seminal works of Pilgrim and Canterford (1987) in the guide of Australian Rainfall and Runoff, referred to below as ARR1987. For each storm event, the process has resulted in a deterministic outcome: a singular hydrograph for a certain probability and duration. Following the average variability method (Pilgrim and Cordery 1975), rainfall depth inputs are obtained from intensity-frequency-duration charts and applied to a singular temporal pattern relevant to

the duration and probability of the design event to generate a hydrograph. This simplistic approach has been widely criticized for its inability to manage parameter uncertainty or account for the probabilistic nature of key variables except for the rainfall depth (Wood 1976, Rahmen *et al*. 2002, Kuczera *et al.* 2006, Nathan *et al.* 2003).

Temporal patterns of rainfall are known to exhibit a wide variability in their observed values and the use of mean or median values of these variables has been shown to be insufficient to represent real-world hydrograph predictions in modelling (Caballero and Rahman 2014a, 2014b). A movement toward the consideration of multiple temporal patterns for the hydrologic modelling of each design storm event has therefore emerged in recent years. Developing methods include sampling of historic temporal patterns (Loveridge and Rahman 2014) as well as artificial disaggregation of rainfall totals using multiplicative random cascade models (Müller and Haberlandt 2018).

In the latest Australian Rainfall and Runoff (Ball *et al.* 2016), referred to below as ARR2016, the consideration of a probabilistic array of multiple temporal patterns is recommended, considering either a Monte-Carlo sampling process or an ensemble of ten temporal patterns, which are made available to practitioners. The ensembles of temporal patterns are contained in a national database, including suitable samples of data from real recorded rainfall events throughout Australia, created using the Bureau of Meteorology's pluviograph data, totalling 2290 pluviographs across Australia (Loveridge *et al.* 2015a).

A serious question has emerged from this evolution that whether we can rely on our OSD systems that have been designed and constructed using deterministic methods to achieve their stated objectives?

5.2 Case Study

5.2.1 South East Queensland

South East Queensland, Australia has a temperate subtropical climate with high rainfall intensities and rainfall patterns that concentrate rainfall into the summer months. In this region, urban development is governed by local city council's development guidelines that broadly mandate OSD for all development sites. OSD is therefore a key consideration for new developments, requiring the installation of tanks, basins or other systems capable of holding volumes of stormwater in accordance with site-based engineering design calculations.

Recommended practice for the design of OSD in South East Queensland is undertaken using numerical runoff routing methods (Institute of Public Works Engineering Australia 2017). Simplified alternatives known as ‘deemed to comply’ solutions are also available to designers that avoid numerical methods, albeit known that the over-simplification of these methods present issues associated with both accuracy and effect (Ronalds and Zhang 2017).

5.2.2 Example Development Sites

Twenty real examples of urban development have been assessed in this study. The area of the sites range from 405m^2 to 4367m^2. The time of concentration, t_c, and the percentage of the impervious area of the sites in pre- and post- development scenarios are summarized in Table 5-1. All of the development sites are located in the South East Queensland region. All have been designed, approved and constructed using the design principles of ARR1987. All sites consist of a single catchment contributing to runoff in both the pre- and post-development scenarios.

Table 5-1 – Summary of Development Sites Hydrologic Parameters

Site Number	Area (m^2)	Pre-Development		Post-Development	
		t_c (minutes)	% Impervious	t_c (minutes)	% Impervious
1	405	5.0	75.0%	5.0	90.0%
2	675	5.5	35.0%	5.0	85.0%
3	706	5.0	60.0%	5.0	85.0%
4	785	5.0	30.0%	5.0	80.0%
5	1012	5.0	0.0%	5.0	50.0%
6	1012	5.0	20.0%	5.0	80.0%
7	1031	6.0	35.0%	5.0	95.0%
8	1214	9.5	37.0%	5.0	83.0%
9	1304	5.0	0.0%	5.0	60.0%
10	1338	5.0	20.0%	5.0	70.0%
11	1406	5.0	0.0%	5.0	50.0%
12	1554	5.0	40.0%	5.0	65.0%
13	1613	7.2	15.0%	5.0	80.0%
14	1889	5.0	60.0%	5.0	90.0%
15	1927	7.0	40.0%	5.0	85.0%
16	2170	9.7	70.0%	5.0	90.0%
17	2275	7.0	20.0%	5.0	75.0%
18	2428	15.0	10.0%	8.0	50.0%
19	4367	7.0	45.0%	5.0	75.0%
20	4367	7.0	45.0%	5.0	70.0%

5.3 Ensemble Analysis

5.3.1 Hydrologic Modelling Methodology

The Queensland Urban Drainage Manual (Institute of Public Works Engineering Australia 2017) gives designers the option to use numerical runoff routing methods for the creation of hydrographs and the assessment of OSD. Acceptable alternatives include RAFTS (Aitken 1975), RORB (Laurenson and Mein 2010), WBNM (Boyd *et al.* 1993, Boyd *et al.* 1996, Boyd *et al.* 2012), PC-DRAIN (Badini 2018) and DRAINS (O'Loughlin and Stack 2014). DRAINS has been selected for the purposes of this analysis, which uses the time area

method for catchment routing and level pool routing for OSD. Recent revisions of DRAINS have also included the ability to assess ensembles of temporal patterns in accordance with ARR2016.

The methodology of the ensemble analysis has been to revisit the hydrologic modelling of each development site in Table 5-1 using the ensemble of ten temporal patterns and techniques provided by ARR2016. The hydrograph yielding the peak flow that is closest to the median of the ensemble of results has been taken as the accepted result, in both the pre- and post- development scenarios. The OSD is considered to be successful in achieving its objectives when the peak flow rate of its outlet is equal to or less than the pre-development peak site discharge.

5.3.2 Performance of Detention

In the graphs presented below, the baseline (0% deviation line) represents the achievement of a detention outlet flow rate that is equal to the value of the pre-development site. The percentage of deviation indicates the extent of failure (negative results) or over-attenuation (positive results) that occurs from the modelling of the site and OSD system using ensemble methods. The horizontal axis represents the frequency of the design storm in terms of AEP.

For the ten smaller sites (with the area less than 1400m^2) the results presented in Figure 5.1 identify a clear pattern in which the OSD systems achieve and exceed the stated objectives for the rarer storm events with lower AEP. However, they fall short of achieving attenuation objectives for the more frequent storm events with larger AEP values. The OSD systems exceed objectives by 10% to 15% in rarer storm events with smaller AEP values, but fail in achievement of objectives by 5% to 10% in frequent storm events with larger AEP values according to the average attenuation.

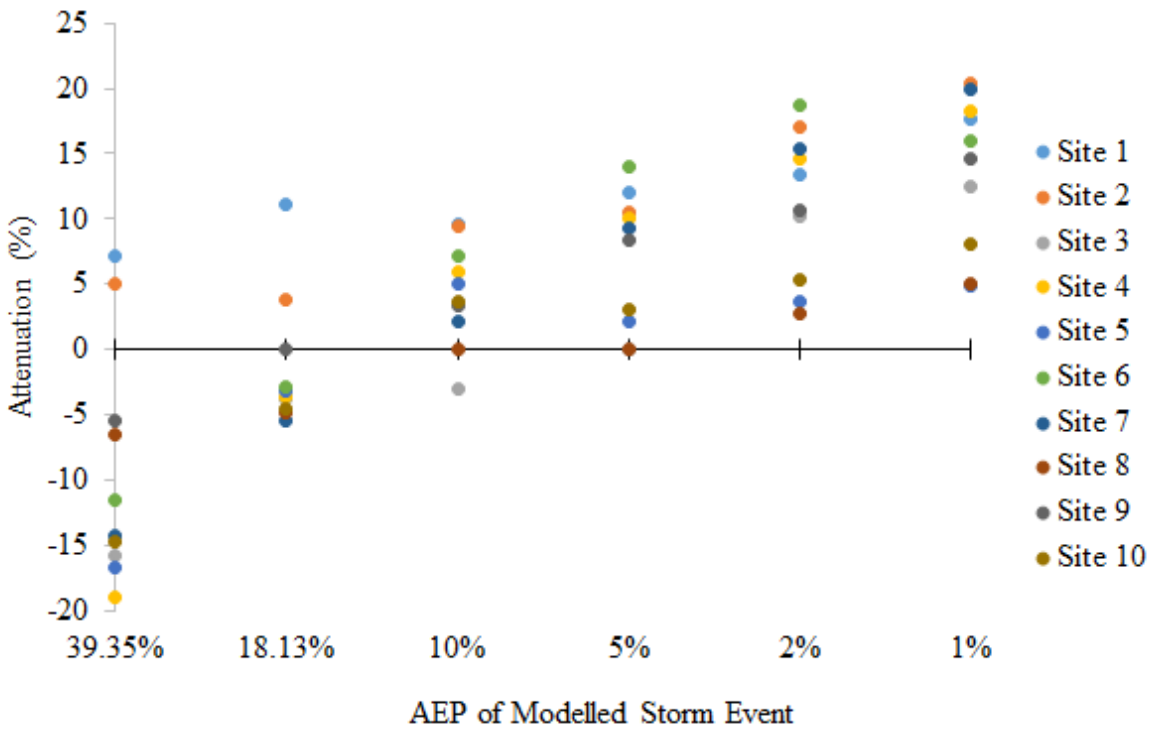

Figure 5-1 – Performance of OSD in the Achievement of Objectives – Small Catchments

For the ten larger sites (with the area larger than 1400m^2), the results presented in Figure 5.2 identify an exaggeration of the results shown in Figure 5.1, with the OSD systems exceeding objectives by 15% to 20% in rare events and failing in achievement of objectives by 10% to 15% in frequent events according to the average attenuation.

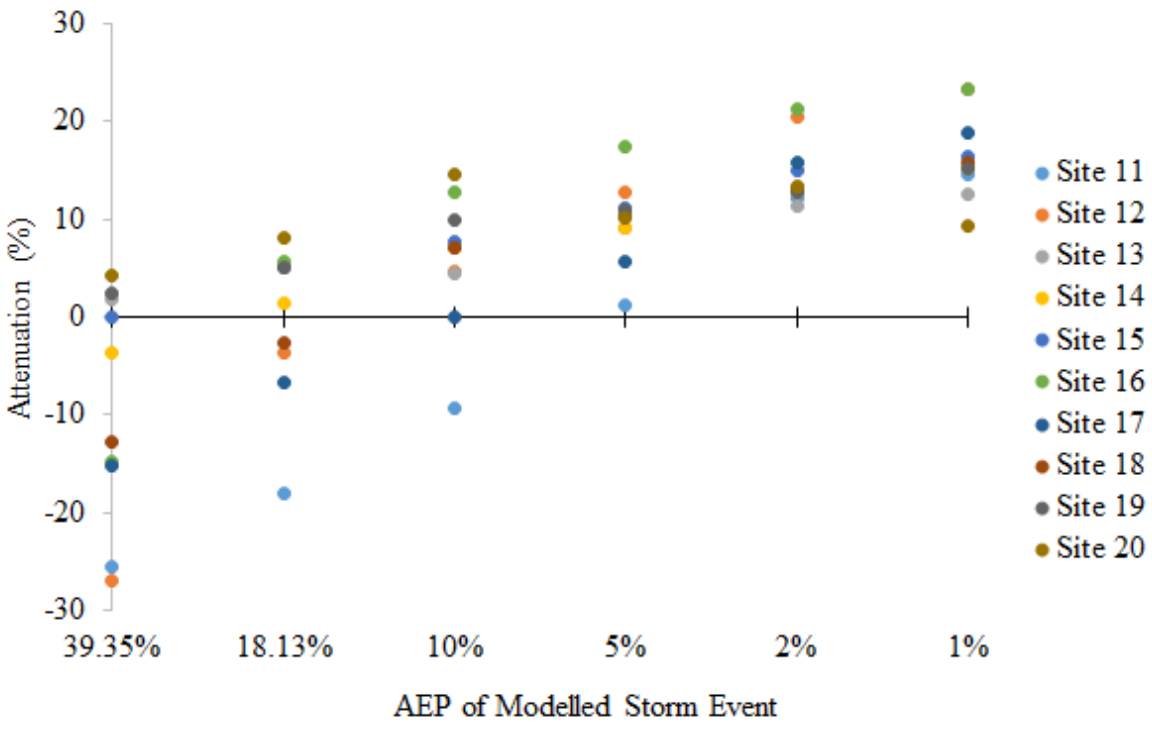

Figure 5-2 – Performance of OSD in the Achievement of Objectives – Large Catchments

5.3.3 Failure of Objectives during Frequent Events

Figure 5-3 shows the detailed results of all development sites and their achievement of objectives during a sample of frequent events (39.35% AEP). The results show a prevalence of failure, with fifteen out of twenty sites unable to achieve objectives.

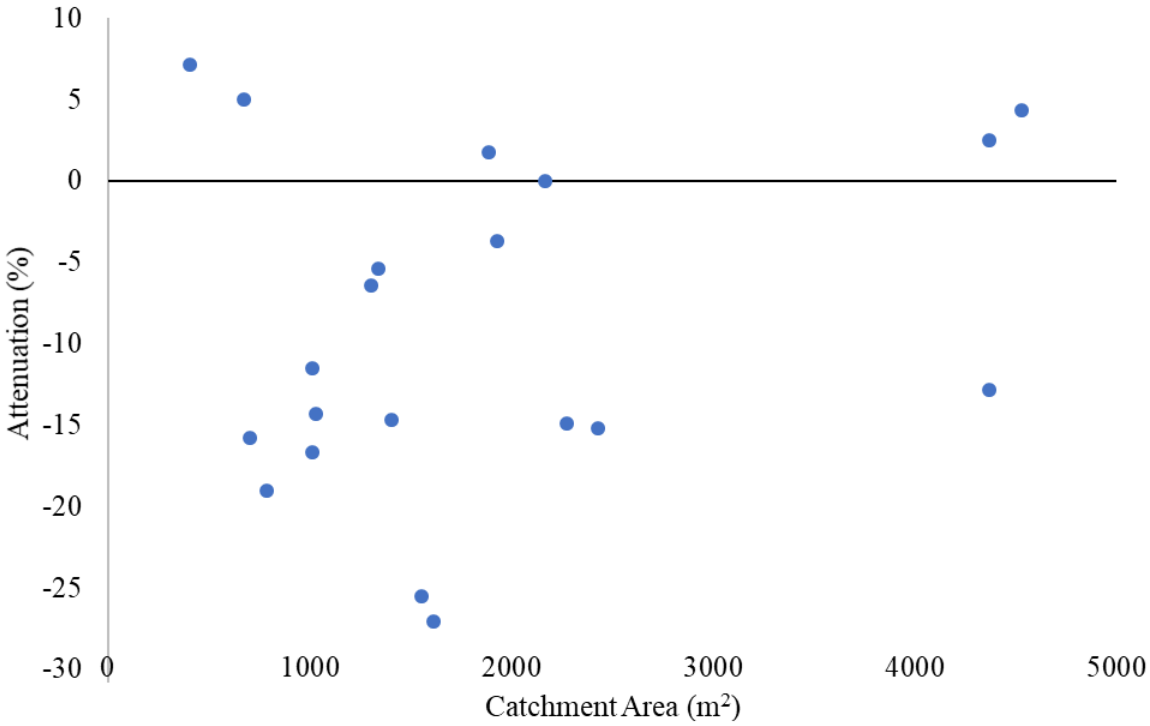

Figure 5-3 – All Development Sites – Achievement of Objectives During 39.35% AEP Event

5.3.4 Correlation Between Peak Discharge and Detention Objectives

Figure 5-4 is presented to address the potential for a simple link to be drawn between changes to peak hydrograph discharge between the ARR1987 and the ARR2016 methodologies and the patterns of OSD performance that have been identified. The figure shows a result for each development site and each AEP, indicating whether the ARR2016 peak runoff calculation (without OSD) results in a positive or negative increase in site discharge when compared to ARR1987.

It is almost unanimously demonstrated that the ARR2016 ensemble methodologies result in a reduction in peak discharge for all AEP events. There is no clear correlation between Figure 5-4 and the results presented in Figures 5-1 and 5-2.

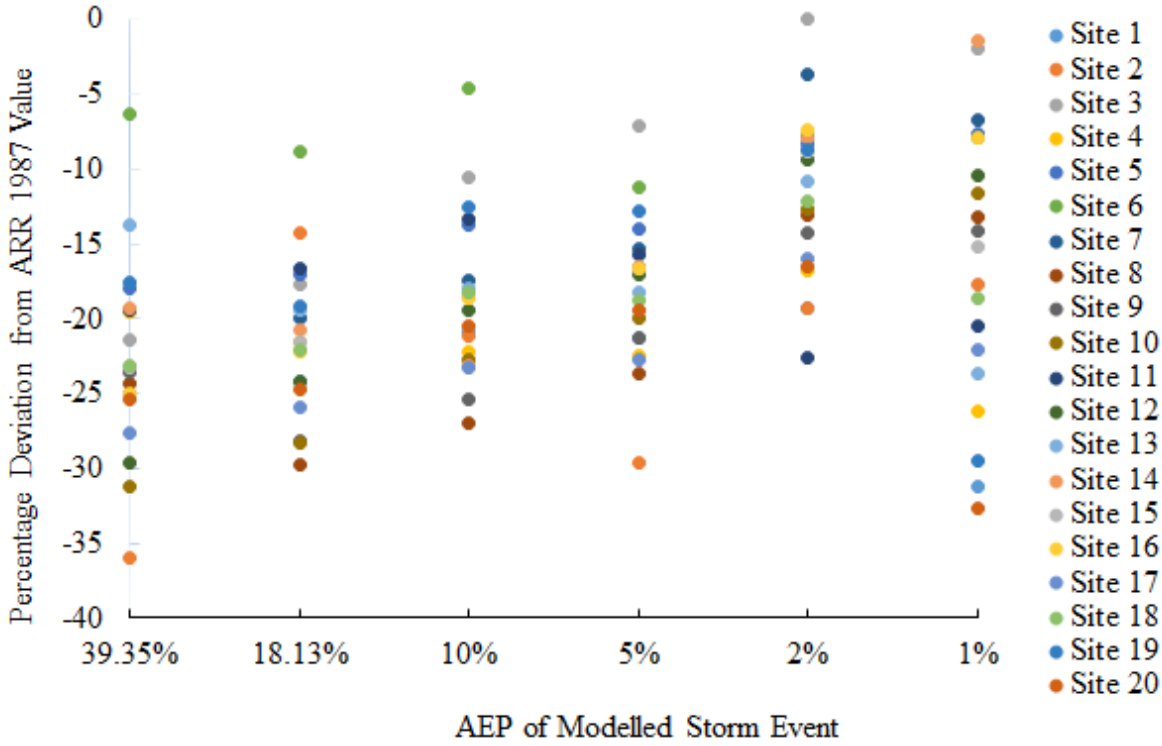

Figure 5-4 – Difference Between ARR1987 and ARR2016 Peak Flow Rates (without OSD) for All Sites and Storms

5.3.5 Ensemble Analysis Summary

The results of the ensemble analysis indicate a real potential that numerous OSD systems constructed in the South East Queensland region may under-perform in their achievement of frequent flow management objectives. Secondly, an over-achievement of stormwater discharge management objectives in major storm events could be consuming excessive construction materials and resources.

Issues of erosion and water quality problems that result from increases to peak discharge in frequent events are well established both globally (McCuen and Moglen 1988) and locally (Walsh *et al.* 2016). This is considered to be an important issue of concern and a topic worthy of continued investigation.

5.4 Continuous Simulation Analysis

The second analysis presented in this study focuses on the identified issues of frequent flow management and considers how the OSD systems perform when they are assessed using real patterns of recorded rainfall within the probability range of frequent events. The continuous simulation method involves running a conceptual runoff-routing model for a long period of time such that all important interactions (covering the dry and wet periods)

between the storm (intensity, duration, temporal pattern) and the catchment characteristics are adequately sampled (Nathan and Ling 2016).

Continuous simulation of stochastically generated rainfall is a technique commonly used to perform a flood frequency analysis of routed runoff hydrographs from a long series of recorded rainfall, which is then used to establish design storm events from within the data (Woldemeskel *et al.* 2016). This method provides a realistic example of the design storm, including not only peak discharge but also temporal shape and pre-burst depths.

In the application used for this analysis, the continuous simulation method is used to model a true and accurate representation of rainfall patterns for the purposes of assessing the performance of OSD during frequent, real storm events that have occurred in the proximity of the development sites within recent history.

5.4.1 Model Development

A runoff-routing model has been developed to perform the continuous simulation with four key sequential components, being input rainfall records from local pluviograph stations, a loss model, non-linear catchment runoff-routing and level-pool routing of detention storage via the establishment of an outlet rating curve. The procedure, described using the flow chart in Figure 5-5, has been carried out for each example development site in Table 5-1 and for each series of recorded rainfall data.

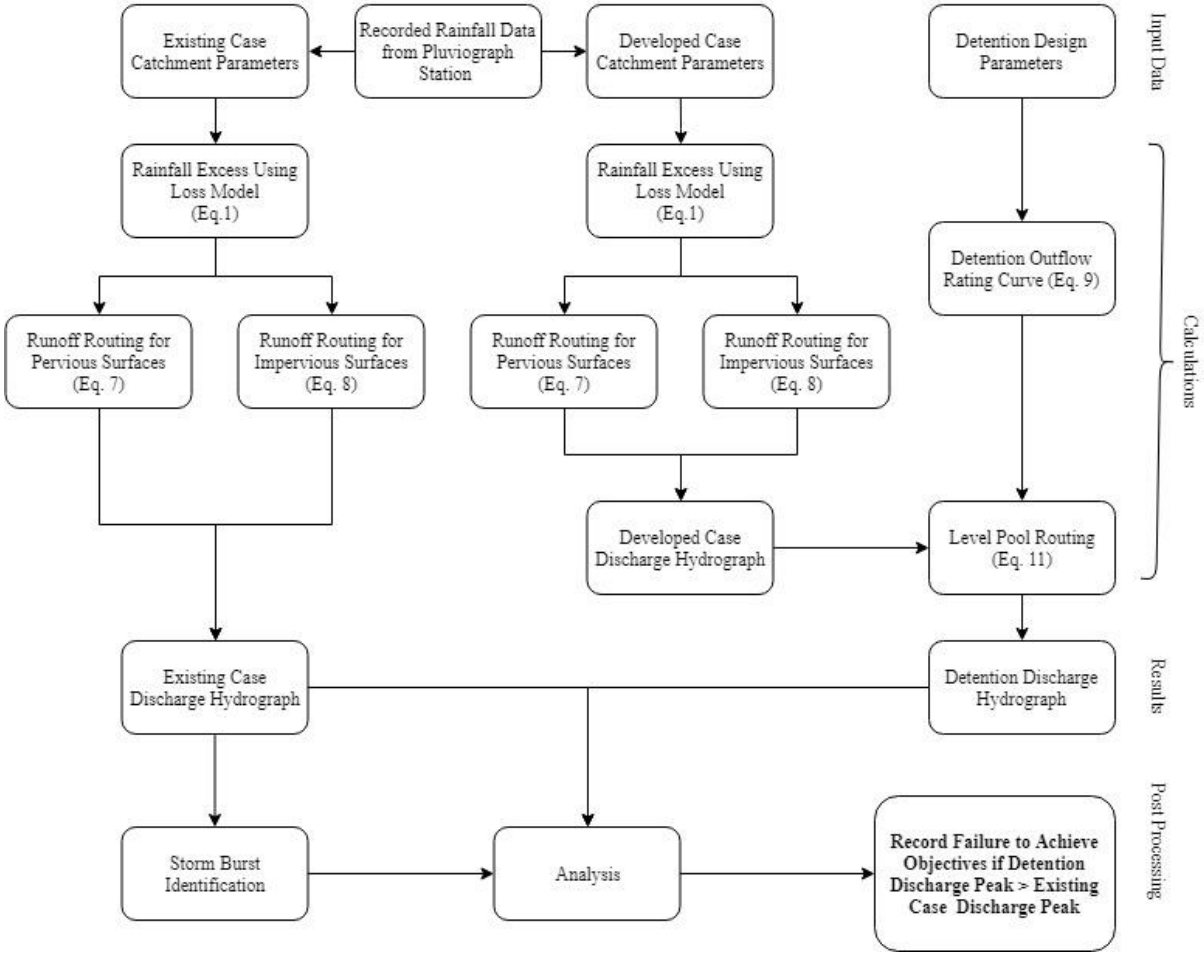

Figure 5-5 – Continuous Simulation Flow Chart

The Queensland Department of Natural Resources, Mines and Energy (2018) provides open access to pluviography records for a number of South East Queensland catchments. Rainfall data was obtained for each of the stations summarized in Table 5-2 below.

Table 5-2 – Pluviograph Station Data

Location	**Station ID**	**Dates**	**Total Rainfall During Period**
Thornton (Brisbane)	1403P001	13/02/2014 – 03/02/2018	2,336 mm
Springbrook (South Coast)	1460P001	08/06/2012 – 03/02/2018	12,992 mm
Illinbah (South Coast)	1460P002	26/06/2012 – 03/02/2018	7,457 mm

For the removal of rainfall losses from the rainfall data series prior to runoff routing, the Initial Loss - Continuing Loss (*IL/CL*) model has been adopted (Hill and Thompson 2016), following the form of equation 5-1:

$$P_L = \begin{cases} P_t & 0 < P_t < P_{IL} \\ P_{CL} & P_t > P_{IL} \end{cases} \qquad 5\text{-}1$$

where P_L is the rainfall loss, P_T is the total rainfall, P_{IL} is the initial loss, and P_{CL} is the continuing loss.

To account for the drying of the catchment between storm bursts, the initial loss calculation was programmed to re-set after a period of time without significant rainfall. The case with rainfall less than 5 mm in three days was adopted as the re-set trigger for identifying and separating individual storm bursts for assessment.

Non-linear routing was considered necessary for this application given the importance of sensitivity to relatively minor alterations in hydrograph shape, and in lieu of simpler methods such as Muskingham-Cunge (Cunge 1969). The routing calculations involve the continuity equation (Equation 5-2) and its general solution (Equation 5-3), where the incoming rainfall excess hyetograph (*I*) is routed through the catchment storage (*S*) to generate the hydrograph outflow (*Q*).

$$\frac{dS}{dt} = I(t) - Q(t) \qquad 5\text{-}2$$

$$\frac{2(S_{i+1} - S_i)}{t_{i+1} - t_i} = (I_{i+1} + I_i) - (Q_{i+1} + Q_i) \qquad 5\text{-}3$$

where t is the time, and subscript i means the time step. *Equation 5*-4 shows the nonlinearity of catchment storage, where k is a dimensionless variable depending on the storage and discharge characteristics of the catchment, and m is a dimensionless constant:

$$S = kQ^m \qquad 5\text{-}4$$

The solution to k developed by Boyd (1996) and utilised in the runoff-routing software WBNM has been adopted for this analysis, provided in Equations 5-5 and 5-6 for pervious and impervious portions of the catchment area (A) respectively.

$$k_{pervious} = \left(C_{lag}\, A^{0.57} Q^{-0.23}\right) Q^{1-m} \qquad 5\text{-}5$$

$$k_{impervious} = \left(0.1\, C_{lag}\, A^{0.25}\right) Q^{1-m} \qquad 5\text{-}6$$

Where C_{lag} is a dimensionless parameter that is used to calibrate the equations to gauged flow or other methods of estimation.

Equation 5-7 is a workable routing equation for pervious catchment runoff that is a combination of Equations 5-3 and 5-5. The solution requires an iterative process to solve the current outflow (Q_{i+1}) at each timestep of the simulation, where the current timestep inflow (I_{i+1}), previous timestep inflow (I_i) and previous timestep outflow (Q_i) are all known parameters.

$$\frac{I_i + I_{i+1}}{2} + \left(\frac{\left[\left(C_{lag}\, A^{0.57} Q_i^{-0.23}\right) Q_i^{1-m}\right] Q_i^m}{\Delta t} + \frac{Q_i}{2}\right) - \left(\frac{\left[\left(C_{lag}\, A^{0.57} Q_{i+1}^{-0.23}\right) Q_{i+1}^{1-m}\right] Q_{i+1}^m}{\Delta t} + \frac{Q_{i+1}}{2}\right) - Q_i = 0 \qquad 5\text{-}7$$

Equation 5-8 follows that same process of combining Equations 5-3 and 5-6 and is used to solve for the impervious portions of catchments.

$$\frac{I_i + I_{i+1}}{2} + \left(\frac{\left[\left(0.1 C_{lag}\, A^{0.25}\right) Q_i^{1-m}\right] Q_i^m}{\Delta t} + \frac{Q_i}{2}\right) - \left(\frac{\left[\left(0.1 C_{lag}\, A^{0.25}\right) Q_{i+1}^{1-m}\right] Q_{i+1}^m}{\Delta t} + \frac{Q_{i+1}}{2}\right) - Q_i = 0 \qquad 5\text{-}8$$

For all catchment routing processes, the results of Equations 5-7 and 5-8 are added at each time step to produce the total catchment discharge hydrograph.

To describe the outflow of the detention system at various storage depths during the simulation (Q_h), a rating curve must be developed. The procedure to develop Equation 5-9 can be found at Ronalds and Zhang (2017).

$$Q_h = \begin{cases} \frac{1}{n} A_w \left(\frac{A_w}{P}\right)^{\frac{2}{3}} S_g^{\frac{1}{2}} + C_w L_w h_{weir}^{\frac{3}{2}} \langle\, h \le D \rangle \\ A_p \left[\sqrt{\left(\frac{2gD(h - h_{DS})}{L f_D}\right)}\right] + C_w L_w h_{weir}^{\frac{3}{2}} \langle for\ h > D \rangle \end{cases} \qquad 5\text{-}9$$

Where A_p, A_w, P, D, L, n, and S_g are the total area, wetted area, wetted perimeter, diameter, length, roughness, and gradient of the outflow pipe, respectively; C_W, L_W, and h_{weir} are the weir coefficient, length of the weir, and height of the weir above the base of the detention, respectively; h is the depth of water in the detention, h_{DS} is the standing water level downstream of the detention, and g is the gravitational acceleration. In this equation the outflow pipe is located at the base of the detention, which is necessary to ensure it is free draining and does not hold water when there is no storm event. The Darcy–Weisbach friction factor, f_D, is described by Equation 5-10 (Brown, 2002), which is dependent upon the Reynolds number, R_e.

$$\frac{1}{\sqrt{f_D}} = -2\,log\left(\frac{5.76}{Re^{0.9}}\right) \qquad 5\text{-}10$$

In this application, a fixed Reynolds Number has been adopted, validated against the programming of current industry software package such as RAFTS, RORB, WBNM, and DRAINS.

The process for calculating the detention effect on the developed case hydrograph output requires a return to Equation 5-3, which needs to be rearranged to yield the specific form of Equation 5-11 to perform level pool routing.

$$(I_i + I_{i+1}) + \left(\frac{2S_i}{\Delta t} - Q_i\right) = \left(\frac{2S_{i+1}}{\Delta t} + Q_{i+1}\right) \qquad 5\text{-}11$$

In post-processing, storm bursts relating to frequent storm events were identified for further analysis. Storm bursts are defined as a period of rainfall exceeding a specified rainfall depth (Loveridge et al. 2015b). In the case of this study, all storms events with rainfall exceeding 5 mm in three days were identified as storm bursts. From the total number of storm bursts identified in the data, the storm bursts relating to the classification of frequent storm events were extracted.

For each of the storm bursts used in the analysis, the hydrograph peak in the pre-development scenario was compared to the peak discharge with OSD systems. When the peak discharge with OSD systems exceeded that in the pre-development scenario, a failure to achieve frequent flow management objectives was recorded.

5.4.2 Simulations and Results Analysis

The total number of identified storm bursts and the total number of these that failed to achieve objectives are summarized in Table 5-3.

Table 5-3 – Results of Continuous Simulation

	Thornton Pluviograph		**Springbrook Pluviograph**		**Illinbah Pluviograph**	
Site	Number of Storm Bursts	Number of Failed Bursts	Number of Storm Bursts	Number of Failed Bursts	Number of Storm Bursts	Number of Failed Bursts
1	5	0	8	0	9	0
2	4	2	7	5	5	2
3	5	4	7	4	8	4
4	4	3	7	6	5	4
5	4	3	7	6	5	3
6	4	3	7	6	5	4
7	4	3	7	6	5	3
8	4	2	7	3	5	3
9	4	3	7	6	5	2
10	4	3	7	6	5	3
11	3	2	7	6	5	3
12	4	3	7	6	5	3
13	4	3	7	4	5	3
14	4	3	7	6	5	4
15	4	1	7	2	5	1
16	3	2	7	3	5	2
17	3	0	7	0	5	0
18	4	3	7	6	5	3
19	4	3	7	6	5	4
20	5	4	7	6	9	8

Table 5-3 shows 345 storm bursts that each represent a frequent storm event. Failure to achieve pre-development peak flow rate mitigation is observed in 202 cases, or 59% of the events. The results of the continuous simulation analysis therefore corroborate with the results of the ensemble analysis, highlighting a significant potential for unanticipated increases in peak stormwater runoff from actual development sites during frequent flow events.

5.4.3 Example Event Analysis: Cyclone Debbie

Many of the failures to achieve peak flow discharge objectives identified in Table 5-3 have occurred during the rainfall event in South-East Queensland that followed Tropical Cyclone Debbie in May 2017. The rainfall pattern at the peak intensity of Debbie had a burst duration of approximately 120 minutes and a peak rainfall intensity of approximately 120 mm/hr. Depending on the development site size and its catchment response, the rainfall burst correlates approximately to a runoff event with an AEP between 63% and 20%.

Figure 5-6 shows a comparison of how the recorded rainfall pattern during Debbie's 120 minute storm burst compares to the design temporal pattern of the 120 minute storm event given by ARR1987. The vertical axis describes the percentage of the total rainfall volume during the storm event as it falls over time on the horizontal axis. While the two data sets achieve a comparable peak, the patterns are distinctively different in shape and timing. In both instances the peak of the storm delivers a 5-min burst with an intensity holding 14.5% of the total storm volume. The design storm has its peak occurring early, at 35 min in the storm event, whereas the observed pattern has the peak landing much later, at 70 min in the storm event.

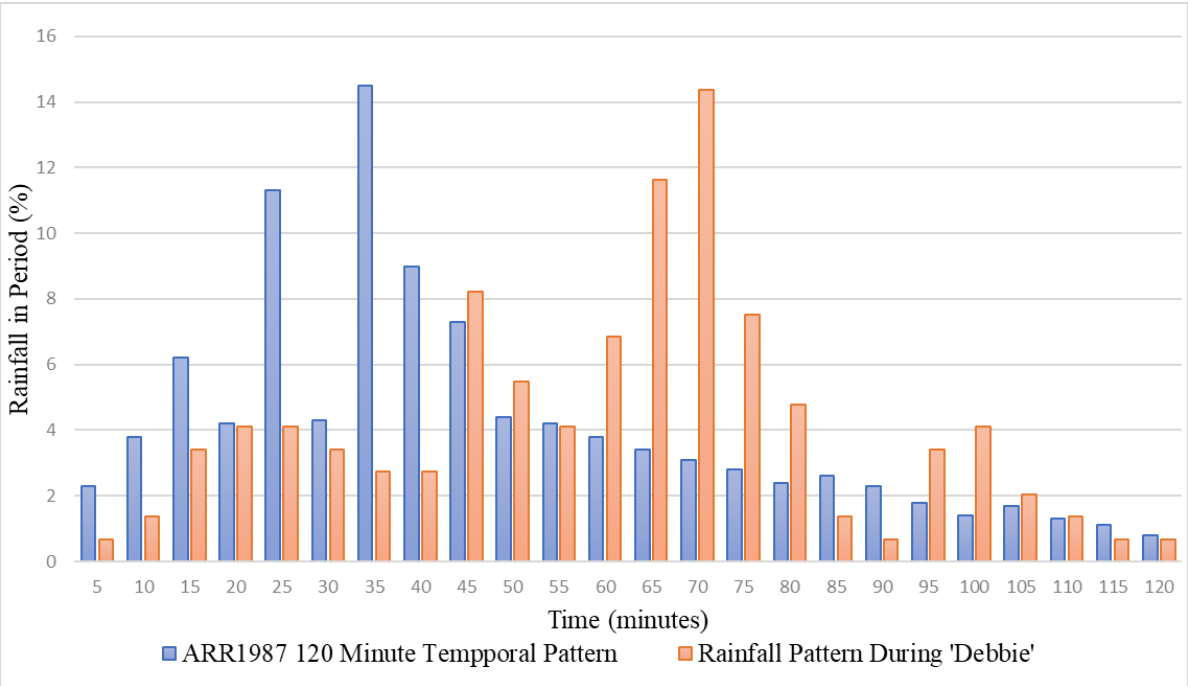

Figure 5-6 Cyclone Debbie Recorded Rainfall Pattern Compared to ARR1987 Temporal Pattern

Figure 5-7 shows the routed hydrograph results that relate to the rainfall during Cyclone Debbie occurring at the example development site No. 15. The results show that the OSD is

effective in reducing a number of the smaller hydrograph spikes to pre-development conditions, however at the largest peak of the burst the OSD outflow is in excess of the pre-development conditions, signifying failure.

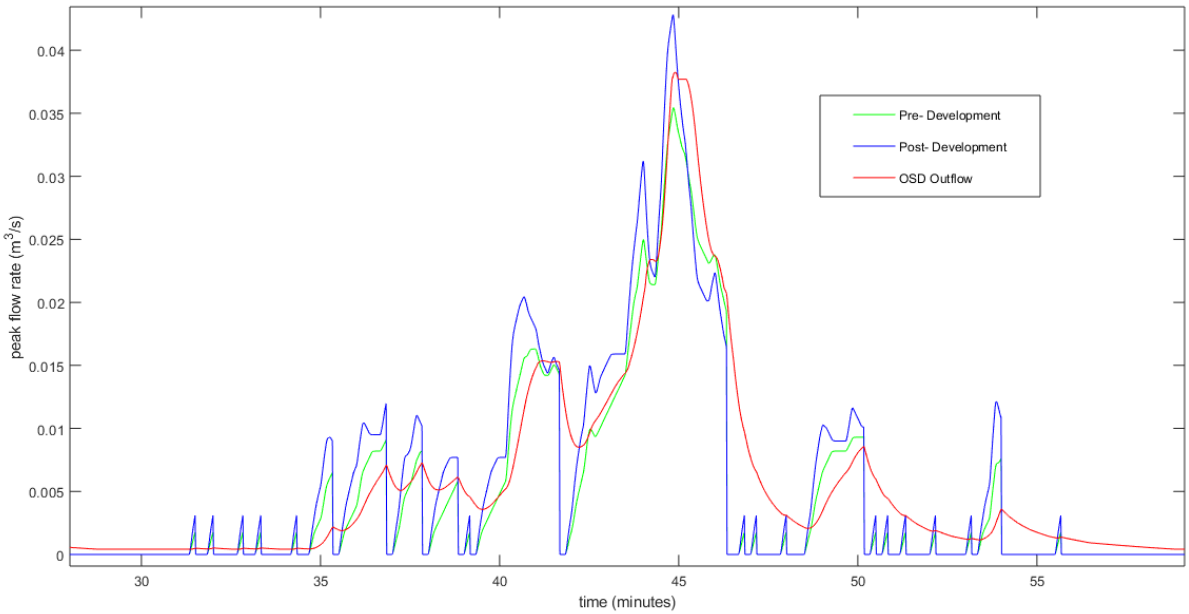

Figure 5-7 – Failure to Achieve Objectives During the Peak of Cyclone Debbie Runoff

The failure shown in Figure 5-7 can be explained by the inconsistency observed between the rainfall patterns in Figure 5-6. During the design and assessment of the performance of this specific OSD, it can be assumed that strict reliance upon the temporal patterns given by ARR1987 have been used to reach the conclusion that the system achieves its objectives. In reality however, the rear-loaded nature and clustering of the actual rainfall pattern that occurred during Cyclone Debbie has resulted in 'unexpected' conditions that the OSD has not been designed to account for.

5.5 Discussion

5.5.1 Broadscale Retrofit for Existing Detention Systems

The data and analyses presented in this paper can be used to assess the potential for a generalized solution to the problem. A universally applied reduction in the OSD pipe outlet diameter is considered to be a potential solution. In practice, this can be achieved via a basic retrofit of existing OSD systems by installation of an orifice plate on the outlet pipe(s). Based on additional continuous simulation assessment, Fig. 5-8 shows a performance curve of the varying failures to achieve frequent flow management objectives that result from incremental testing of reductions to the OSD pipe outlet diameter. All twenty development

sites in Table 5-1 in response to rainfall from all three pluviograph stations in Table 5-2 were modelled to create Fig. 5-8.

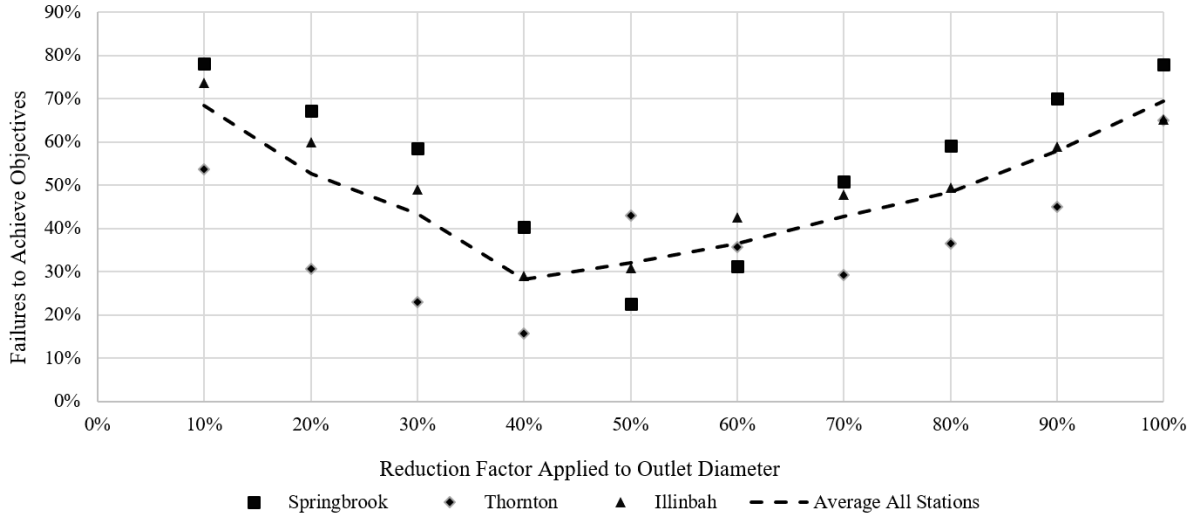

Figure 5-8 – Performance Curves of Outlet Diameter Reductions in the Improvement of Frequent Event Failure

Figure 5-8 indicates that a 40% reduction factor applied universally to the OSD pipe diameter of all the development sites yields the optimal results. After this retrofit is physically introduced to all the OSD systems, the average probability of failure is reduced from 59% to less than 30%.

Further assessment of this proposal using the ensemble analysis provides a less promising view however. Assuming that the 40% OSD outlet reduction factor is universally applied to all twenty development sites, Table 5-4 shows how the systems would perform for both frequent (63% AEP) and rare (1% AEP) design events. In this table, a pass is assigned to an outcome where the OSD achieves a reduction in peak flow that results in a flow rate that is less than pre-development conditions, whereas a fail is assigned to an outcome in which the OSD does not achieve its intended mitigation of peak runoff to pre-development conditions.

Table 5-4 – Assessment of Broadscale 40% Reduction Factor Applied To OSD Pipe Diameter

Site Number	Frequent Events	Rare Events
1	Pass	Pass
2	Fail	Pass
3	Pass	Fail
4	Fail	Fail
5	Pass	Fail
6	Pass	Fail
7	Pass	Pass
8	Pass	Fail
9	Fail	Fail
10	Pass	Fail
11	Fail	Fail
12	Pass	Pass
13	Fail	Fail
14	Fail	Fail
15	Pass	Fail
16	Pass	Fail
17	Pass	Fail
18	Pass	Fail
19	Pass	Fail
20	Pass	Fail

The results from the ensemble analysis of the retrofit strategy in Table 5-4 agree with the continuous simulation results of the retrofit strategy in Figure 5-8, showing that the occurrence of failure to achieve frequent flow objectives can be reduced to six out of twenty sites (i.e. 30% failure). However, this comes at the cost of a significant increase in expected failures to meet objectives in rare events.

5.5.2 Moving Forward with Ensemble Analysis

Based on the results in Table 5-4, a broadscale retrofit solution is not considered to be a viable recommendation. In exchange for a marginal improvement to performance in frequent storm events, an increased expectance of failure in major events is considered to

be a likely and unacceptable consequence. This leaves the solution to the problem limited to case-by-case re-design using improved methods.

Using the ensemble design methodology, all twenty sites have been re-modelled in DRAINS and the OSD systems have been reconfigured to ensure that objectives are met for all design storm events. The modifications required are described in Table 5-5, including alterations to both outlet pipe diameter and storage volume. Also presented in Table 5-5 are the results of the revised OSD system's performance in response to the real recorded rainfall using the continuous simulation analysis.

Table 5-5 – Assessment of Case-by-Case Retrofit to Comply with ARR2016

	Design Modification		**Continuous Simulation Results (Number of Failures)**		
Site Number	Pipe Decrease	Volume Increase	Thornton Pluvio	Springbrook Pluvio	Illinbah Pluvio
1	0%	0%	0	0	0
2	10%	0%	2	5	4
3	33%	8%	1	0	0
4	20%	0%	3	3	3
5	20%	35%	2	6	4
6	56%	9%	0	0	0
7	33%	15%	2	2	2
8	0%	0%	2	3	3
9	20%	13%	3	6	4
10	16%	0%	3	3	4
11	20%	28%	2	6	4
12	47%	0%	2	6	4
13	11%	0%	2	3	4
14	37%	23%	1	0	0
15	11%	0%	1	0	0
16	33%	8%	0	0	0
17	33%	19%	0	0	0
18	20%	11%	3	3	2
19	16%	0%	2	1	1
20	0%	0%	4	6	8

Table 5-5 presents an important finding that considering the ensemble of ten temporal patterns in the design of the OSD systems does not guarantee the achievement of peak flow management objectives in response to real rainfall. Whilst all the OSD systems fully

achieved objectives for all design storm events using the ensemble analysis, OSD systems failed to achieve objectives for a total of 135 storm bursts, or 39% of the total bursts, according to the continuous simulation analysis. This is however reduced from the 202 storm bursts, or 59% of the total bursts, in which OSD systems failed when designed using single temporal pattern methods of ARR1987. The findings therefore support the movement away from usage of singular temporal patterns and toward a need to consider multiple temporal patterns in the design of OSD systems.

5.5.3 Ensemble Prediction Uncertainty

In alignment with current practice, this study adopted the closest-to-median hydrograph result from one of the ten ensemble analyses, and disregarded the remaining nine hydrographs. Assessment of the OSD performance was then based upon a pass/fail test of the selected hydrographs. This process results in prediction uncertainty.

Typically, the range of peak flow results from the ensemble analysis varied by 10% to 20% in both pre- and post-development scenarios. The objective of the OSD is often to reduce peak discharge by quantities that are close to the variation in peak discharge results from the spread of the ensemble of hydrographs, meaning that there is potential for storm events outside of the closest-to-median hydrograph to cause additional potential for failed objectives. This reality compounds the study and provides basis for further investigations into the consideration of statistical acceptance criteria when considering multiple temporal patterns.

5.6 Summary

A concern has been raised that OSD systems constructed for recent developments in Australia are failing to meet frequent flow management objectives, due to a reliance on singular temporal patterns for design events according to their specification. This concern was confirmed using real examples of development sites and recorded rainfall series of frequent storm events.

The key finding of this study is that the usage of one design rainfall pattern or even an ensemble of ten rainfall patterns cannot reliably produce an OSD system design that will always be effective when exposed to real patterns of naturally occurring rainfall. Based on this study, certification of OSD performance should be restricted to confidence limits of no greater than 39% and 59% for frequent storm event management using the ARR1987 and ARR2016 methods, respectively.

A generalized solution was considered that could reduce the number of failures in frequent events. However, the resulting trade-off in achieving significant improvements in frequent events is an unacceptable risk of failures in rare events, which can result in flooding.

Increasing the number of temporal patterns used for the design and assessment of OSD is confirmed to improve the success rate of achieving objectives. As a recommended solution to the problem, as many potential temporal patterns should be considered as practically possible.

6. Assessing the Impact of Urban Development and On-Site Stormwater Detention on Regional Hydrology using Monte Carlo Simulated Rainfall

STATEMENT OF CONTRIBUTION TO CO-AUTHORED PUBLISHED PAPER

This chapter includes a co-authored paper. The bibliographic details of the co-authored paper, including all authors, are:

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My contribution to the paper involved the development of the research methodology, the execution of the research and the preparation of the manuscript.

(Signed) _____ (Date) 28/2/19

Rodney Ronalds

(Countersigned) _____ (Date) 06/03/19

Supervisor: Professor Hong Zhang

Overview of Chapter

Urban development is a contributor to increased peak runoff and adverse hydrologic effects in regional catchments. On-Site Stormwater Detention (OSD) is a common way to mitigate these problems, however it is well known that OSD can have the opposite effect when it is installed at inappropriate locations. Parameter uncertainty and the need for a probabilistic approach to hydrograph generation are also factors that add to concerns regarding our reliance on OSD for the protection of regional hydrology. This study contributes to awareness of these issues and a practical solution to the problem. A hydrologic model for Monte Carlo simulation of regional catchment hydrographs has been developed using interrelated modules based on previous studies. A sample of ten regional catchments has been modelled with three simulation scenarios: i) the status quo, ii) a land parcel of varying sizes is urbanised at varying locations within the regional catchment, and iii) the urbanised land parcel includes OSD. The focus on the results has been the identification and analysis of two key parameters that influence the regional catchments' peak runoff, being the size and location of the urbanised land parcel. A regression analysis of the model results has revealed recurring patterns that have been used to develop new equations for predicting the mean impact of urbanisation and OSD on regional catchment peak runoff. The study highlights the significance of rainfall pattern uncertainty and the importance of considering land parcel location when planning for OSD as part of urban land development projects.

6.1 Introduction

Increased imperviousness is the inevitable outcome of urbanisation. The impact on regional hydrology is increased peak runoff and a variety of associated adverse effects, including increased flooding, erosion, widening of channels, sediment deposition, pollutant delivery to streams, and other ecological disruptions (Arnold and Gibbons 1996, Booth 1991, Burns *et al*. 2012, Hood *et al*. 2007, Berland *et al*. 2017). Hollis (1975) showed that as a regional catchment reaches 30% imperviousness, the 100-year peak flow of runoff can double in comparison to those of their corresponding undeveloped catchments. Muñoz *et al*. (2018) showed that even as catchments that are already partially developed experience intensifying urbanisation, increasing peak flow of runoff will continue to rise.

On-Site Stormwater Detention (OSD) is a well-established and effective solution to reduce peak runoff from a developed land parcel and act to mitigate the adverse impact of urbanisation on regional hydrology (Bennett and Mays 1985). OSD works by collecting and temporarily storing runoff within a development site prior to its release to the downstream drainage network. The detained runoff is typically held in either underground tanks or above ground basins that allow for controlled outlet during and shortly after the storm event has finished.

The focus of OSD design is commonly the maintenance of pre-development peak runoff at the outlet of an urbanised land parcel, which is often referred to as “micro-management” policy (Olenik 1999, Walesh 1989). McCuen (1974) provided some of the first evidence that this approach may accidently increase flow rates in other parts of the regional catchment as a result of coincident hydrograph peaks at stream confluence locations. Early studies into solving this issue aimed to identify exclusion zones for OSD in the regional catchment, such as the lowest 20% (Bedient and Flores 1982) or the lowest one third (Leise 1991). More recent studies have focussed on the identification of optimal locations for OSD within the regional catchment that avoid the negative regional effects (Bellu *et al*. 2016, Duan *et al*. 2016, Kaini *et al*. 2007, Palmeri and Trepel 2002, Ravazzani *et al*. 2014, Shuster and Rhea 2013, Su *et al*. 2010, Tao *et al* 2014, Travis and Mays 2008, Wang *et al*. 2017, Zhen *et al*. 2004). Unfortunately, the perfect land for OSD is not always available within the regional catchment due to a variety of town planning and land ownership issues. The skills and resources required to analyse regional catchments and specify optimal OSD parameters are also not always available. These factors make micro-management style policies for OSD

prevalent around the world (Fang *et al*. 2017, Pezzaniti *et al*. 2003, Schueler and Claytor 2000).

The successful design of OSD requires the ability to create and manipulate the shape of hydrographs to achieve the desired outflows. Many simple yet widely adopted methods for OSD design rely on idealised triangular or trapezoidal hydrographs (Abt and Grigg 1978, Donahue *et al*. 1981, Chow *et al*. 1988, Basha 1994, Boyd 1980, Carrol 1990, Hong *et al*. 2006, Hong 2008, Ronalds and Zhang 2017). The limitations of simplistic graphical methods are well known (Cordery 1971), and the more advanced and accurate method in common practice is to use hydrologic runoff routing models.

Temporal patterns of rainfall are key to the generation of hydrographs using runoff routing and in turn, the accurate design of OSD. However, like most hydrologic parameters they are not consistent or deterministic (Nathan *et al*. 2016). Future climate uncertainty is also expected to make recorded temporal patterns even less reliable for design purposes, with increased temporal pattern variability expected to occur in parallel with climate change (Mamo 2015, Fadhel *et al*. 2018). The development of probability-based methods such as Monte Carlo simulation have evolved to account for parameter uncertainties and take a probabilistic approach to hydrologic modelling. Monte Carlo simulation has been used for joint probability analyses of stream flow in Australian catchments by Rahman *et al*. (2002), Nathan *et al*. (2003) and Babister *et al*. (2016a). Similar techniques have also been applied to catchments in Great Britain (Svensson *et al*. 2013) and Italy (Kottegoda *et al*. 2014). Monte Carlo simulation is an effective method of assessing precipitation uncertainty, and often used as a component of Bayesian flood forecasting to provide predictive distribution of flood events with uncertainty estimation (Han and Coulibaly 2017, Krzysztofowicz 1999).

This study differs to others by aiming to understand and quantify the impact on regional peak runoff that can be expected at a specific location downstream of an urbanised land parcel, considering development of that land parcel with, and without OSD. With Monte Carlo simulation adopted to account for temporal pattern uncertainty, the aim is to provide a simplistic, universally applicable method to estimate the mean impact on peak regional catchment runoff.

6.2 Materials and Methods

An integrated hydrologic model was developed to perform runoff routing of probability-based rainfall temporal patterns and generate hydrographs of regional catchment runoff. The methodology focussed on extracting and comparing the peak runoff results for three key scenarios:

1. the existing 'status quo';
2. a land parcel within the regional catchment is urbanised (without OSD); and
3. the urbanised land parcel includes OSD.

The hypothetical location of the land parcel within the regional catchment and the area of the land parcel were both varied under controlled conditions for each scenario.

The model was calibrated using flood frequency analysis outputs and used to simulate a sample of ten actual regional catchments of varying sizes and locations. The response of the regional catchments' peak flow results to variation in the land parcel's location and size parameters were analysed to identify recurring and predictable patterns. A regression analysis on the results was finally used to develop a system of equations for predicting the mean impact of urbanisation and OSD on the regional catchment peak flow.

The model, as described by the flow chart in Figure 6-1, incorporates Monte Carlo simulation for the generation of random temporal patterns (1,000 patterns for each scenario), a loss model to account for catchment infiltration, non-linear catchment routing, Muskingham stream routing, and level pool routing of OSD.

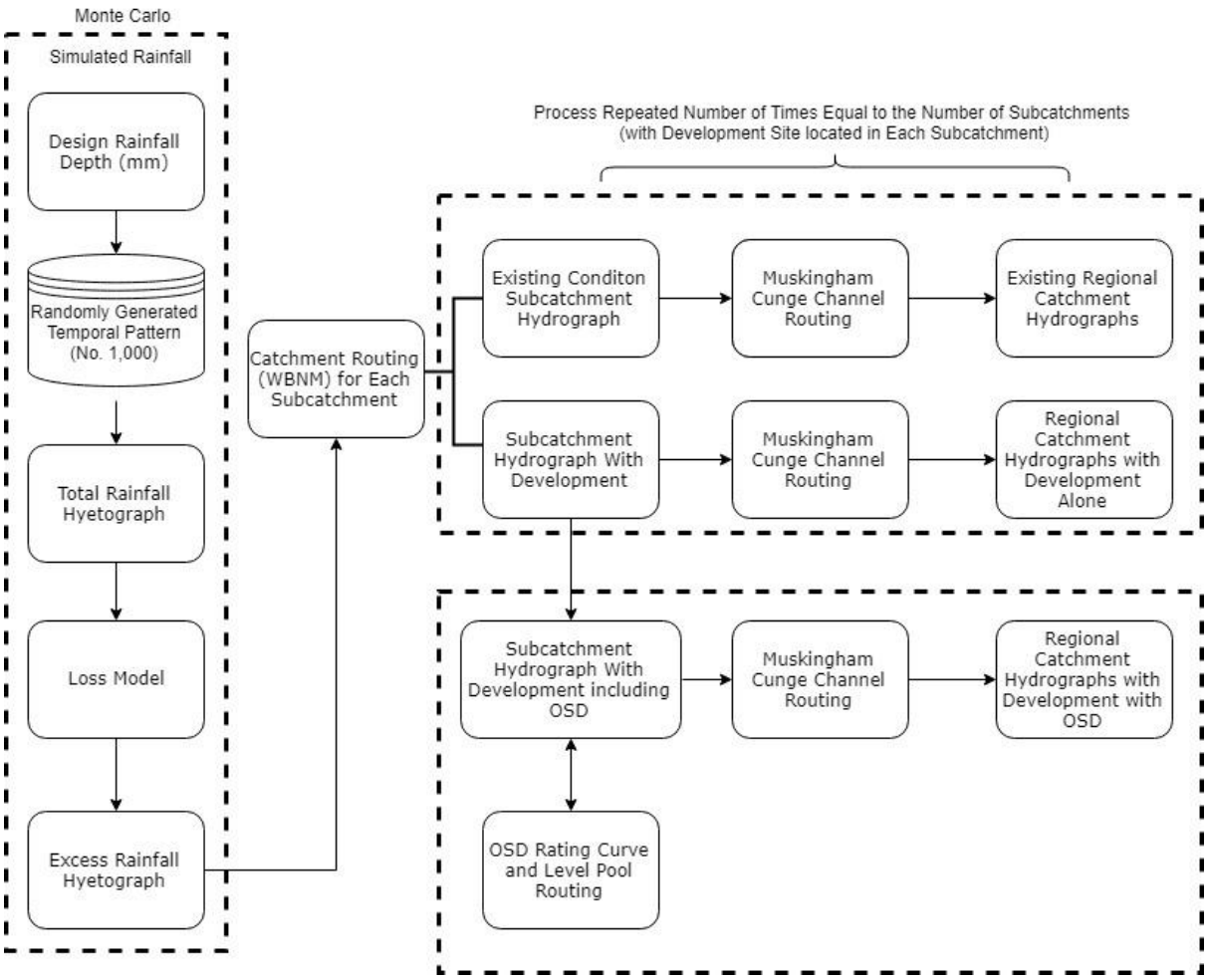

Figure 6-1 – Model Development Flow Chart

6.2.1 Monte Carlo Temporal Pattern Simulation

Monte Carlo simulation of random temporal patterns was adopted using the Multiplicative Cascade Model as described by Olsson (1998). An initial rainfall depth, *R*, over a time period of *T*, was distributed between time steps in a series using the weights $Wi,_1$ and $Wi,_2$ to disaggregate the rainfall, where *i* is the cascade level:

$$W_{i,1}, W_{i,2} = \begin{cases} 0 \ for \ \ P(0/1) \\ 1 \ for \ P(1/0) \\ x \ for \ P(x/(1-x)), 0 \ < x < 1 \end{cases} \qquad \begin{matrix} W_{i,1} = x_i \\ W_{i,2} = 1 - W_{i,1} \end{matrix} \qquad 6\text{-}1$$

In the cascading process, *P* is the probability of the rainfall amount in a time step branching from one position in the starting cascade to a split between two positions in the next cascade, where *P(0/1), P(1/0)* and $P(x/(1-x))$ are the probabilities of no rainfall, all of the rainfall, and a distribution of $x/(1-x)$ respectively. The random variable $0 \leq x \leq 1$ was used to disaggregate the rainfall from a single value in the starting cascade to two random values in the next cascade. In this process, the volume of rainfall in the storm event remains the same

with each cascade as the number of points in the time series is doubled and the rainfall disaggregates throughout the time series. The cascading process continued until a satisfactory number of cascades resulted in a suitable time step resolution ($\Delta t = \frac{T}{2^i}$) to reveal an artificial rainfall temporal pattern. The modelling agreed with a study of 24 catchments in Northern Germany by Müller and Haberlandt (2018), revealing optimal accuracy and model stability when the disaggregation process resulted in $\Delta t = 5\ to\ 7.5$ minute time steps.

6.2.2 Regional Catchment Conceptualisation and Routing Model

The regional catchment model was conceptualised using a combination of catchment and stream flow routing, with the regional catchments divided into ten equal area sub-catchments. Runoff from each sub-catchment was routed to its outlet and combined with the upstream hydrograph for stream flow routing to the downstream sub-catchment. Multiple sizes of hypothetical land parcels within the regional catchment were modelled with areas of 1ha, 2ha, 5ha, 10ha, 20ha, 50ha and 100ha.

Rainfall from each of the randomly generated temporal patterns was adjusted before routing to account for losses. The Initial Loss - Continuing Loss (IL/CL) model was adopted for this process in accordance with Hill and Thompson (2016), where:

$$Loss = \begin{cases} Rain\ \langle for\ 0 < Rain < IL \rangle \\ CL\ \langle for\ Rain > IL \rangle \end{cases} \qquad 6\text{-}2$$

For the conversion of rainfall excess to runoff from the sub-catchments the continuity equation was utilised, where the catchment outflow (*Q*) is related to the catchment inflow (*I*) and storage (*S*) at each time step:

$$\frac{dS}{dt} = I(t) - Q(t) \qquad 6\text{-}3$$

$$S = kQ^m \qquad 6\text{-}4$$

The solution to the coefficient *k* developed by Boyd *et al*. (1993) and utilised in the runoff-routing software WBNM was adopted as per Equation 6-5 and Equation 6-6 for pervious and impervious portions of the catchment area (*A*) respectively. The non-linearity parameter *m* was taken as 0.77, which has been demonstrated to be the most appropriate for potentially saturated conditions (Rezaei-Sadr *et al*. 2012). The C_{lag} coefficient is a dimensionless parameter that is used to calibrate the equations, with values between 0.5 and 2.0 adopted for each catchment and validated using gauged flood frequency analysis results.

$$k_{pervious} = \left(C_{lag}\ A^{0.57} Q^{-0.23}\right) Q^{1-m} \qquad 6\text{-}5$$

$$k_{impervious} = \left(0.1\, C_{lag}\, A^{0.25}\right) Q^{1-m} \qquad 6\text{-}6$$

For the routing of streamflow between the confluence of each sub-catchment the Muskingham routing procedures were adopted in accordance with Nash (1959):

$$Q_{n+1} = C_1 I_{n+1} + C_2 I_n + C_3 Q_n \qquad 6\text{-}7$$

Where:

$$C_1 = 1 - \frac{K(1-c)}{\Delta t} \qquad 6\text{-}8$$

$$C_2 = 1 - \frac{K(1-c)}{\Delta t} - c \qquad 6\text{-}9$$

$$C_3 = c \qquad 6\text{-}10$$

$$c = \frac{-\Delta t}{e^{K(1-X)}} \qquad 6\text{-}11$$

The value of X is a physical parameter that reflects the flood peak attenuation and hydrograph shape flattening of a diffusion wave in motion. A constant value of 0.4 was adopted for all the catchments used in this study, in accordance with the recommendations of Xiaofang (2008). The value for K is a variable dependent upon the catchment imperviousness (*Imp*) and area *(A*) which was modelled in accordance with Boyd *et al*. (1987):

$$K = 3600(1 + Imp)^{1.9}.\, 1.3(A)^{0.38} \qquad 6\text{-}12$$

Figure 6-2 provides a conceptualisation of the model set-up.

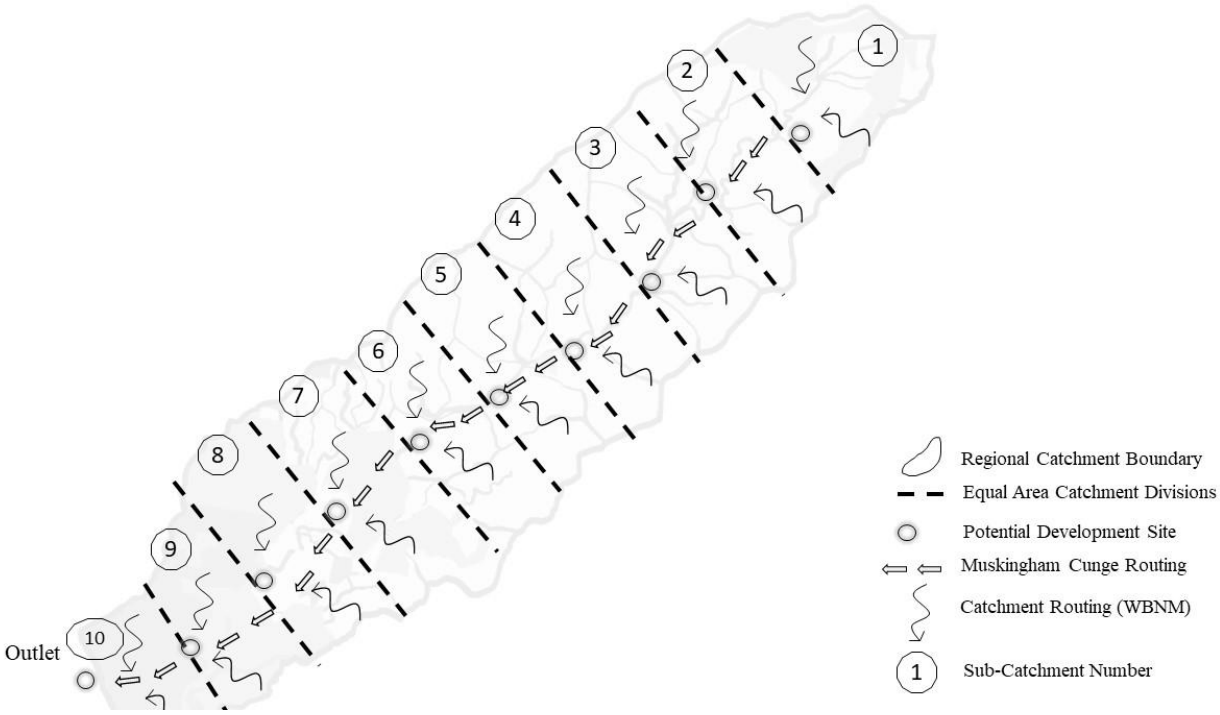

Figure 6-2 – Regional Catchment Conceptualisation Model

6.2.3 Detention Routing

The effect of the OSD was modelled using level pool routing. To describe the outflow of the OSD at various storage depths during the simulation (Q_h), a rating curve was developed in accordance with Ronalds and Zhang (2017):

$$Q_h = \begin{cases} \frac{1}{n} A_w \left(\frac{A_w}{P}\right)^{\frac{2}{3}} S_g^{\frac{1}{2}} + C_w L_w h_{weir}^{\frac{3}{2}} & \langle for\ h \leq D \rangle \\ A_p \sqrt{\frac{2gD(h-h_{DS})}{Lf_D}} + C_w L_w h_{weir}^{\frac{3}{2}} & \langle for\ h > D \rangle \end{cases} \qquad 6\text{-}13$$

The parameters A_p, A_w, P, D, L, n, and S_g are the total area, wetted area, wetted perimeter, diameter, length, roughness, and gradient of the outflow pipe, respectively; C_W, L_W, and h_{weir} are the weir coefficient, length of the weir, and height of the weir above the base of the detention, respectively; h is the depth of water in the detention, h_{DS} is the standing water level downstream of the detention, and g is the gravitational acceleration. In this equation the outflow pipe is located at the base of the detention, which is necessary to ensure it is free draining and does not hold water when there is no storm event. The Darcy–Weisbach friction factor, f_D, is described by Equation 6-14 (Brown 2002), which is dependent upon the Reynolds number, Re.

$$\frac{1}{\sqrt{f_D}} = -2\, log\left(\frac{5.76}{Re^{0.9}}\right) \qquad 6\text{-}14$$

6.2.4 Sample Regional Catchment Details

A sample of ten regional catchments was used for modelling, with their locations shown in Figure 6-3. All are located along the eastern coastline of Australia and within local government areas that mandate the usage of OSD. Care was taken to select catchments that include partial urbanisation without significant major on-line storage systems like water storage reservoirs. The sizes of the catchments vary from 3,000ha to 213,000ha and the imperviousness ranges from 10% to 25%, as measured via aerial photography.

Rainfall depths and durations were obtained from Intensity-Frequency-Duration data for each specific catchment, relating to a 1% Annual Exceedance Probability (AEP) event. The flood level resulting from the critical 1% AEP flood event is the mandated prescriptive for exclusion of development and enforcement of development controls throughout Australia (Cook 2017). All rainfall depth and IL/CL parameters were taken from the Australian Rainfall and Runoff data hub (Babister *et al*. 2016b) and summarised in Table 6-1.

Table 6-1 – Detailed Hydrologic Parameters from the Sample of Regional Catchments

Bureau of Meteorology ID	Regional Catchment Name	Area (ha)	Impervious (%)	6h Rainfall Depth (mm)	12h Rainfall Depth (mm)	24h Rainfall Depth (mm)	Losses (mm)
146012	Currumbin QLD	3000	12	290	415	565	IL = 43, CL = 3.4
142001	Caboolture QLD	9400	18	248	334	447	IL = 43, CL = 3.4
201001	Oxley NSW	21300	15	276	403	557	IL = 20, CL = 2.3
205014	Never Never NSW	5100	10	238	359	523	IL = 45, CL = 4.1
208015	Landsdowne NSW	9600	17	176	237	324	IL = 42, CL = 5.3
141003	Nambour QLD	3800	20	255	376	544	IL = 40, CL = 2.4
141008	Eudlo QLD	6200	10	256	371	533	IL = 25, CL = 2.5
210011	Williams NSW	19400	22	178	249	349	IL = 33, CL = 4.2
143003	Oxley QLD	6000	25	157	205	277	IL = 24, CL = 1.6
143003	Ourimbah NSW	8300	25	173	227	304	IL = 58, CL = 3.5

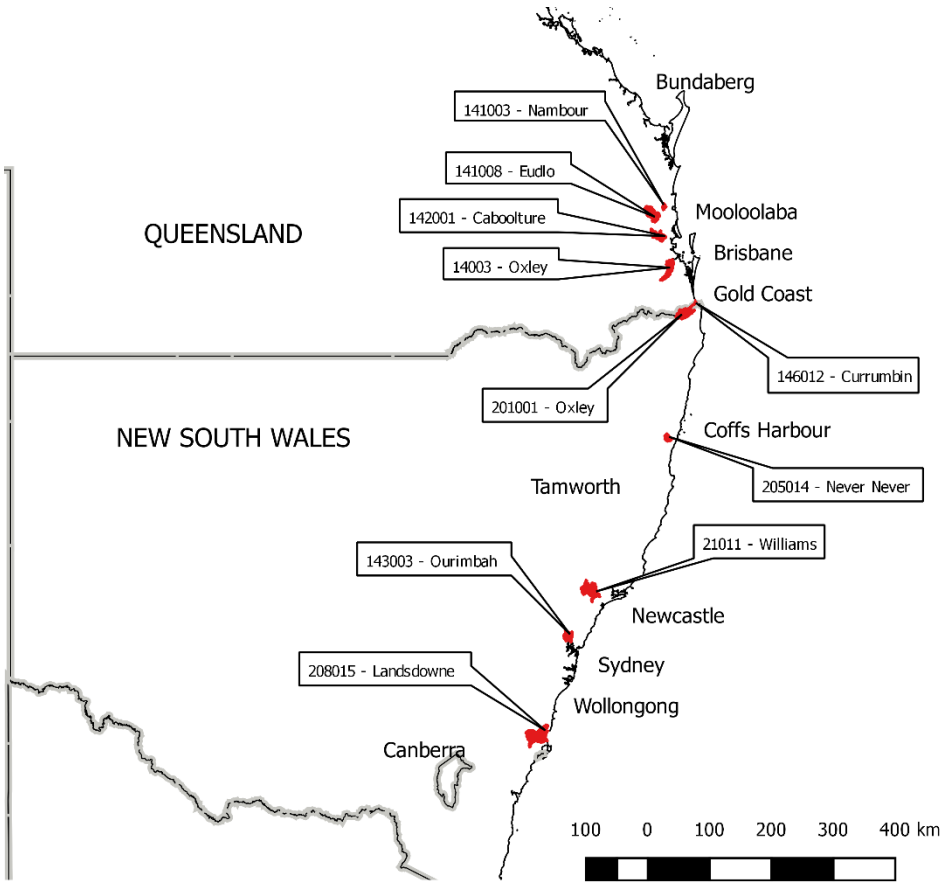

Figure 6-3 – Locations of Sample Regional Catchments

The effect of urbanisation on the hypothetical land parcels was modelled as an increase of imperviousness to 90%. The volume of each OSD was modelled as 300m^3/ha, which was determined from experimentation to be an effective size for reducing post-development peak discharge to pre-development conditions. The OSD volume is also commensurate with the policies of Council's that adopt site storage requirements that target pre-development flow rate conditions, such as Brisbane (Brisbane City Council 2014), Kogarah (Singh *et al*. 2007), Wollongong (Silveri and Rigby 2006) and various others in the Greater Sydney region (Phillips and Yu 2015).

From each simulation of 1,000 random temporal patterns, the effectiveness of OSD in achieving pre-development peak runoff at the development site outlet was found to be in the order of 30-60%. This outcome is commensurate with the expected performance of real-world OSD systems that have been design using event-based techniques (Ronalds *et al*. 2017).

6.2.5 Verification of Model Setup

Each regional catchment used for modelling is gauged, with flood frequency analysis results available from the Australian Rainfall and Runoff Regional Flood Frequency Estimation Model (RFFEM) (Rahman *et al*. 2015). The accuracy of the Monte Carlo simulation has been verified by comparing the model results to the outputs of the RFFEM. In Table 6-2, the mean (μ), lower confidence limit (μ - 2δ) and upper confidence limit (μ + 2δ) from the Monte Carlo model results are shown in comparison to the mean and 95% confidence limits published by the RFFEM.

Table 6-2 – Verification of Monte Carlo Model to RFFEM Outputs (1% AEP)

	Monte Carlo Results (m^3/s)			**RFFEM Results (m^3/s)**		
Catchment	**$\mu + 2\delta$**	**μ**	**$\mu + 2\delta$**	**Lower 95% Confidence**	**Peak Flow**	**Upper 95% Confidence**
Currumbin QLD	168.1	376.7	585.4	278.2	371.4	582.0
Caboolture QLD	514.8	1098.6	1682.5	756.7	1107.7	2145.2
Oxley NSW	953.8	1900.5	2847.2	1444.2	1906.6	2937.7
Landsdowne NSW	289.1	679.2	1069.3	467.49	684.44	1368
Never Never NSW	194.4	359.6	524.9	253.79	308.7	476.3
Nambour QLD	297.8	539.3	780.8	342	530.4	1037.1
Eudlo QLD'	375.2	605.8	836.4	348	608.92	1548.5
Williams NSW	787.4	1600.1	2412.8	1188.9	1624.76	2583.7
Oxley QLD	172.3	414.1	655.9	299.32	397.87	1177.2
Ourimbah NSW	228.9	522.2	815.4	246.3	508.9	1689.9

6.3 Results and Discussions

For all the regional catchments modelled in this study, recurring patterns for the impact of development and OSD on regional catchment peak runoff were observed.

The factor of impact to peak runoff as a result of development alone (F_{dev}) was found to be an increase when the land parcel is in the upper portions of the catchment. As the land parcel gets nearer to the outlet of the catchment F_{dev} is reduced to negative, indicating decreased regional catchment runoff. The factor of impact on regional outflow because of OSD (F_{det}) was found to follow an inverse relationship to F_{dev}, resulting in decreases to regional outflow when the land parcel is located in the upper portions of the catchment and an increased F_{det}

at the lower portions of the catchment. Figure 6-4 diagrammatically shows the recurring trendlines of F_{dev} and F_{det} that were observed with respect to land parcel location within the regional catchment.

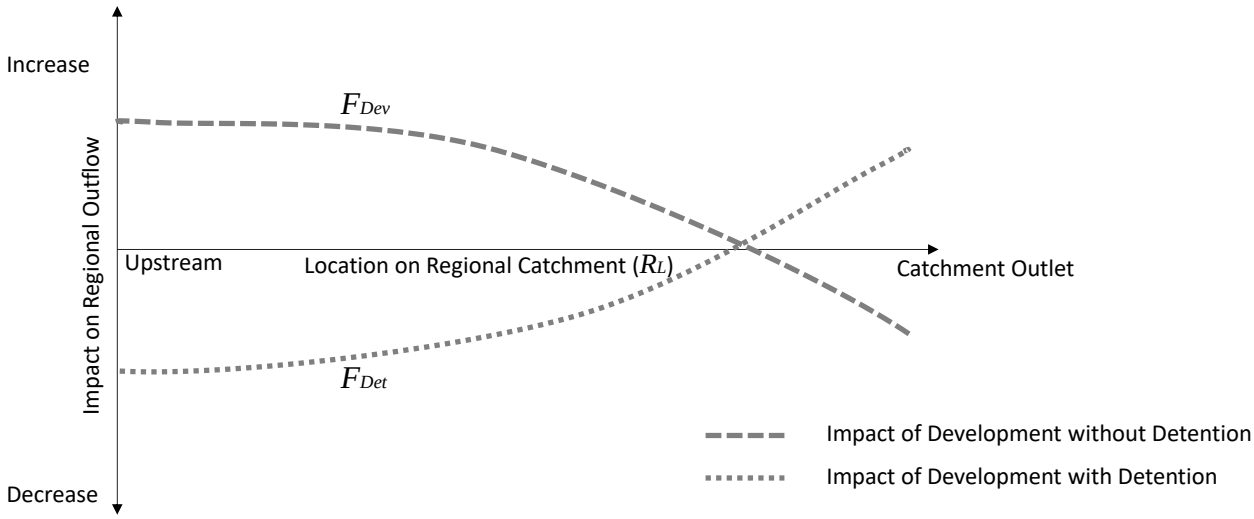

Figure 6-4 – Diagrammatic of Observed Response to Regional Catchment Peak Flow for Development with and without OSD at Varying Land Parcel Locations

6.3.1 Factor of Impact Resulting from Development (F_{dev})

Figure 6-4 shows a second order polynomial trend that repeatedly occurred during the assessment of the regional catchments investigated in this study. The general form of F_{dev} is provided in Equation 6-15, where R_L identifies the land parcels location in the regional catchment based on the distance of the site from the regional catchment outlet (L_{site}) and the total regional catchments flow path length ($L_{regional}$).

$$F_{dev} = aR_L{}^2 + bR_L + c \qquad 6\text{-}15$$

$$R_L = \frac{L_{site}}{L_{regional}} \qquad 6\text{-}16$$

The coefficients *a*, *b* and *c* were found to vary in a linear relationship with respect to the ratio of the development size to the regional catchment size (R_C). Table 6-3 shows the linear gradients of *a*, *b*, and *c* with respect to R_C, as well as the coefficient of determination (R^2), indicating the strength of linearity.

Table 6-3 – Linear Relationship of F_{dev} Polynomial Coefficients to R_C

	a		***b***		***c***	
Catchment	**Linear Gradient**	**R^2**	**Linear Gradient**	**R^2**	**Linear Gradient**	**R^2**
Currumbin QLD	-0.178	0.956	-0.150	0.997	0.240	0.996
Caboolture QLD	-0.091	0.996	-0.100	0.920	0.157	0.994
Oxley NSW	-0.093	0.998	-0.114	0.980	0.168	0.995
Landsdowne NSW	-0.075	0.973	-0.196	0.999	0.332	0.999
Never Never NSW	-0.141	0.944	-0.132	0.914	0.300	0.944
Nambour QLD	-0.025	0.983	-0.028	0.946	0.104	0.986
Eudlo QLD	-0.173	0.986	-0.090	0.820	0.215	0.989
Williams NSW	-0.017	0.677	-0.135	0.972	0.187	0.998
Oxley QLD	-0.168	0.996	-0.072	0.986	0.206	0.995
Ourimbah NSW	-0.136	0.990	-0.073	0.974	0.258	0.998

To establish a universal solution applicable to catchments that are not included in the study, Figure 6-5 below provides the results of curve fitting exercises for each coefficient, considering all data points generated by the study.

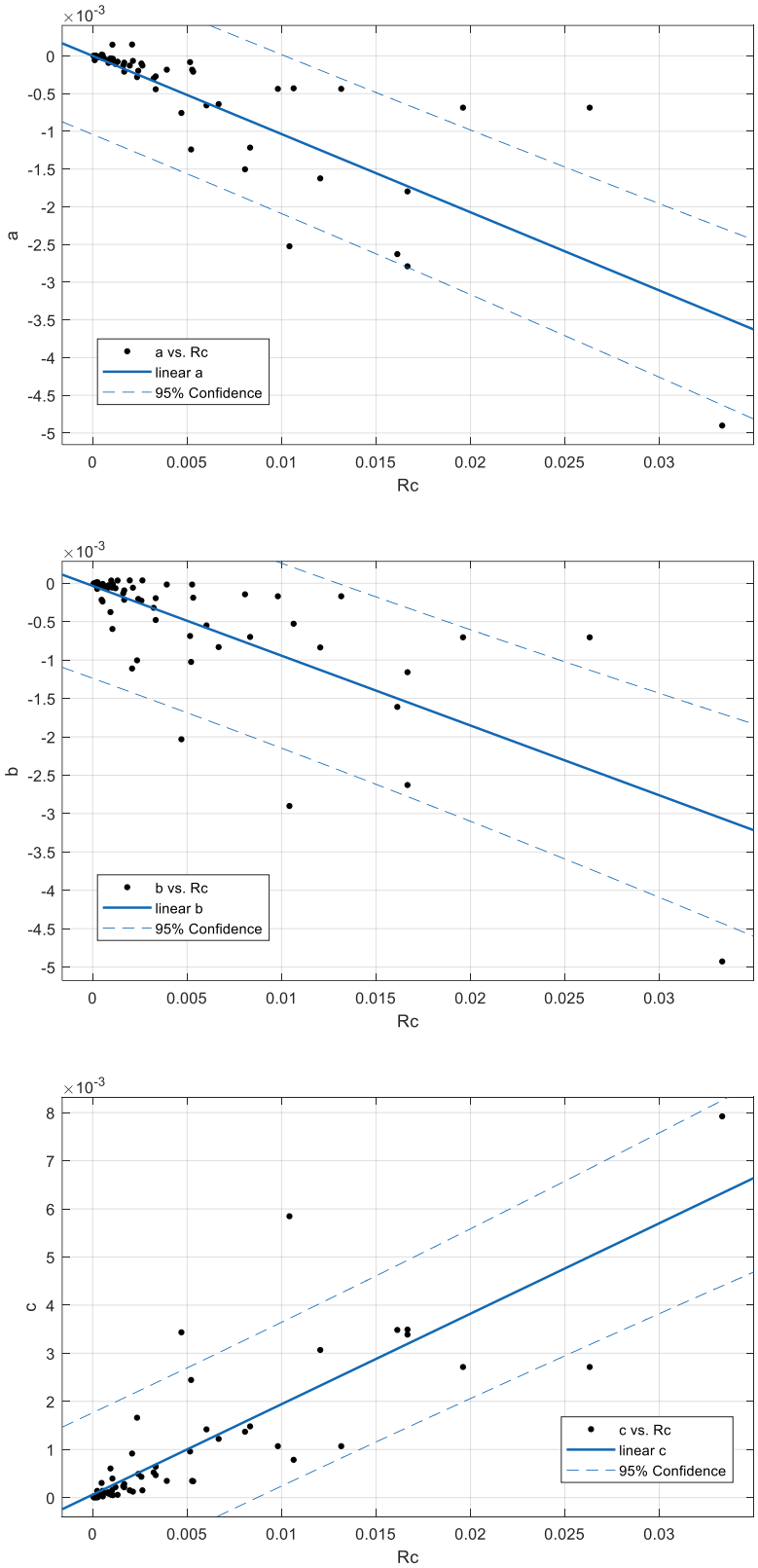

Figure 6-5 – Linear Regression of Coefficients a, b and c with Respect to R_C

Based on the regression analysis of coefficients a, b and c with respect to R_C, the empirical equation form for each coefficient is defined as follows:

$$a = -0.1R_c \qquad 6\text{-}17$$

$$b = -0.1R_c \qquad 6\text{-}18$$

$$c = 0.15R_c \qquad 6\text{-}19$$

Combining Equation 6-15 with the linear forms of *a*, *b* and *c* yields Equation 6-20, where F_{dev} is a function of R_C and R_L:

$$F_{dev} = R_c\left[-0.1{R_L}^2 - 0.1R_L + 0.15\right] \qquad 6\text{-}20$$

6.3.2 Factor of Impact Resulting from Development with OSD (F_{det})

The general form of F_{det} is provided in Equation 6- 21, which was observed to follow an inverse relationship to F_{dev} as shown in Figure 6-4.

$$F_{det} = i{R_L}^2 + jR_L + k \qquad 6\text{-}21$$

The coefficients *i*, *j* and *k* were also found to vary with a linear relationship respecting R_C for each regional catchment. Table 6-4 shows the results for each catchment individually and Figure 6-6 shows the results of the curve fitting exercises for each coefficient considering all data points generated by the study.

Table 6-4 – Linear Relationship of F_{det} Polynomial Coefficients to R_C

	i		***j***		***k***	
Catchment	**Linear Gradient**	**R^2**	**Linear Gradient**	**R^2**	**Linear Gradient**	**R^2**
Currumbin QLD	0.408	0.999	-0.248	0.972	-0.081	0.884
Caboolture QLD	0.439	0.999	-0.237	0.986	-0.155	0.982
Oxley NSW	0.177	0.996	-0.066	0.938	-0.019	0.413
Landsdowne NSW	0.453	0.980	-0.377	0.977	0.091	0.991
Never Never NSW	0.292	0.974	-0.272	0.985	0.112	0.960
Nambour QLD'	0.066	0.998	-0.080	0.249	-0.168	0.982
Eudlo QLD'	0.172	0.995	-0.034	0.883	-0.024	0.596
Williams NSW	0.413	0.977	-0.245	0.955	0.080	0.920
Oxley QLD	0.226	0.964	-0.156	0.878	-0.106	0.108
Ourimbah NSW	0.267	0.964	-0.210	0.954	0.018	0.416

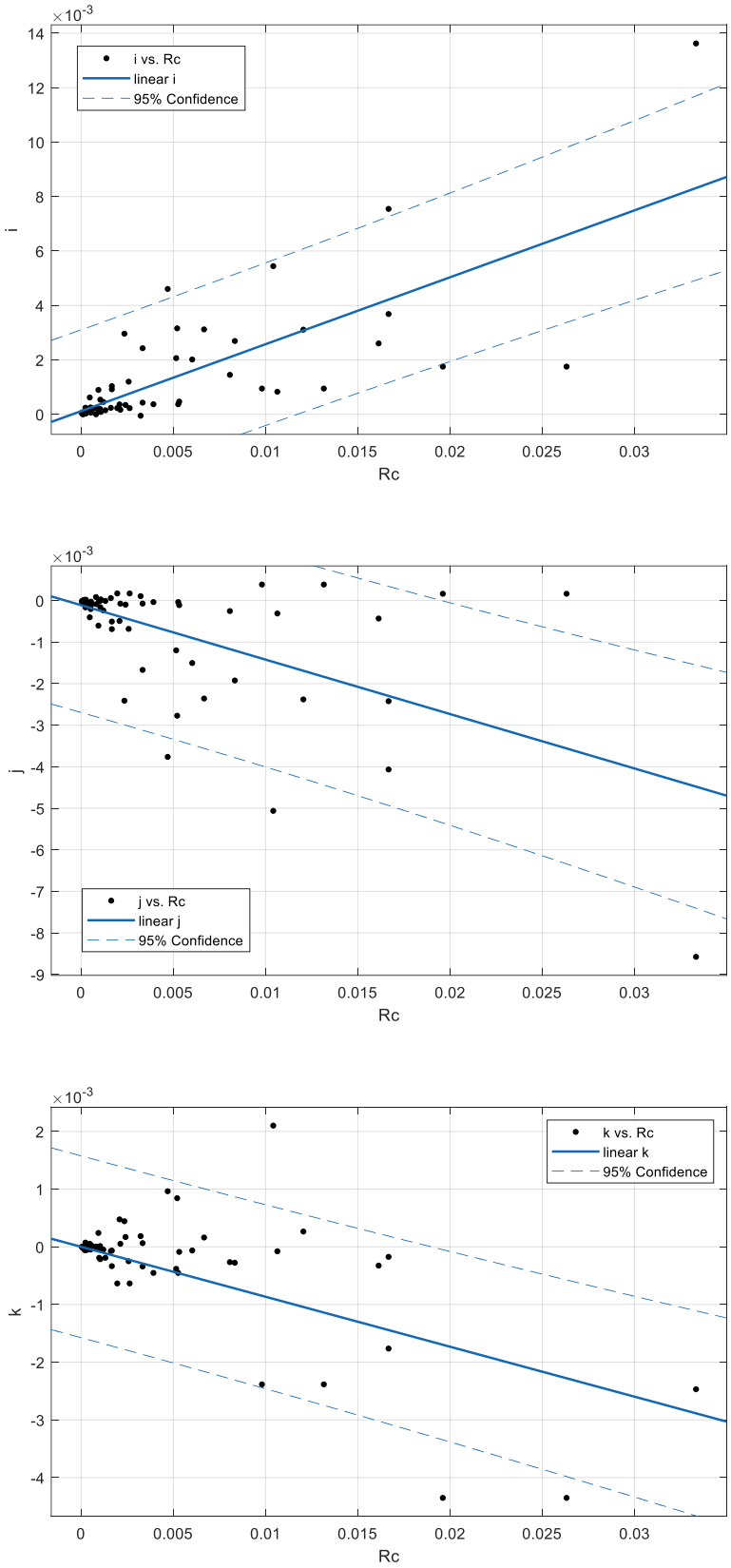

Figure 6-6 – Linear Regression of Coefficients i, j and k with Respect to R_C

Based on the regression analysis of coefficients *i*, *j* and *k* with respect to R_C, the empirical equation form for each coefficient is defined as follows:

$$i = 0.3Rc \qquad 6\text{-}22$$

$$j = -0.15Rc \qquad 6\text{-}23$$

$$k = -0.1Rc \qquad 6\text{-}24$$

Combining Equation 6-21 with the linear forms of *i, j* and *k* yields Equation 6-25, where F_{det} is a function of R_C and R_L:

$$F_{det} = R_c\left[0.3{R_L}^2 - 0.15R_L - 0.1\right] \qquad 6\text{-}25$$

Equation 6-20 and Equation 6-25 are the key findings of this study that may be used in future applications to predict the mean impact on regional peak runoff that is the result of urbanisation alone, and with OSD respectively.

6.4 Verification and Example Case Study - Tallebudgera Creek Catchment

The Tallebudgera Creek catchment outlets to the Pacific Ocean via a narrow river mouth at Burleigh Heads on the Gold Coast, Queensland, Australia. Most of the recent urbanisation is in the lower portions of the catchment surrounding the primary streamline, with rapidly expanding and intensifying beachside and estuarine suburbs including Burleigh Waters and Elenora located in the lower portions of the catchment area. At the river gauging station located in the lower, urbanised portion of the catchment (BoM ID: 146007) the upstream catchment area is 5,700ha. Figure 6-7 provides a diagrammatic of the catchments shape, land use and gauging station location.

This example of a typical regional catchment has been used to demonstrate the recurring patterns observed through all ten of the sample catchments, and to demonstrate the ability of Equation 6-20 and Equation 6-25 to predict F_{dev} and F_{det}.

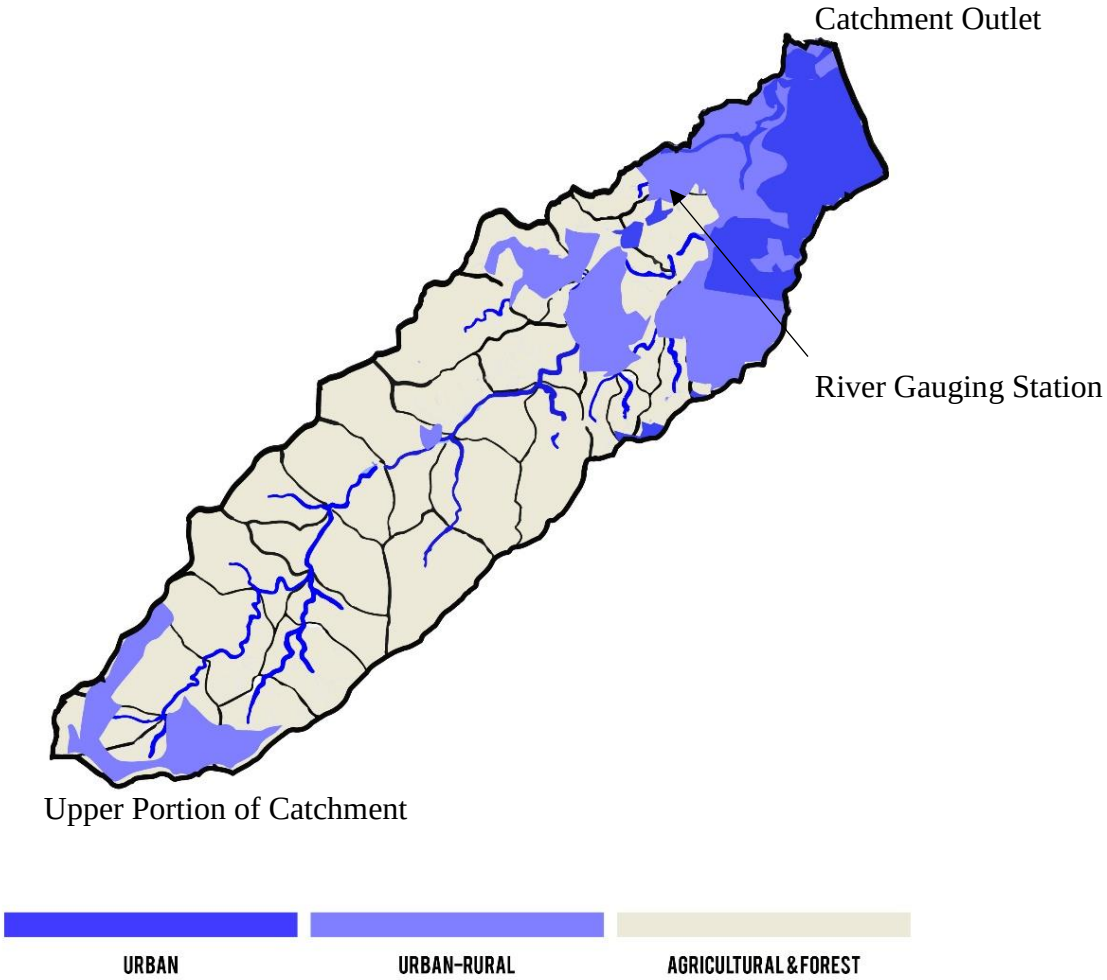

Figure 6-7 – Tallebudgera Creek Catchment Shape and Land Usage

To validate the accuracy of the 'status quo' model used for simulation of the Tallebudgera Creek, the Monte Carlo simulation results for 1,000 temporal patterns modelling the 1% AEP rainfall event in the existing case scenario are shown in Figure 6-8. The full spread of hydrograph results are shown, with the mean identified in solid linework. The mean peak runoff from the Monte Carlo modelling was calculated as 731.7m^3/s, which compares well to the RFFEM outputs of 732.3m^3/s.

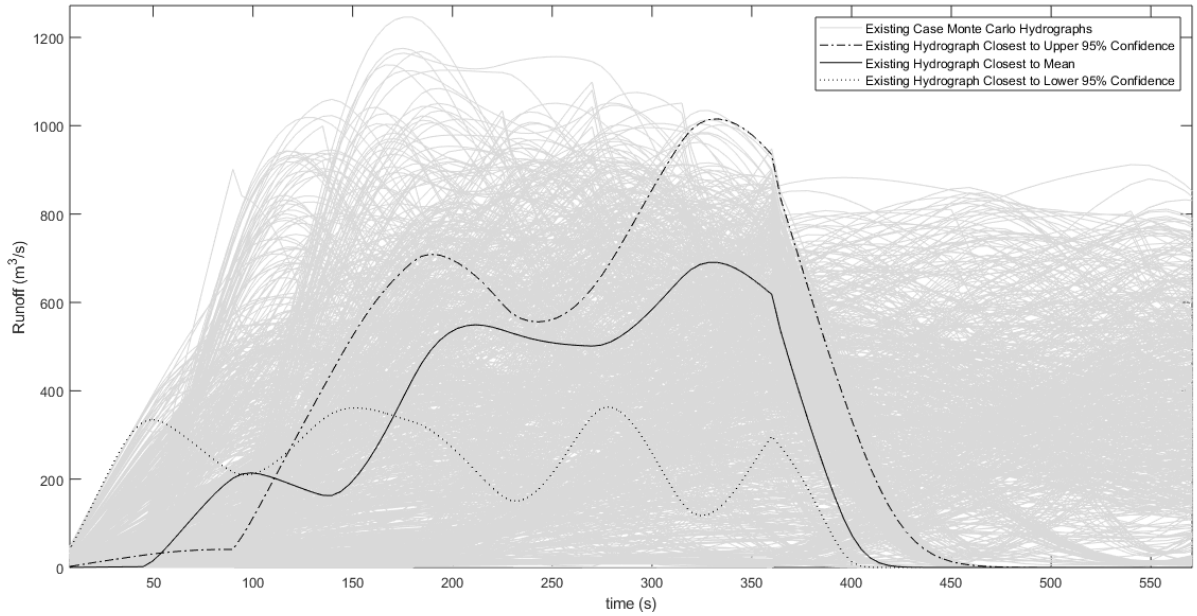

Figure 6-8 – Tallebudgera Creek Monte Carlo Simulation of Regional Catchment Hydrographs

A 10ha urban land parcel has been hypothesised. This would be representative of a 50-100 lot townhouse project or a large commercial development with associated roadworks and carparks creating impervious surfaces.

The full results of the Monte Carlo simulation are presented in Figure 6-9. A grey line is presented for each model simulation, showing the percentage of impact on regional catchment peak flow that results from development with, and without OSD at incremental locations within the catchment. The horizontal axis represents the location of the development site within the regional catchment, with the outlet on the right-hand-side. 1,000 grey lines are shown for the scenario of development alone and 1,000 grey lines are shown for the scenario of development with OSD. A dashed line is shown to represent the mean of the Monte Carlo simulation results for each scenario.

The calculated prediction of the mean impacts using Equation 6-20 and Equation 6-25 are also shown in solid lines. The calculated F_{dev} and F_{det} from the equations are shown to achieve a close fit to the mean of the Monte Carlo model results, even though this catchment was not considered in the regression or determination of the equations.

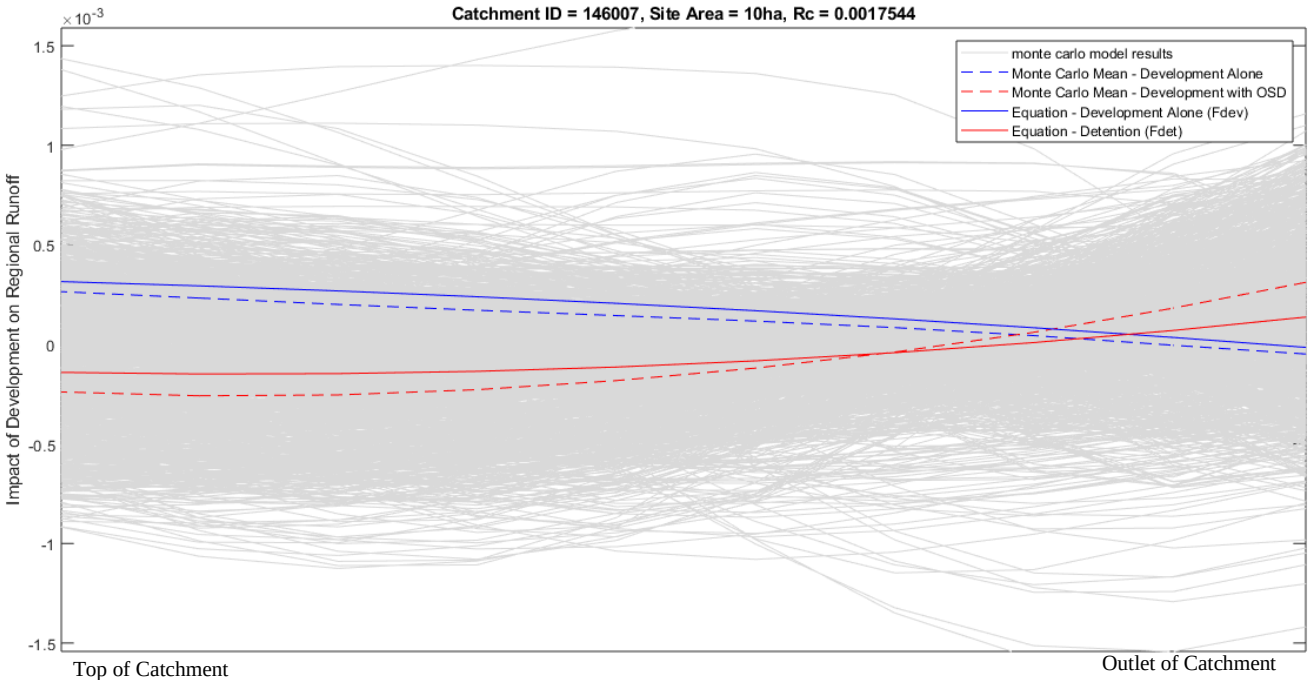

Figure 6-9 – Comparison of Monte Carlo and Equation Results for Development Impacts in the Tallebudgera Creek Catchment

6.4.1 Specific Test Location Analysis

Two specific locations of land parcels have been focussed upon for detailed analysis of the model results, one in the upper 20% of the catchment (R_L = 0.2) and another in the lowest 10% of the catchment (R_L = 0.1). Table 6-5 summarises a comparison of the results from using Equation 6-20 and Equation 6-25 for predicting F_{dev} and F_{det} respectively against the Monte Carlo model results at each location.

Table 6-5 – Comparison of Calculated F_{dev} and F_{det} to Monte Carlo Results

	Upper Catchment		**Lower Catchment**	
	% Impact	**m³/s Impact**	**% Impact**	**m³/s Impact**
Calculated F_{dev} from Equation 6-20	+0.022	+0.161	-0.004	-0.027
Mean Monte Carlo F_{dev} (μ)	+0.023	+0.168	-0.004	-0.027
Upper Confidence Monte Carlo F_{dev} ($\mu + \delta$)	+0.041	+0.300	+0.020	+0.150
Lower Confidence Monte Carlo F_{dev} ($\mu - \delta$)	+0.005	+0.037	-0.020	-0.150
Calculated F_{det} from Equation 6-25	-0.021	-0.154	+0.002	+0.015
Mean Monte Carlo F_{det} (μ)	-0.022	-0.161	+0.005	+0.037
Upper Confidence Monte Carlo F_{det} ($\mu + \delta$)	-0.039	-0.285	+0.022	+0.161
Lower Confidence Monte Carlo F_{det} ($\mu - \delta$)	-0.005	-0.037	-0.010	-0.073

The results in Table 6-5 show a validation of Equation 6-20 and Equation 6-25 by comparison to the results of the Monte Carlo simulations.

Further review of the simulation results was undertaken to assess the number of occurrences of increases or decreases to the regional catchment peak flow. In the upper portion of the catchment, 990 of the 1,000 F_{dev} simulations showed an increase in regional catchment peak runoff resulting from urbanisation of the land parcel. When OSD was modelled however, the F_{det} results reduced to only 71 of the 1,000 simulations. In the lower portion of the catchment, 576 of the 1,000 F_{dev} simulations showed an increase in regional catchment peak runoff, which was increased to 718 of the 1,000 F_{dev} results when OSD was modelled.

6.5 Summary

A model was developed for Monte Carlo simulation of 1,000 randomly generated temporal patterns on ten regional catchments with seven differing land parcel sizes located throughout each catchment. A summary of the findings is as follows:

- Monte Carlo simulation of temporal patterns with design rainfall depth and duration has been shown to generate comparable results to regional flood frequency analysis for the mean and 95% confidence limits of peak flow magnitude;
- Under the dominating influence of temporal pattern uncertainty, peak runoff from all of the regional catchments modelled in this study behaved similarly in response to urbanisation and OSD at varying locations in the regional catchment;
- The mean impact of urbanisation (F_{dev}) and urbanisation with OSD (F_{det}) on regional catchment peak runoff were both found to be dependent upon the location of the land parcel (R_L) and the ratio of land parcel to regional catchment area (R_C);
- A regression analysis of the model results was used to develop equations for the prediction of F_{dev} and F_{det}. The equations have been tested and verified using an eleventh regional catchment;
- The modelling has shown that OSD can reduce a 99% chance of increased runoff to less than 8% when a land parcel is in the upper reaches of a catchment. In the lower portions

of the same catchment however, the same OSD has a 72% change of increasing runoff, compared to a 58% chance without;

- This study has highlighted the importance of considering land parcel location in the determination of need for OSD as part of urban land development projects; and
- This study has highlighted temporal pattern uncertainty as a major concern for the design of, and reliance on OSD for the protection of regional catchment hydrology.

7. Discussion

7.1 Industry Impact and Significance

This research project should be of particular interest to civil engineering practitioners and policy-makers for two reasons.

Firstly, the research has contributed to the growing awareness of the importance of variable rainfall probability as an input parameter for stormwater detention design. There have been several studies that have shown the importance of considering variable rainfall probability for the accurate prediction of flooding (Bhuiyan *et al.* 2010, Caballero and Rahman 2014a, 2014b, Kottegoda *et al.* 2014, Loveridge and Rahman 2014, Müller and Haberlandt 2018, Rahman *et al.* 2002), however the specific impact on stormwater detention design and performance has not been analysed using the methodology presented in this thesis until now. The findings of this research should highlight great concerns regarding the performance of on-site detention in response to variable rainfall patterns. It is hoped that the findings will either prompt or accelerate movements toward usage of ensemble, Monte Carlo or continuous simulation of multiple temporal patterns for detention design.

As an alternative outcome and in the perpetuation of design with limited temporal pattern methodologies, this research has highlighted the inability to rely upon model results and the need to consider uncertainties and confidence limits. The limitations of reliance on model results using either simple event or ensemble methodologies has been quantified. This is expected to serve as a guide for designers and decision makers to better inform the public on the expected outcomes of detention performance and the potential risks associated with failure to achieve peak flow management objectives.

Secondly, the research has contributed to the long-standing awareness of the adverse impact that stormwater detention can have on regional hydrology. Whilst many studies have warned of the regional effect and developed regional strategies to assign and optimise the location of detention in the regional catchment (Booth *et al.* 2002, Duan *et al.* 2016, Emerson *et al.* 2005, McEnery and Morris 2011, Saunders 2008), this research has provided a practical tool to calculate the impact of a detention system on the downstream hydrology.

Whilst known to carry potentially significant shortcomings, micro-management policies for stormwater detention are expected to prevail around the world as an attempted solution to the adverse impacts of urbanisation. There will inevitably be limitations on public resources that do not always allow for regional scale strategies that involve either distributed detention

or at-source detention mandates for site storage rates or permissible discharge rates that consider the regional hydrology. This research provides a practical solution to this issue by means of a simple tool to calculate the regional impact of a detention system at a specific downstream location. Designers or assessment authorities can use the equations presented in this research to consider the benefits of a detention system at known areas of concern within the regional catchment. This is expected to inform decisions regarding the installation or avoidance of detention in favour of infrastructure upgrades.

Setting public policy for stormwater detention is complex and requires careful consideration of the hydrologic benefits, public opinion and resources required (Fang 2017, Lees and Lynch 1992, O'Loughlin *et al.* 1995, van de Sterren *et al.* 2009). The unfortunate reality is that no two development sites are the same and each must consider its impact on the regional catchment hydrology independently. The tools presented in this research are easily appreciable as beneficial for the regional hydrology and easily able to be checked in a design or assessment process, making them practical additions to public policies for stormwater detention with micro-management objectives.

7.2 Climate change and uncertainty

Recently, climate variability and climate change have been recognized as having a profound impact on the temporal patterns of rainfall. With climate change, each fraction of the rainfall temporal pattern is expected to scale with temperature in a different way and with an increase in variability (Mamo 2015). Predictions are that warmer temperatures are forcing the intensification of temporal patterns (Wasko and Sharma 2015) and that peak fractions of temporal patterns are expected to become peakier and non-peak fractions are expected to become less peaky (Fadhel *et al.* 2018). Past recordings of temporal patterns are therefore likely to become inappropriate representations of future design events.

Hettiarachchi *et al.* (2018) has predicted that changes in the projected temporal patterns alone can increase the risk of flood magnitude up to 35%, with the cumulative impacts of temperature rise on temporal patterns and the storm volume increasing flood risk from 10 to 170%. The temporal patterns are also expected to vary for all durations of rainfall, from small and frequent storm events of as little as 5-minute durations up to yearly patterns used for water balance modelling (Nguyen *et al.* 2010).

This research has contributed to an awareness of the impact that variable rainfall probability has on the performance of detention systems, which is going to be a key consideration for design in the future of climate change and uncertainty.

The usage of the multiplicative cascade technique for the disaggregation of a design rainfall depth to create random temporal patterns is a way to make due consideration for the potential variability of temporal patterns that are expected to arise from climate change. As there is no certainty surrounding the shape of temporal patterns that the future may expect, the usage of statistical results from the sampling of thousands of random temporal patterns is an effective means to provide a working solution that considers the potential effect of future climate change. The equations that are presented in this research for calculating the impact of detention on regional catchment hydrology are therefore applicable to a future of climate change and temporal pattern uncertainty.

7.3 Future research

7.3.1 Hysteretic Rating Curves

In Chapter 4 a rating curve is presented for modelling the outflow of a detention system, comprising a combination of pipe and weir flow. The rating curve is generally formed by plotting the tank's discharge against the surface elevation (or stage) of the water in accordance with the traditional practices (Linsley *et al.* 1985). A Reynolds number is required in the process to define the flow of the outlet pipe. As a general rule and in alignment with current best practice, if the flow within the pipe is laminar it is assigned a number of less than 2000, whereas if the flow is fully turbulent, it may be assigned a value of 4000 or greater (Mays 2010). The selection of the Reynolds number is limited to a single value, resulting in a steady flow curve whose shape is strongly dependent upon the assumption made regarding turbulence.

In the actual physical process of a detention system filling and emptying during a storm event, it is expected that the turbulence will vary over time and the observed rating curve would be looped, or hysteretic. Figure 7-1 by Chow (1988) provides a description of a steady rating curve, as proposed in Chapter 4, versus a dynamic or looped rating curve experiencing hysteresis, as it would be expected in reality. In physical experimentations of large ponds with piped outlet systems, there is also evidence of a backwater effect that is the result of waves splitting and then falling back onto the water body (Fread, 1973). In the case of a tanked or small detention system as used in the case studies of this research, this internal wave effect is also expected, likely resulting in multiple varying shape or spiralling looped rating curves.

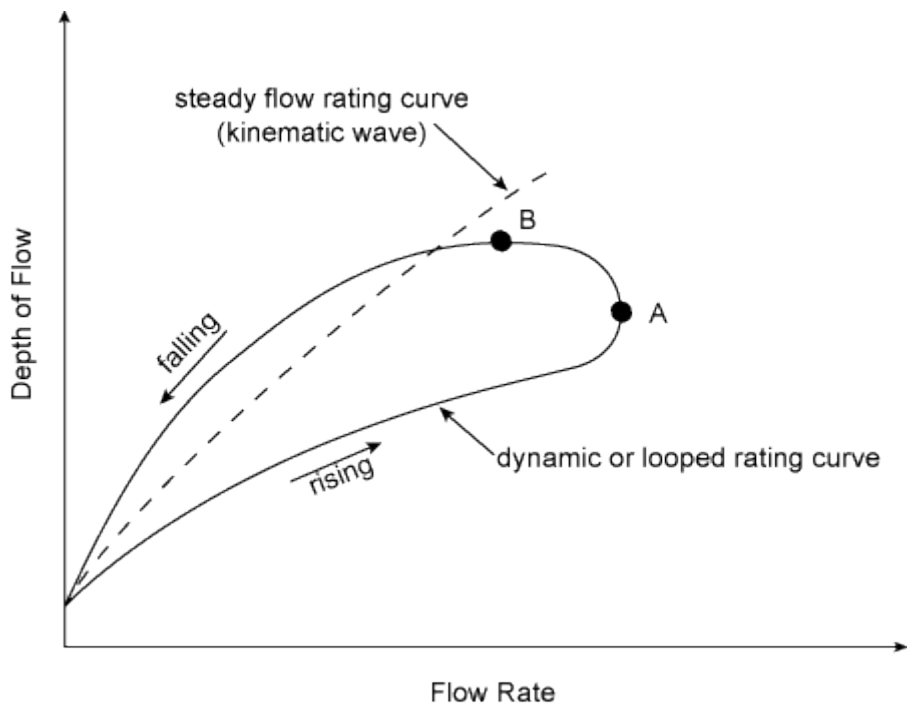

Figure 7-1 – Uniform and Looped Rating Curves (Chow 1988)

As part of this research project an investigation into the usage of looped rating curves for modelling detention discharge was considered. None of the standard software packages that are used for detention modelling are programmed with the ability to model looped rating curves. Assessing the accuracy of modelling results and the resulting performance of detention systems in response to the potential for looped rating curves was considered to be a significant research objective.

A pilot project was undertaken involving participation by a student as part of an undergraduate thesis project (Mehta 2017). A physical experimentation was carried out involving a scaled model of a detention tank with varying sized outlets and a methodology of observing and recording rating curve profiles. The results conclusively demonstrated the existence of looped rating curves. The significance of the backwater effect was also demonstrated by observation of varying sized and spiralling loops, resulting from changing energy slope, air pockets and aerated water in the tank. It was however impossible to use regression to identify any reliable patterns in the results that could be used for any practical means, largely due to equipment and scope limitations.

Further research is recommended into the use of hysteretic rating curves as a way to improve the accuracy of detention modelling and reliability of results. Whilst significant, the outcomes of this further research objective were not considered to align with the overall objectives of this thesis.

7.3.2 Ensemble Uncertainty

In the ensemble analysis presented in Chapter 5, the methodology of selecting the pre- and post-development peak flow rate to be used in the comparative analysis involved the selection of the hydrograph peak that was the closest to the median of the ten results. This is the standard industry practice in Australia as recommended by Ball *et al.* (2016).

This methodology yields a significant amount of prediction uncertainty from the discarded results that are above and below the closest-to-median. It is assumed that the closest-to-median result is representative of the total data set, which is not entirely accurate. The process does not assess the individual potential for failure to meet objectives that may result from any of the other nine hydrographs that are generated from the ensemble of temporal patterns.

Furthermore, the objective of the detention system is often to reduce peak runoff by quantities that are close to the range of peak results from either the pre- or post-development ensemble of hydrographs.

Further research is recommended into the quantification of prediction uncertainty resulting from the closes-to-median approach to ensemble analysis. A comparison of alternative approaches such as the selection of maximum, minimum or individual assessment of the complete set would be beneficial in this process.

7.3.3 Regionalisation of Impact Evaluation

In Chapter 6, empirical formulas are presented for calculation of the mean impact of urbanisation and detention on the regional hydrology at downstream locations.

The formulas have been developed using regression of results obtained from ten catchments along the eastern coastline of Australia. The usage of Monte Carlo rainfall patterns generated using the multiplicative cascading technique is a deliberately adopted way to account for random variance of temporal patterns probable at any geometric location. This is however untested outside of the sample of the ten regional catchments.

Further research into the impacts of urbanisation and detention on the regional hydrology of additional catchments in differing spatial areas could lead to further refinement of the equations and improved reliability.

8. Conclusions

The thesis has presented and responded to three significant research questions:

1. What are the real impacts of micro-management style stormwater detention on regional catchment peak discharge?
2. Can we practically predict the peak flow impact of an urban development and its detention system at a specific downstream location, without conducting a full catchment analysis?
3. Can the answer to these questions suitably account for variable rainfall probability?

By responding to these questions this thesis has provided a significant contribution to knowledge in the areas of hydrologic modelling, the effect of stormwater detention on regional hydrology, and variable rainfall probability.

8.1 Summary of Research Achievements and Significance

The incremental development of a novel regional hydrologic model has been presented over three chapters which are also stand-alone journal articles. The model is capable of assessing the impacts of urbanisation and detention at varying locations within a regional catchment using an original combination of advanced hydrologic theories. The model has also been used to develop a unique system of equations that is capable of predicting the impact of future urbanisation and detention at specific locations within a regional catchment.

In parallel with the development of the regional hydrologic model, significant contribution has been made toward the awareness of the importance of variable rainfall probability in the design and performance of stormwater detention. The shortfalls in performance of stormwater detention using either singular or ensembles of ten temporal patterns have been quantified using a novel continuous simulation model. This has definitively shown the importance of temporal patterns as a key determinant in the successful performance of a detention system. In response to this finding the practical application of a Monte Carlo simulation technique for the generation of multiple random temporal patterns has been described and demonstrated for future hydrologic modelling.

8.2 Summary of Responses to Research Questions

A specific response to each of the research questions is summarised as follows:

1. The impact of micro-management style stormwater detention on regional catchment peak discharge can be an unwanted increase in flow when the urbanised land parcel is located in the lower portions of the regional catchment. This question has been answered using a combination of literature from previous studies, modelling using packaged software products, and modelling using a new and original regional hydrologic model.
2. The mean impact of urbanisation and detention at a specific downstream location within a regional catchment is predictable. This question has been answered using a regression analysis of results generated from the new and original regional hydrologic model. A unique system of equations has been provided to make these predictions that has been tested and verified.
3. Variable rainfall probability is considered to be one of the greatest threats to the successful design and reliance upon performance of stormwater detention. The response to this question is delivered in two parts. Firstly, the impact of variable rainfall probability on micro-management style stormwater detention at the localised outlet of a land parcel has been described, by quantifying the likelihood of failure to achieve pre-development peak flow objectives when either singular or ensemble temporal pattern methods are adopted. Secondly, the ability to manage issues with variable rainfall probability by using Monte Carlo techniques has been demonstrated and incorporated into the new regional hydrologic model and new system of equations.

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