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Impacts of Land Developments and Land Use Changes on Urban Stormwater Management

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DECLARATION

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

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ABSTRACT

Impacts of land developments and land use changes on urban stormwater management

With the rapid urbanization happening around the world, the nature of the natural hydrological cycle has been changed and it causes many adverse effects like urban flooding, erosion and degradation of water quality in urban areas. Due to the increasing population, urbanization will continue rapidly and this increases impervious lands which generate more runoff. Anthropogenic climate change has influenced the strength of storm events and reduced the recurrent intervals. Current urban stormwater management systems are becoming increasingly lacking with rapidly increasing demands and climatic effects. Groundwater has been found as a key factor in creating inadequacy in urban drainage to carry stormwater runoff in catchments having a shallow groundwater table. Water sensitive urban design (WSUD) and modifications to urban stormwater management systems (USWMSs) according to the best management practices (BMP) should be implemented after systematic analysis to overcome the situation.

This study has focused on assessing urban land development activities and changing patterns of land use in urban areas as the main anthropogenic stress on urban hydrology. In addition, the adaptation to natural phenomenon such as climate change has been studied. A numerical hydrological model was used to analyse the behaviour of catchments and their characteristics. Urban flood identification and prevention was one of the major concerns of this study. Several urban stormwater drainage systems have been assessed under three case studies.

The stormwater drainage system of Canning Vale Central catchment, which is one of the urban catchments in Western Australia, has been assessed by using numerical modelling in case study number one. The model was developed by using existing mapped data and data collected from an ongoing telemetric observation system and several field visits. Surface runoff has been routed by using different modelling techniques such as hydrological surface runoff and two-dimensional (2D) surface runoff modelling. Groundwater has been treated as a critical issue during the modelling. The effects of land use changes and their sensitivity to the USWMS have

been assessed. Necessary recommendations to improve the USWMS and mitigate localised flood issues have been given. Flood vulnerability maps have been developed to identify the critical areas where there is the potential to be flooded under different Average Recurrent Interval (ARI) events. These flood vulnerability maps will be used by the local authorities to develop recommendations and guidelines for future developments of infrastructure during land development and subdivision works.

The urban ungauged catchment of Victoria Park in Western Australia has been assessed by using a 2D surface runoff routing model. The catchment has built flood storage areas (stormwater basins) and the inadequacy of them in protecting against recent storm events has caused local concern. The area has been developed rapidly in recent decades and land use has been changed to more impervious surfaces than was expected at the time the basins were designed. These changes to the land use—together with anthropogenic climate change—has caused runoff from rapid storms to exceed the basin top water level. The catchment's existing stormwater basins' capacities were assessed against different ARI events during case study number two. Flood vulnerability maps and water level contours have been developed to identify the possible inundations and flood depths of basins and surrounding areas.

The proposed urban development of Wellard Residential Development site has been modelled to analyse the USWMS according to the WSUD by using BMPs. Major issues those distinguish the pre-development hydrology from post-development hydrology have been identified as the land use changes and changes to the natural stormwater flow paths by urban stormwater drainage system are made. The use of BMPs to overcome the situation and the modelling of BMPs within urban stormwater management model has been studied, analysed and discussed. Current urban stormwater management guidelines and strategies have been reviewed during the modelling process. Suitable adaptations based on BMPs have been recommended and the results will support the stormwater management section of the urban water management plan (UWMP) which will ultimately support the environmental sustainability of the development.

The overall study is based on hydrological modelling of different USWMSs and urban hydrology. Land use change was considered as the main anthropogenic stress upon urban hydrological catchments. Factors such as encountering groundwater in stormwater drainage have been analysed to support the study. Recommendations based on WSUD and BMPs have been given to mitigate the adverse effects of urban land use changes to urban stormwater management.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF ABBREVIATIONS

Average Annual Maximum Groundwater Levels (AAMGL)
Average Recurrent Interval (ARI)
Best Management Practices (BMP)
Digital Elevation Model (DEM)
Digital Terrain Model (DTM)
District Water Management Strategy (DWMS)
Environmental Protection Agency (EPA)
General Circulation Models (GCMS)
Generalized Likelihood Uncertainty Estimation (GLUE)
Geographic Information System (GIS)
Graphical User Interface (GUI)
Intensity Frequency Duration (IFD)
Light Detection and Ranging (LiDAR)
Low Impact Development (LID)
Mean Squared Error (MSE)
Monte Carlo Markov Chain (MCMC)
Multiple User Corridor (MUC)

Nash–Sutcliffe Efficiency (Nse)

One-Dimensional (1D)

Public Open Spaces (Pos)

Regional Climate Models (RCMS)

Regional Water Management Strategy (RWMS)

Sustainable Flood Retention Basins (SFRB)

Sustainable Urban Drainage Systems (SUDS)

Synthetic Aperture Radar (SAR)

Top Water Levels (TWL)

Two-Dimensional (2D)

Urban Stormwater Management System (USMS)

Urban Stormwater Management Systems (USWMSS)

Urban Water Management Plan (UWMP)

Water Sensitive Urban Design (WSUD)

Western Australia Planning Council (WAPC)

XP Stormwater and Wastewater Management Model (XPSWMM)

CHAPTER 1

1. INTRODUCTION

1.1. Background of the study

Management of stormwater brings considerable socioeconomic and environmental benefits to the community in various ways all over the world. In countries such as Australia, this includes addressing the water scarcity problems and water reuse as well. Local governments and environmental authorities are mainly responsible for the management of urban stormwater. They can play an important role in maintaining and improving stormwater systems and related resources through their collective and respective actions. Modern hydrologists, environmental engineers and town planners rely on water sensitive urban designs (WSUD) to prevent stormwater management issues and to safeguard urban lives and the urban environment. With the rise of modern human nations, cities are becoming more complicated in their designs and urban lands, buildings and other infrastructure values are increasing rapidly.

Urbanization usually comes with land development and land use changes that directly impact to the urban catchment hydrology. The effect of urbanization on stormwater management plays an important role when designing, expanding and maintaining urban stormwater management systems (USWMSs). The urban land development process converts natural bare lands with pervious surfaces into impervious areas. These changes reduce the overall catchments' infiltration capacities. They also reduce the surface roughness of the land. Urban drainage consists of pits and a pipe network that bypasses natural flow paths and provides low roughness surfaces to the attenuated runoff. All these changes impact to the USWMSs by increasing peak flow rates and runoff quantities. Climatic changes influenced by anthropogenic carbon emission lead to a decrease in the recurrent intervals of storm events and increase their intensities. The land use change involving the removing of green spaces has reduced evapotranspiration. All these factors ultimately make the designing of USWMSs much more complicated and challenging.

Historically, conventional stormwater management has focused on peak flow rate control through the use of detention basins (Simpson 2012). This usually dealt with end-of-line flow limitations by having detention or retention storage only. Stormwater source control has appeared over the past few decades as an alternative solution to end-of-line flow limitations for managing stormwater in urban areas (Braune et al. 1999; Martin et al. 2007). Intensity-duration-frequency curves also have been traditionally used in the designing of urban runoff treatment and management systems, together with various types of storage systems such as retention and detention basins, bio-retentions, soakage wells and swales (Thorkild et al. 1988). Stormwater BMP guidelines developed to enhance the USWMSs basically dealt with these concepts.

Urban flooding as a result of increasing urbanization, land use change and climate change can harm human lives, the urban environment, habitat and properties. Rebuilding of flooded urban cities and restoring residents' lifestyle back to normal costs millions of dollars and takes time. The other impact that comes directly from land use change is water quality and flood inundation. Ill-treated USWMSs can mix with flooded sewer systems during storm events and flow along with surface runoff, which can create a great threat to health. Excess urban runoff during storms flows into water storage areas, bypassing the water treatment systems and can stagnate within them. Therefore mitigation of urban flooding and excess runoff, providing proper USWMSs inclusive of treatment facilities, is important. Sustainable USWMSs, achieved as a result of a combination of BMPs designed by using WSUD concepts, is the key to liveable urban neighbourhoods. However, the safety of people and the protection of their valuables must be in balance with technical and socio-economic restrictions (Theo et al. 2004).

Keeping these facts in mind, this study aims to assess several existing and proposed urban stormwater drainage systems and find out what and where could be improved to establish a proper and efficient USWMS. The main consideration has focused on stormwater quantity as well as possible modifications/improvements to the drainage system to address the BMPs and achieve sustainability through WSUD. It also addresses the implementation of stormwater BMP guidelines upon new land developments and their effectiveness. Hydrological modelling has been selected as the best solution to analyse rapidly developing urban catchments' hydrology.

Numerical models play an important role in assessment of urban stormwater and urban flooding. There are number of numerical models related to runoff quality and quantity which are cited in the literature (i.e. MUSIC, PURRS, XPSWMM, PCSWMM, MIKE, SWMM etc.) (Elliott & Trowsdale 2007; Vijay & David 2002). The range of these models varies from very simple conceptual models to complex hydrodynamic models (Christopher 2001). Traditional catchment runoff routing models are not capable of representing complex urban catchments which are comprised of underground drainage, road networks, buildings, treatment BMPs such as bio-retention swales and multiple user corridors (MUCs) and linked storage systems. Some models have been developed during last decade as urban stormwater management models (i.e. XPSWMM). Application of one-dimensional (1D) and two-dimensional (2D) modelling components to model complex overland urban runoff flow is an improvement to these models. The capability to model urban surface runoff flow with 2D components whilst representing open canal, river and drainage networks with 1D components and coupling both components during a model run gives a complete urban stormwater management model (Syme et al. 2004). Some models have additional features such as coupling of groundwater mounding with urban stormwater drainage (i.e. XPSWMM, HBV and MIKE21). XP Stormwater and Wastewater Management Model (XPSWMM) is one of the leading numerical models in urban stormwater assessments (XPSoftware 2009). It was selected as the modelling tool in this study because of its popularity within Australia and some other parts of the world. XPSWMM is a comprehensive modelling system encompassing a graphical user interface (GUI) and an analytical engine (XPSoftware 2009). Its simultaneous hydrology and hydraulics analysis capabilities are considered when selecting it as a suitable model to analyse urban catchments' hydrology dealing with urbanization and land use changes.

Three case studies have been carried out to assess the impact of urban land development and land use changes as the main anthropogenic stress on USWMSs. A major case study of Canning Vale Central catchment drainage assessment was carried out to assess the impact of projected land use changes on the existing urban drainage system by the process of subdivision of the land and new developments. It was identified that the shallow groundwater table in the catchment is playing a major role in the lack of capacity of the existing drainage system, over and above the effect of increased runoff quantity due to land use changes. The increased runoff quantity due to land development and land use changes, the effect of a shallow groundwater table on the stormwater drainage in groundwater lodged catchment, groundwater base flow through drainage, on-line and off-line basins and swales, other flow controller such as weirs and siphons have been considered during the modelling process. The urban catchment was modelled by using both 1D and 2D elements. However, the suitability of the two approaches of hydrological layer used surface runoff routing and surface runoff routing by using 2D hydraulic layer have been analysed. The sensitivity of the urban catchment characteristics towards the peak flow rate has been analysed. Recommendations to improve stormwater management drainage to face predicted storm events were given. Localised flooding was assessed by generating flood vulnerability maps. The results of this case study will be used to improve the BMPs and guidelines to the Central catchment by the local authorities.

In the second case study of Victoria Park storage sump assessment has been carried out to determine the adequacy of the existing storage basins within the Town of Victoria Park to face the predicted storm events. The urban town of Victoria Park has been developed rapidly during past decades. As a result of urbanization and land use changes creating more impervious surfaces over the years, most of the sumps lack the required capacity. Inadequacy of sump capacities against recent storm events has been identified by the initial drainage implementation. This study analysed the required sump capacities and current flood vulnerability of the area against several average recurrent intervals (ARI) events. 2D hydraulic surface runoff routing was used to assess the ungauged urban catchment. Infiltration tests were carried out to find out the rate of infiltration within basins to predict the dry time periods. Flood vulnerable maps and water elevations under several storm events were produced as a result.

The third case study was carried out to assess the use of BMPs in urban land development and subdivision works to mitigate the effects of land use change to USWMSs. Wellard Residential Development site was used as the urban catchment.

1D modelling component of the XPSWMM was used to model several BMPs such as the implementation of rainwater gardens, roadside bio-pockets, lot-wise storage tanks and rainwater tanks, retention and detention basins. The model was developed to find out peak volumes of the end-of-line retention basins and required capacities of runoff treatment BMPs. Several WSUD techniques, BMPs and guidelines by local authorities have been reviewed to decide the suitable sustainable stormwater management approach to the development site. Urban stormwater management guidelines based on WSUD concepts have been achieved by controlled flood storage volumes and by using the infiltration modelling methods. To achieve these current stormwater management guidelines and higher standards, a host of structural and non-structural urban stormwater BMPs have been recommended to support an urban water management plan (UWMP) for the development site.

1.2. Aims and objectives

- To assess several urban stormwater drainage systems to determine the effects and sensitivity of catchment characteristics on runoff behaviour of the catchment.
- To analyse the effects of urban land development and land use changes on urban stormwater management systems.
- To couple the 2D surface water modelling techniques together with traditional 1D modelling to represent the urban catchment.
- To assess the effect of a shallow groundwater table on the urban drainage in groundwater lodge urban catchment.
- To assess the modelling of an ungauged catchment by using 2D hydraulic surface runoff routing.
- To develop potential flood distribution and flood vulnerability maps for different urban land development and different rainfall scenarios.
- To analyse the implementation and effectiveness of stormwater management guidelines based on best management practices used in modern urban stormwater management systems.

- To recommend necessary improvements and best management practices for urban stormwater management systems considering water sensitive urban design practices.

1.3. Significance of the study

Urban cities are facing the problem of flood inundation with more intense storm events and insufficient or outdated USWMSs. This can be a threat to human lives and properties, the functions of daily life and the quality of the urban environment. The unexpected occurrences of severe storms in the recent past, which are influenced by climate change effects, make it a vital and more critical issue to the modern world (IPCC, 2007). Whilst the clear fact of urbanization as a main anthropogenic stress to urban flooding exists, urbanization cannot be stopped or limited with a growing population and their demands. An understanding of the effects of land use changes to USWMS provides the starting point of achieving sustainable landscaping of urban developments, which can reduce the anthropogenic stress upon urban hydrology. The development of numerical modelling techniques to model urban catchments and USWMSs provides an accurate tool to assess the impact of urbanization on urban hydrology and to assess the BMPs that can mitigate disasters such as urban flooding. Identification of the sensitivities of urban catchment characteristics towards the modelling results will guide future urban modelling processes by helping to select and prioritise the modelling parameters. The study suggests an alternative method to model urban groundwater together with surface runoff and 1D drainage flows. It also provides an example of 2D surface routing method to analyse an ungauged catchment with lesser available data.

Some of the urban and sub-urban cities or parts of cities in Western Australia were being encountered with localised flooding during the recent storm events. Two of such flood prone areas, where local voicing was raised against the localised urban flooding during minor rainfall events has selected as case study #1 and #2 to analyse the USWMS by using numerical modelling. The results of Canning Vale drainage assessment provide suggestions to further improvements to the existing drainage system. It also provides flood vulnerability and inundation maps identifying flood prone areas for several different scenarios based on land use changes and rainfall events. These results can be used when deciding guidelines for the future developments. The results of the second case study provides flood inundation maps showing flood storage basins those may not be sufficient to bare the capacities of future rainfall events. It also provides the water level contours of flood prone areas and basins against several storm events to predict the risk to adjacent properties.

The results of the third case study suggest improvements to current USWMS and new possible methodologies to manage stormwater in urban areas in terms of achieving sustainability by using BMPs. It also provides modelling techniques to represent the BMPs within urban numerical modelling. Finally the results of the Wellard case study support the UWMP's 'Stormwater Management Strategy' section, which is mandatory to carry on the development of the site.

1.4. Limitation of the study

The effect of land use changes to the urban water quality has not been analysed in this study. It only assesses the land use change effected urban stormwater quantity. The groundwater interaction in to the urban stormwater management systems has been reviewed and discussed during the literature. It was applied for the catchments with shallow groundwater conditions are existed. However the method provided within the software to model groundwater mounding could not be used with its full equation due to the lack of data about soil properties. The linear reservoir method with an equation consists two changeable parameters was used as the groundwater modelling method. It was sufficient for analyse the major rainfall effects since the sensitivity of the groundwater to peak runoff rate is lesser. However the study had the limitation of using broader groundwater modelling techniques for the minor rainfall events. The land use scenarios have been assumed according the aerial photo graphs, landscape architectural designs and field data. For model the hydraulic structures, as-construction drawings have been used. In the case study #3 described within the thesis, the proposed structural drawings and landscape architectural drawings have been used. Literature related to impact of climate change to urban hydrology has been discussed in brief under Chapter 2, but the climate change effect has been neglected during the study to limit the scope of work.

1.5. Overview of the thesis

The overall thesis is consists of eight chapters. Chapter one provides the introduction to the overall research, hence the thesis. It consists of background of the study, aims and objectives, significance of the study and overview of the thesis. The background provides the brief discussion about the base of study. Its aims, objectives and significance have been given in this chapter while describing the scope of work.

Chapter two provides the review of literature prior to and during the research. Literature review is basically about urban hydrology and land use changes, climate change effects to urban hydrology, urban stormwater drainage, numerical modelling and review of the models and BMPs and WSUDs associated with the modern USWMSs. Effects of land use change and climate change have been discussed as main anthropogenic stresses to the urban hydrology. The stormwater management drainage and effect of groundwater to USWMSs has been discussed together with urban catchment characteristics. The importance of studying and analysing the urban hydrology and urban stormwater management has been emphasised by considering the possible disasters such as urban flooding. The stormwater management guide lines and WSUD associated with BMPs have been reviewed under this chapter. Finally the hydrological modelling and current modelling applications of analysing the urban hydrology have been reviewed. The XPSWMM model's capabilities and its applications have been discussed as the modelling tool used in the study.

Chapter three has been dedicated to research methodology including data collection procedures. Overall methodology of the study and modelling process has been introduced. Three case studies conducted during the study have been out lined briefly together with the methodology. Other than that, theories behind the modelling tool and the modelling techniques have been discussed in this chapter. Routing methods, groundwater mounding, 1D 2D coupling have been discussed by providing the based equations which are used by the modelling tool. The infiltration rate and ARI rainfall calculations are given in the chapter as basic theories.

Chapter four represents the sensitivity analysis, calibration and verification of the urban catchment models used in the study. The sensitivity analysis was used to find the best modelling approaches and the most sensitive catchment parameters during the modelling. Calibration process was discussed followed by the verification of the model. The values for land use change catchment characteristics were defined during this process. The calibration was based on observation data and the verification was done by using independent data set. Overall model validation and parameter fixing is described with theories in the chapter.

Major case study of Canning Vale Central catchment drainage assessment has been described with results in the Chapter five. The total catchment comprises of about 320ha has been modelled and the modelling methodology is been given. The results of outflows from each sub-catchment and comparison of results with previous studies and observation data is given. The flood inundation maps are introduced to assess the flood vulnerability of the catchment. Groundwater has been treated as a major parameter during this case study and it attempts to find the impact of land use changes and shallow groundwater on urban stormwater management and urban drainage.

The Chapter six discusses the case study of Victoria Park stormwater sump assessment. Use of 2D surface modelling to analyse the un-gauged urban catchment has been described in the chapter. The maximum top water levels and measured infiltration rates were used to analyse the sump capacities under standard ARI events. Flood inundation maps are given to this catchment to identify the localised flooding. Chapter seven represent the case study of effect of water sensitive urban designs. The case study tries to analyse the effect/ efficiency of best management practices used under the stormwater management guide lines. The modelling process represent the urban catchment hydrology consists with proposed drainage and BMPs. Results show the using of hydrological modelling to achieve the stormwater management criteria ordered by the local authorities and support the water sensitive urban design approach.

Finally Chapter eight summarises the results of the thesis study. Recommendations from the results and recommendations to future studies are given under the final chapter. Appendix A gives the flood inundation maps as part of results of the case study of Canning Vale Central catchment drainage assessment.

1.6. Publications associated with the study

Following publications assists the integrity of the literature review and their results have been used throughout the thesis.

- **Basnayaka A. P.**, R. Sarukkalige (2011). Comparing Hydrology and Hydraulics Surface Routing Approaches in Modelling an Urban Catchment. 2nd International Conference on Environmental Engineering and Applications, (ICEEA 2011), Shanghai, China.

- **Basnayaka A. P.**, Sarukkalige R., Werellagama I. (2011). Numerical Modelling of Flood Vulnerability in Urban Catchments for Flood Forecasting. International Journal of Science and Development, 2(5).

- **Basnayaka A. P.**, Sarukkalige R., Werellagama I. (2011). Impact of Urbanization on Flood Vulnerability in Shallow Groundwater Catchment. Proceedings to 5th International Conference on Flood Management (ICFM5), 27-29 September 2011, Tokyo-Japan", "Floods: From Risk to Opportunity", IAHS Red Book Series. (in the press)

- **Basnayaka A. P.**, Sarukkalige R. (2012). Sensitivity Analysis of Catchment Characteristics in Urban Stormwater Management Modelling. Proceeding to Research Development and Practice in Structural Engineering and Construction, (ASEA-SEC-1), Perth, 28 November 28 – 2 December, 2012.

- **Basnayaka A. P.**, Sarukkalige R., Kannangara D. (2012). Effectiveness of stormwater Best Management Practices in Urban Land Developments. Conference on Water, Climate & Environment, BALWOIS 2012, Ohrid, Macedonia, 28 May – 2 June, 2012.

CHAPTER 2

2. LITERATURE REVIEW

2.1. Introduction

Urban stormwater management and planning is being challenged by continuous and rapid development of cities around the globe outdating their existing stormwater management systems and by anthropogenic climate changing precipitation patterns (Willems et al. 2012). Urban land use changes have caused to increase the runoff hydrographs' peaks and the runoff volumes at the downstream of catchments by having higher impervious area with low surface roughness coefficients compare to the natural pervious land use. The urban drainage system and the road ways by pass the natural water ways and again cause to decrease the time of concentration. Urban catchments situated with shallow groundwater table tend has faced the situation where their underground drainage is submerged. Effect of groundwater can cause to inundate the urban areas by reducing infiltration and seeping in to the stormwater drainage occupying the stormwater drainage.

There are numerous issues on managing urban stormwater with the effects of combined anthropogenic stresses upon the urban hydrology. Urbanization and land use change affected surface runoff, variation of weather patterns, intensified storm events and increased demand have proved the necessity of implementing sustainable stormwater management strategies through best management practices (BMPs) based on water sensitive urban designs (WSUD) in urban cities. The BMPs such as bio retention basins, soakage wells and treatment swales have been introduced to the urban stormwater management systems (USWMSs) to sustain the urban water quality. The concepts such as source control rather than traditional end-of-line retention and detention basins have been improvised. As a part of the WSUD concept, BMPs such as rainwater tanks and submerged storage areas have been introduced to USWMSs to control runoff generation as close as to the source. These concepts are also capable of addressing the water scarcity of the countries like Australia by helping to water re-use. Australia, as one of the pioneers in water sensitive urban designs, has developed its own methodology and catchment guidelines in USWMS designs. A number of hydrological assessments and stormwater management strategies, prior to and during the new land development processes, have been established as a result.

When there are no adequate stormwater management systems or when the existing systems are out dated with the increased demand, issues related to the urban water quality and quantity can be arisen. Un-treated urban runoff carries various types of pollutants has caused to water quality issues. Moreover, increased frequencies and intensities of major storm events affected by the climatic conditions and insufficient USWMSs have caused more often urban floods. Therefore study of urban hydrology and USWMSs are important aspects to protect the liveable urban environments.

To mitigate the urban stormwater management issues, analysis of combined (or even isolated) effects of all above mentioned phenomena towards the urban stormwater management is a quite complex process. There should be a proper method or tools to analyse the urban hydrological scenarios. Urban stormwater management models and statistical analyses can be identified as such tools which are used numerously all over the world. Various types of urban stormwater management models concentrated on one or few phenomena have been created in the recent history. Traditional runoff routing models have been modified in to the urban watershed models those can represent the characteristics of urban catchment more effectively. Recent models are supported with the modern techniques such as light detection and ranging (LiDAR) data which has broader applications in the fields such as geography, geology, remote sensing and contour mapping. The features of modern stormwater management models such as capability of identifying input data of drainage networks, soil properties, topography, land use changes and other spatial data through geographic information system (GIS) have enhanced the user-friendliness of them and accuracy of model results.

2.2. Urban hydrology and land use changes

There is a strong trend of urbanization throughout the world, which leads to the land use changes in large scale. The urbanization process changing the land use is inevitable with the increasing population and the resource scarcity. The modern crisis of uncontrollable dense populations attracted to the resources in urban cities has affected to the urban environment, and one such a major vulnerable element of the environment is urban hydrology. Impact of urbanization, mainly by land use changes on urban stormwater management has been discussed immensely in the recent literature (i.e. Goonetilleke et al. (2005); Carlson and Arthur (2000) and Pauleit et al. (2005)). Recent studies by Suarez et al. (2005); Semadeni-Davies et al. (2008a) and Semadeni-Davies et al. (2008b) have discussed the effect on urban stormwater management by both urbanization and climate change. Land use change can be characterized by the complex interaction of behavioural and structural factors associated with demand, technological capacity, and social relations, which affect both demand and environmental capacity (Lin et al. 2007). Lin et al (2007) also mentioned that the land use changes in a watershed can impact water supply by altering hydrological processes such as infiltration, groundwater recharge, base flow and runoff. Urban development accompanied by increasing in impervious surfaces such as roofs, roads and paving, construction of manmade drainage systems, compaction of soil and modifications to vegetation directly affects its natural stormwater paths and existing stormwater network (Elliott and Trowsdale 2007). Also the drainage systems inclusive of drains, manmade channels, manholes and gutters increase the rate of runoff through the drainage (Selvalingam et al. 1987). Land use changes associated with urbanization increasing more impervious surfaces characterized by low infiltration and accelerated runoff (Jian et al., 2009). It has resulted to the changes in characteristics of surface runoff hydrographs by increasing stormwater runoff volumes and peak flows (Goonetilleke et al. 2005 and Barbosa 2012). This causes to exceed the capacity of the urban multiple user corridors (MUCs) and stormwater drainages which will ultimately cause to flood the cities. Again with the removal of vegetation the evapotranspiration is reduced, and leads to stormwater to be retained in the surface for more time.

Considering the land use change by de-forestation of large scale catchments which are eventually transferred either to be agricultural lands or residential developments, the impact to the catchment hydrology can be both by climate change effects and increased runoff generation. The increment of peak runoff from such large catchments by this eventual process of natural catchment land use is been transferred to the agricultural land use and to the urban land use is reviewed by many studies in the literature, notably (Andre´assian 2004; Best et al. 2003 and Zhang 1999). Other than the total natural catchment is been converted to the paved infrastructure, there can be conversion of forest to pasture or the afforestation of grassed catchments through the recreational landscaping of public open spaces (POS), play grounds, multiple user corridors etc. (Siriwardena et al. 2006). Siriwardena et al. (2006) cited that the runoff increment of such large catchment which was subjected to just deforestation as estimated as 40% from the natural catchment runoff. The possible urban development top of deforestation in such a catchment will increase this value to much higher figure. Recent studies conducted by Lin et al. (2007); Agarwal et al. (2002); Parker et al. (2002); Luijten (2003); Rounsevell et al. (2003); Stewart et al. (2004) and Manson (2005) cited the land use changes and their effects to urban environment, especially to the urban runoff by using different analytical modelling solutions.

For landscape and environmental planning, the ‘sprawl’ of low-density settlements and urban development along transport corridors is causing particular concern in highly-industrialized countries (Pauleit et al. 2005). Cities experiencing population growth have a choice to either increase density in their core through infill and vertical development or to incorporate rural and less developed land along the peri-urban fringe, a process known as sprawl (Lily et al. 2011). Infrastructure related to transport such as roads, round-a-bouts, pedestrian foot paths, tunnels and bridges together with buildings is the major land use change in an urban catchment compare to its pre-development natural land use. The natural flow paths existed in the pre-development catchment can be obstructed, re-directed or accelerated by these features. The pervious land use in the natural catchment is changed to more impervious surfaces and the surface roughness is changed from course to smooth by these features. Bitumen and concrete used to develop the road networks and foot paths and concrete, corrugated materials and glass etc. used to cover the buildings and their associated paved areas are usually having zero (or nearly zero) infiltration and surface roughness value of about 0.014~0.015 (Chow 1959). This helps all the rainfall landed within these surfaces to become as immediate runoff from the catchments. In natural catchment, portion of the rainfall would be infiltrated by the un-covered soil surfaces.

To reduce the increased impervious land use percentage in urban cities, modern landscape architectures and town planners include green spaces by many other methods to their urban plans. These are mainly associated with public open spaces (POS) and some of the stormwater management features such as bio retention basins, flood storage areas, swales and Flood corridors or multiple user corridors (MUC) (SoSJ 2003). They enhance the landscaping features and bio diversity by increasing green spaces and having variety of plants. However, the urban land use change even including those items is still covers more impervious area percentage compare to the pre-development natural land use. However Pauleit et al. (2005) has cited that there is a lack of information on the environmental effects by urban land use change and the dynamics of green-space.

2.3. Impact of groundwater on urban stormwater drainage

Urban drainage systems inclusive of underground drains, manmade channels, manholes and gutters increase the rate of runoff through the drainage (Selvalingam et al. 1987) and decrease the time of concentration. This has created the intense peaks in the runoff hydrographs which the stormwater management designers and engineers trying to reduce by having controlling devises attached to the drainage systems such as weirs, treatment spots like rainwater gardens and bio pockets, lot wise storage areas such as rain water tanks, infiltration storage areas like soakage pits, water retaining and detaining structures, subsurface storage areas and slotted pipe systems like French drains etc. Fewtrell et al. (2011) has discussed the recent studies on the effects of urbanization to the urban drainage and urban flooding in terms of drainage network structure drainage network efficiency, drainage pathway distribution and model resolution.

Urban catchments situated within the river estuaries, nearby coastal areas and low elevation urban catchments with shallow groundwater table can have the effect from rising groundwater level especially in winter and rainy seasons. Elevated shallow groundwater table causes to submerged underground stormwater and sewer drainage. Localised urban flooding can be the result of inadequacy of capacities of underground drainage systems due to their submerged conditions. Road drainage network is draining the groundwater when the groundwater level reaches to the level of the drainage. Soil layer near to the surface which usually absorbs the initial rainfall can be saturated by the shallow groundwater effect and can cause to increase the urban runoff. The groundwater can be leaked to the drainage from the defected water tightened joints and the unsealed bottoms of the manholes (Berthier et al. 2004). Wise versa stormwater infiltration through un-sealed manholes and drainage joints can cause rising groundwater tables in urban areas (Göbel et al. 2004). The groundwater table is always dynamic and characteristics with the seasonal variations and quick response to the heavy rains. Groundwater table rises to the natural surface level stopping infiltration and occupying stormwater drainage network leads most of the rainfall to flow as surface runoff. In other hand groundwater recharge in urban areas is dramatically reduced by the accelerated base flow through the stormwater drainage and less infiltration and accelerated surface runoff by the urban impervious surfaces (Wheater and Evans 2009). Also (Pitt, Clark, and Field 1999) cited the potential groundwater contamination problems associated with stormwater infiltration. Urban stormwater runoff flows along the urban surfaces absorbing more pollutants can infiltrate in to the groundwater through a thin soil layer when proper treatment measures are absent and can directly get in to the groundwater table through submerged stormwater drainage affecting the groundwater quality. Therefore groundwater should be treated as one major parameter during such urban catchments.

2.4. Climate change effect and urban stormwater management

Climate change influenced by the greenhouse gas emissions too has adverse effects on urban hydrology. Shorten recurrent intervals of storm events is one of the adverse effect heavily affected to the urban stormwater management. Elliott and Trowsdale (2007) cited that new urban water management approaches have been developed to deliver improved environmental, economic, social and cultural outcomes in last two decades. Developing the evidence base for mainstreaming adaptation of stormwater systems to climate change by Gersonius et al. (2012) gives an evidence of recent such approaches. Pyke et al. (2011) cited that stormwater management systems, may need to meet performance expectations under future climate change scenarios. Studies such as that by Semadeni-Davies et al. (2008a) and Semadeni-Davies et al.

(2008b) proved the possibility of analyse the climate change effect and urbanization effect to the urban stormwater management within a same model structure. There for study the climate change effect to the urban stormwater management is important. Semadeni-Davies et al. (2008a) cited that even there is a lack of both tools and guidelines for climate change impact assessment in hydrology, the assessment of the potential impact of climate change on water systems has been an essential part of hydrological research over the last couple of decades. However The recent studies by Willems et al. (2012); Gersonius et al. (2012) and Pyke et al. (2011) have discussed the influence of climate change on urban stormwater management quality and quantity.

Climate change is expected to include increases in the frequency and severity of storms (Suarez et al. 2005). Banaszuk and Kamocki (2008) cited as it has been concluded that the most vulnerable areas are those where precipitation currently occurs mainly in the form of winter snowfall and stream flow is largely generated by spring and summer snowmelt, based on the global model of climate change. In such areas temperature increase may lead to an increased winter runoff and a reduced spring flood pulse (Bergkamp and Orlando 1999). Semadeni-Davies et al. (2008a) cited that the assessment of the potential impact of climate change on water systems has been an essential part of hydrological research over the last couple of decades. However these studies have suggested that there can be changes to the other urban stormwater management systems influenced by the climate change.

To investigate climate change effects analysing trends in long-term historical records of rainfall is needed. The projected changes in rainfall statistics based on future scenarios in greenhouse gas emissions simulated in climate models (atmosphere–ocean circulation models: General Circulation Models (GCMs) and Regional Climate Models (RCMs)) or statistical extrapolation based on historical observations need to be transferred to changes in the urban drainage model inputs (Willems et al. 2012). The question is how to assess the urban stormwater management systems considering the climate change effects which have been modelled under regional scale. Downscaling of results from global circulation models or regional climate models to urban catchment scales are needed because high resolution of temporal and spatial data is required to analyse the urban catchments (Willems et al. 2012 and Schilling 1991).

2.5. Sustainable stormwater management

Water sensitive urban design (WSUD) is a relatively new urban development philosophy for sustainable urban water cycle management (Morison and Brown 2011). Coombes et al. (2000) quoted that the developments which are 'water-sensitive' involve water conservation and stormwater retention strategies employed at the urban allotment or 'cluster' level to reduce infrastructure costs. However this strategy has been implemented in Australia over the past decade (Kazemi et al. 2009a). Low Impact Development (LID) is a similar concept developed and used in USA and PGDER (1999) cited that LID implements engineered small-scale hydrologic controls to replicate the pre-development hydrologic regime of watersheds through infiltrating, filtering, storing, evaporating, and detaining runoff close to its source. There are many strategies based on to achieve sustainable urban stormwater management. Butler and Parkinson (1997) proposed three strategies to achieve sustainable urban drainage based on reduce potable water "use", reduce and then eliminate the mixing of industrial wastewater with domestic waste, and reduce and then eliminate the mixing of stormwater and domestic wastewater.

Stormwater management strategies and guidelines should be based on sustainability. Rijsberman and van de Ven (2000) cited two definitions for sustainable development; "a development that fulfils the needs of the present generation, without compromising the ability of the future generations to fulfil their needs" and "maintenance of the natural resource base of future generations". Both sound the utilization of resources considering the future generation's needs, but the second definition says about the natural resources. Designing of stormwater management systems considering water as a reusable source, maintaining the level and quality of groundwater and maintaining the quality of surface water bodies etc. addresses the sustainability based on future generations. Sustainable urban drainage systems (SUSD) is the integrated term for such measures used by many authorities and cite by (Augusto Pompêo 1999; Benzerra et al. 2012; Butler and Parkinson 1997; Ellis 1995; Mitchell 2005; Rijsberman and van de Ven 2000). Sustainable urban drainage systems (SUDS), known as Best Management Practices (BMPs) in North America, USA, Australia and some other countries (Jones and Macdonald 2007; Kaplowitz and Lupi 2012). Rather than getting rid of water accumulated in urbanised areas ended up just by transferring the problem either towards other areas or to the future, planning and management of urban stormwater drainage in terms of sustainability has arisen (Augusto Pompêo 1999). BMPs which reduce the volume of runoff discharged to receiving streams, such as minimizing directly connected impervious surfaces, providing on-site storage and infiltration and implementing stream buffers and restoring riparian cover along urban streams can help to prevent further degradation and even result in improvements of streams which receive stormwater discharges (EPA 1999).

Sustainable urban drainage must maintain a good public health barrier, avoid local or distant pollution of the environment, minimise the utilisation of natural resources (e.g. water, energy and materials), and be operable in the long term and adaptable to future requirements (Butler and Parkinson 1997). Common SUDS are swales (grassed-lined ditches), porous pavements, filters and sediment traps, green roofs and roof-top gardens, infiltration surfaces and rain gardens/bio-retention, and the ubiquitous detention pond (Semadeni-Davies et al. 2008a).

Best Management Practices should be seen as an opportunity for development and improvement of social, educational and environmental conditions in urbanized and surrounding areas (Barbosa 2012). BMPs for stormwater can be structural or non-structural management practices. Non-structural BMPs attempt to improve aspects of water quality through efforts such as ordinances and education to change landowner and others' behaviour. Structural BMPs are physical undertakings and construction projects such as dry basins, wetlands, and filter strips aimed at reducing the impact of stormwater runoff (Kaplowitz and Lupi 2012). A range of types of BMP are under study, including both source control and end of pipe systems (Jefferies et al. 1999).

2.5.1. **Source controlling**

Source control, introduced during the 1980s, is a technique aimed at temporary storage in urban lots for flow reduction and when reduction of volumes is required (Augusto Pompêo 1999). Temporary storage based on infiltration soak wells and storage that directly attenuates water from roofs and paved surfaces, such as rainwater tanks, is one popular source control option for modern urban drainage planners (Coombes et al. 2000). To use the soak wells efficiently, the soil should be high permeable and the groundwater level should not be encountered within the depth of these soak wells. The mulches and filling soil on urban lots provide a good infiltration media and extra height to the surface above the groundwater table. Sometimes the infiltration of filled soil is better than the pre-development soil infiltration, especially in areas where more clay mixed soils can be found. However this source control technique should be addressed at an individual lot scale and individual attitudes will highly influence the use and maintenance of the system (Augusto Pompêo 1999).

Another source controlling method is attenuating and treating the road runoff at the high stage of the catchment near to the source. This runoff includes runoff from driveways (in lots), excess runoff from lots, recreational areas and any other source in the catchment for major rainfall events. There are several techniques can be used to attain the storage and treatment of road runoff. Subsurface storage associated with residential lots—and used especially in commercial and business complexes to attenuate water—is another method of source controlling. Collected water is stored for fire fighting emergencies, industrial usage, washing and cleaning purposes, gardening and sometimes even as a drinking water source, after purification. Industrial parks and business/commercial complexes can have more than 80 per cent impervious surfaces and letting all these run off into the downstream flow causes a huge issue in treatment and attenuating them downstream, considering the extremes of the peak flow rate. Therefore sub-surface storage methods are sustainable in drainage design. They can be referred to as underground rainwater tanks in some cases, which are overflowing to the sub-surface (Coombes et al. 2000). Some systems can be used as flow controllers at the source, by using control weirs. Also the usage of slotted pipes as sub-storage units, which attenuate and infiltrate the surface runoff to the sub–surface, is another version of combined storage and infiltration systems.

Bio-retention basins, which are a type of vegetated WSUD system, can be used to promote biodiversity by designing and managing them with different plant varieties. This practice is another runoff treatment and attenuating method (Kazemi et al.

2009b). A bio-retention basin, also called a rain garden by landscape architects, naturalizes stormwater recharge and has other ecological attributes (Western Australian Planning Commission 2007). The WAPC (2007) again cited that the ecological attributes of the bio-retention basin as its ability to effect nutrient cycling, air and water pollutant abatement, carbon, habitat augmentation and connectivity, street-side beautification, reduction of building heating and cooling costs and urban heat island mitigation through direct shading and indirect evaporative cooling. Several recent studies on bio-retention basins in Australia have been carried out (Kazemi et al. 2009a, 2011). The importance of the bio-retention basins is that they can help remove pollutants from runoff and in the meantime support the concept of a liveable urban environment by contributing to sustainability in landscaping. Water within the bio-retention basin infiltrates through a layered organic–mineral soil (WAPC 2007). They can be used as on-line treatment units in urban stormwater management systems which provide extra volume capacity to deal with runoff and slow down the downstream runoff flow rate by providing high roughness values in the flow path. Bio-retention systems can be easily adapted to landscaping designs and usually can be placed alongside streets, car parks and traffic islands (Kazemi et al. 2009a). They are being used commonly in Australian urban areas as a WSUD system component to treat and attenuate1 year ARI event's runoff. Stormwater trenches and grassed swales are similar versions used commonly, but mainly convey the runoff while treating rather than attenuating.

The use of permeable pavement as a sustainable infrastructure material (Sansalone et al. 2011) to infiltrate the road runoff is another solution to source control. The permeability of soil is important to achieve efficiency in such an infiltration system. Also the type of source, which the type and amount of waste and pollutant load can vary according to, and factors such as gradient of the pavement and roads can be the key parameters in deciding the suitability of permeable pavements to a particular urban catchment because of the clogging factor, which reduces the efficiency of such a system. Again there can be adverse effects from the use of permeable membranes, as in the case of new BBC centre at Pacific Quay, Glasgow, associated with extensive use of porous paving. A permeable membrane, letting water pass through the porous media without natural infiltrating through the subsoil, allowed contaminants to leach into the River Clyde (Jones and Macdonald 2007).

However, stormwater infiltration facilities (strips, porous pavements, basins, etc.) that collect pollutants accumulated on carriageways, have to be integrated in risk assessments on water resources in urban zones and should be maintained to reduce the risk (Hill et al. 1998). Also there is the potential to raise the groundwater table by stormwater infiltration methods, especially large bio-retention basins, and adversely impact sub-surface infrastructure, undermining the benefits of naturalizing the urban water cycle (WAPC 2007).

2.5.2. **End-of-line control**

End-of-line flood controlling structures such as retention and detention basins are still popular in stormwater management. Scholz and Sadowski (2009) cited that aesthetically pleasing retention basins have been predominantly used for flood protection, adhering to sustainable drainage and best management practices. Stormwater control structures (sometimes called Best Management Practices or BMPs) like dry extended detention ponds or wet retention ponds have been installed, mostly in new developments, to intercept stormwater on its way to surface waters (U.S. Environemntal Protection Agency 2006).

A common practice of WSUD in Australia is to use the road network as the conveyor of excess runoff of ARI events greater than 10 year (major rainfall events). The excess runoff to the drainage system is controlled and/or attenuated fully within the catchment at the end of the catchment (or the end of the pipe system) according to the local authority’s guidelines (WAPC 2008). Usually the 100 year ARI critical duration event is used to design the retention or detention basins. Control peak flow measures can be varied, but common practise is to match the pre-development and post-development 100 year peak flows. Similar pre-development conditions to the post-development peak flow are achieved by using storage and controlling structures such as weirs. Providing treatment units to match major rainfall events is advised by local authorities (WAPC 2008). To match the pre and post development situations, flood retention basins are used commonly. They can store the excess volume of urban runoff generated due to the post-development land use change and limit the outflow from the catchment.

The designing of retention and detention basins should be incorporate with many aspects such as the infiltration capacity of design basins, possible clogging, outflow controls such as weirs and maximum water retention time with care for public health (i.e. mosquito breeding issues). The traditional practice of designing retention basins mostly accounted for the peak flow rates of the catchments being calculated by the rational method and then the infiltration rates being calculated by using equations such as a modified Darcy's Law. However, some studies in the recent past provide some modelling methods for designing infiltration basins by considering complex urban catchment characteristics, which may not be represented by the usual direct rational method calculations. As an example of such a practice, Scholz and Sadowski (2009) have recommended a rapid conceptual classification model for Sustainable Flood Retention Basins (SFRB) used to control runoff in a temperate climate.

However, there are some problems associated with end-of-line large-scale retention and detention basins. Artificial recharge of urban aquifers with stormwater has been used extensively in urban areas to dispose of stormwater and compensate for reduced groundwater recharge (SoSJ 2003; Hill et al. 1998). As a result, considerable amounts of stormwater sediment contaminated with heavy metals and organic compounds can accumulate over time in the upper layers of infiltration beds and can be a threat to surface and groundwater quality (Hill et al. 1998). Therefore, Lassabatere et al. (2010) underlined the need for efficient monitoring of infiltration basin sedimentation and its impact on water infiltration capacity.

2.6. Australian stormwater management guidelines

All over the world, principles and guidelines are outlined for the development and implementation of WSUD through BMPs to achieve sustainable levels of environmental enhancement in urban flood ways and corridors (Ellis 1995). However (Benzerra et al. 2012) cited that the task is difficult because of the multi-dimensional requirements of a sustainable development approach (economy, society and environment), as well as the lack of structured methodology and information at various levels of the hierarchy. Urban stormwater management planning should include the planning of the urban grid and its expansion, the zoning of activities, the road and transport network, landscape aspects and other issues. (Augusto Pompêo 1999). Also Lin et al. (2007) cited that the development of an integrated approach to assess land use changes, land use patterns and their effects on hydrological processes at the watershed level is crucial to land use and water resource planning and management.

Town-planners, architects and water managers are expected to evaluate the possibilities and the potentials of the water system and to integrate the urban water cycle into the town plan during the initial phase of town planning (Icke et al. 1999). Many of the Australian local governments and environmental protection agencies have now developed guidelines to manage urban stormwater in new land development sites and sub-division processes, urging plans and strategies for stormwater management in quality and quantity, based on best management practices (BMPs) prior to land development processes. The Western Australia Planning Council (WAPC) has developed guidelines intended to assist regional, district and local land use planning, as well as subdivision and development phases of the planning process to sustain BMPs in stormwater management (WAPC 2008). These guidelines are based on State Planning Policy 2.9 (WAPC 2006a), which is a requirement of the State Water Strategy for Western Australia (Government of Western Australia 2003).

Figure 1 shows the sequence of implementation of the planning policy during a development process. The preparation of a regional water management strategy (RWMS), which includes the broader catchment description, local water issues, a plan and methodology of regional scale water resource management, is done by the Department of Water, under the Government of Western Australia. During this stage they might consult many local authorities related to water resources and environmental planning; for example the Swan River Trust and the Department of Environment and Conservation. It is a general structure for the broader region. District water management strategy (DWMS) preparation is a process of scaling down the RWMS and including more strategic components that are unique to that district in detail. Again, the preparation of DWMS is a responsibility of the state government. Such strategies are already in place for most of the districts in Western Australia. They guide the developers preparing the LWMS and UWMP supporting to their development projects.

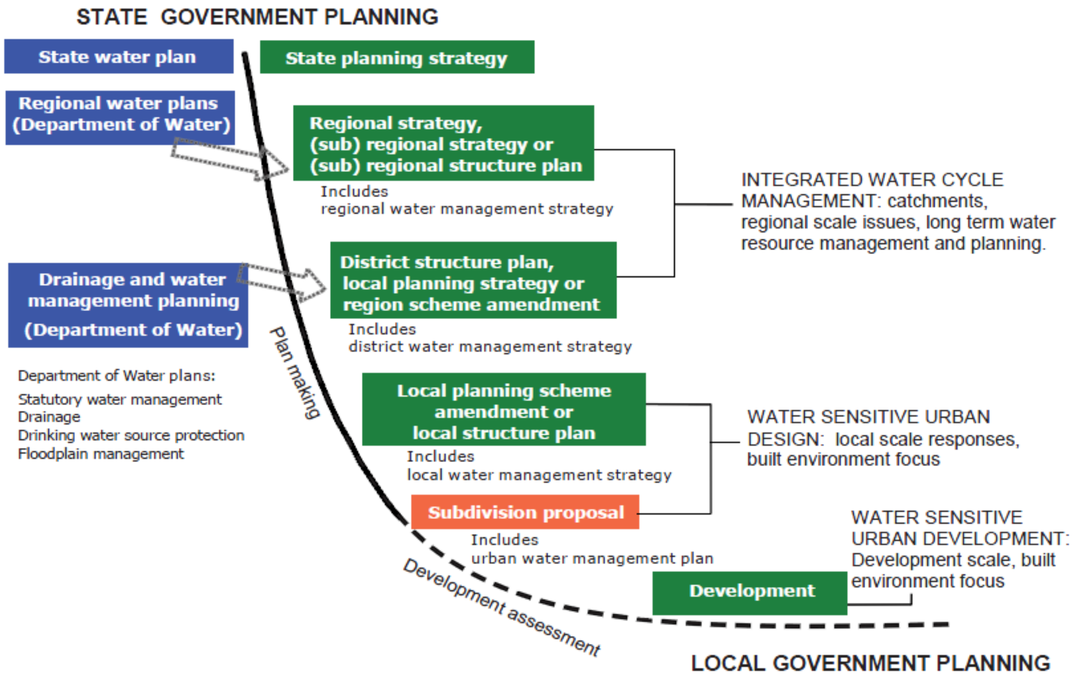


Figure 1. The sequence of planning processes related to stormwater management (DoW 2008b).

DoW (2008a) provides guidance on urban water management issues to be addressed at the stage of local planning the land development process, while supporting the rezoning of local schemes and/or local structure planning. The documents also guide the preparation of supporting documentation of LWMS to the approval of urban structure and landscape for new residential, rural-residential, commercial or industrial development (including redevelopment) areas. In the next stage, after the approval of LWMS supporting initial identification of the zoning (or re-zoning), preliminary structural plans and landscape architectural work, DoW (2008b) provides guidance on the urban water management issues that need to be addressed at the subdivision stage of a development and explains the integrity of a UWMP. It is expected that a proper structural plan and landscape architectural plans for the development site will be made at this stage.

Urban surface water modelling assessment by using approved software (e.g. XPSWMM) should be carried out prior to addressing the stormwater management section in a LWMS or UWMP. The following tasks should be addressed during the modelling process (WAPC 2008).

- Floodplain and wetland modelling to determine minimum building levels, catchment breaks for development and receiving water levels.

- Flow monitoring of existing surface water streams to establish current requirements.

- Identify how to manage post-development flows to meet catchment target flows.

- Drainage modelling to determine the detailed land requirements and flood ways needed to cope with major and minor storms (1 in 1 year, 1 in 5/10 year and 1 in 100 year), based on the receiving environment's requirements and/or design criteria provided in an endorsed water management strategy or plan.

- Establish acceptability of location of surface water flow paths (streams) and floodwater storage areas (floodplains) in consultation with the drainage service provider.

- Identify and address potential impacts on surface water-dependent ecosystems that are to be protected. Demonstrate that any potential impacts on flow will not have a significant environmental impact. Where any changes to the hydrological regime are proposed, this should be demonstrated to be consistent with the guidelines for ecological water requirements for urban developments currently being developed by Department of Water.

The document also highlights the modelling components to be carried out with the LWMS and UWMPs as:

- Demonstrating how post-development flows will meet catchment criteria.

- Modelling of up to 1 in 1 year ARI event to determine capability for retention/detention and water quality treatment, where/if required.

- Modelling of "minor" and "major" stormwater systems to identify and size flow paths (via pipes or overland flow) and required flood detention volumes.

- Refinement of 1 in 100 year floodway if required.

2.7. Urban flooding

Western Australia, where the study was based, is being subjected to intensive short duration rainfalls in recent history, which can cause localised urban flooding and even worse riverine flooding. Nevertheless, flooding is a serious issue for urban cities all around the world, which is gradually intensifying in frequency with urbanization and changing climatic conditions. Modern urban environments providing accelerated runoff mainly by land use change and manmade drainage has intensified the effect from such events in urban cities. The worst case is when no effective measures have been taken for flood mitigation with regards to structural and non-structural best management practices in the design stage and during the maintenance of urban stormwater management systems. However, there is a positive trend towards research on flooding, looking at the number of recent publications related to flood control, mitigation, flood-related urban design and especially flood modelling. Numerous publications in the literature on flood modelling, covering the whole range of riverine, estuarine and coastal flooding, including urban flooding, can be found. Research works on urban flood modelling include: Boyd et al. (1996); Mignot et al. (2006); Neal et al. (2009); Sanders (2008) and Yu (2010). Campana and Tucci (2001), and Fewtrell et al. (2011) cited works on the use of LiDAR data for urban flood modelling and flood mapping. A two-dimensional shallow water equation based on Manning's roughness has been used in many models to generate the spatial flood distribution (Fewtrell et al. 2011, Mignot et al. 2006 and XPSoftware 2009).

Increased runoff in the urban environment may cause urban flooding which affects day-to-day activities, properties and even human lives. The recent flooding in Australia caused the loss of properties worth billions of dollars. Again the inundation of urban areas after flooding may be extended to a few days by inadequate water management system, and can be caused to economic losses and temporarily interfering with the lifestyle of the people. Flood prevention by conducting careful analysis of the urban hydrological cycle and supporting adequate stormwater management drainage considering the results of this analysis is important to safeguard the quality of life within urban environments.

Continuous research into anthropogenic stress on urban hydrology and stormwater management has been carried out throughout recent years to find solutions to overcome disastrous situations such as urban flooding. Urban flooding is the worst and devastating result for urban environments caused by these factors. Moreover, the climatic effect intensifying the frequency of major rainfalls has caused more frequent urban floods in past recent decades. Built up pressure in sewer systems by unexpected stormwater runoff allowing them to be overfilled and mixing with surface runoff can create a worst case situation in terms of surface water quality. Treatment of water bodies affected by such hazards is an immense, costly and time consuming task. To mitigate urban flooding, engineered solutions have routinely been adopted to reduce flood peaks through the provision of storage area (Wheater and Evans 2009). To predict the effects of future urban development to flood regimes, the design hydrograph must be estimated and rainfall runoff modelling should be carried out (Campana and Tucci 2001). The design hydrograph for the study areas in Western Australia can be obtained from the web-based application promoted by the Bureau of Meteorology, which is designed by using historical rainfall data (BoM 2012). Otherwise the methods of producing hydrographs are cited in Pilgrim (1987).

2.8. Numerical modelling

One way of analysing catchment hydrological behaviour is by creating a numerical model, which represents the hydrological features and processes of the actual catchment numerically. Numerical modelling is essential to analyse the rainfall runoff process in a gauged or un-gauged catchment because of the limitations of measurement techniques and measured data and because of the requirement for predictions of future catchment hydrological behaviour patterns when it comes to decision making aspects of planning (Beven 2001). Numerous scientific research projects themed on finding an analytical solution to predict catchment hydrological behaviour are being carried out all over the world and there are numerous analytical models in different generic types. These differ in their assumptions and are based on catchment characteristics and also differ in the modelling techniques they use for the numeric analysis spatially and temporally that are being created as a result of those studies. Extremely complex hydrological phenomenon, which cannot be represented by a single or combination of mathematical solutions smoothly, has led to the still increasing number of rainfall runoff models.

Wagener et al. (2004) have described the rainfall runoff models in context as tools routinely used for hydrological investigations in engineering and environmental sciences, which can be applied to extend stream flow time series in space and time, to evaluate management strategies, catchment response to climate, land use variability, for the calculation of flood design, as load models linked to water quality investigations, for real-time flood forecasting and to provide boundary conditions for atmospheric circulation models.

The hydrological modelling process can be described as a sequence of processes starting from the perceptual modelling process where the exact hydrological processes are decided. Wagener et al. (2004) cited that the number of free parameters above a certain level does not increase the model performance significantly. Therefore carrying out a sensitivity analysis before and during the modelling process to identify the key hydrological processes—those have significant influence on accuracy of modelling results—is important. Deciding the perceptual model can be varied according to the experience of the scientists, the catchment's behaviour (e.g. whether the model is being built to analyse an urban catchment or river based watershed), available gauged data and the pattern of expected results. It is not necessary to fully express the perceptual model as a mathematical model which is used in the next stage of the modelling process of the conceptual model (Beven 2001). In fact a perceptual model inevitably cannot be represented by a mathematical theory with its complexity, by reason of having several processes affect each other's performances. The conceptual model is the stage for deciding the equations. Hypotheses and assumptions are being made during this stage to simplify the mathematical equations. The catchment of equations decided during the conceptual modelling stage maybe transformed directly to digital computer program code, which can be used in a computer and this stage will be procedural modelling. If the equations cannot simply be solved by an analytical solution then the boundary conditions for the real system can be given, which requires an additional stage of numerical analysis to define the procedural model (Beven 2001).

Catchment hydrology depends on catchment characteristics such as rainfall pattern, soil properties, flow paths, the watershed's width and gradient, land use, wind and humidity together with evapotranspiration etc. All or some of those catchment characteristics can be considered according to their sensitivity to the catchment hydrology when building a non-linear function in the hydrological system. Models based on such a complex non-linear function can be fed into a computer as a numerical program and can create a numerical computational model which can solve the complex algorithm. With the advancement of computer technology, numerical modelling has been improved significantly and the time it takes to run a model is reduced. Also it has given us the possibility of reducing the scale of spatial resolution and reducing the time step size for one iteration process, which ultimately gives more accurate results.

Before applying a model based on a catchment of equations that has been decided upon selected hydrological processes (considering their sensitivity) and transformed in to a digital code into practice, it is necessary to calibrate it by using gauged data to ensure the model represents the actual catchment hydrology within it (Beven 2001; Wagener et al. 2004). When deciding parameters to be entered into the conceptual model stage and in the calibration stage, the sensitivity of modelling processes based on catchment characteristics plays a major role. Therefore sensitivity analysis and estimation of predictive uncertainty have become central research topics in the hydrological modelling community (Abebe et al. 2010). Liu and Sun (2010) cited sensitivity analysis as the study of how the variation in the output of a numerical model can be apportioned, qualitatively or quantitatively, to different sources of variation in the input of a model. Also, new technologies such as remote sensing techniques can be used to calibrate the spatial data-based hydrological models. For example, DoW (2009) have cited the use of distributed remote sensing data for model calibration and evaluation. They have developed a method to calibrate flood inundation models by using satellite photos. Model verification is the next stage after the calibration process and basically it represent the process of testing the model in to data set, independent from the calibration data set, and commonly it can be a split-sampling test where the data set is divided in to two periods (Wagener et al. 2004). The model can be verified by random observation data which is not related to the calibration data in space and/or time. The calibrated and verified model can be used to analyse the hydrological behaviour of catchments, including urban hydrological catchments, depending on model capabilities.

There are a number of models that have been implemented historically for stormwater runoff quantity analysis. SWMM (Gironás et al. 2010); Mouse (DHI 1996); Hydroworks (Wallingford 1997); MIKE 21 (DHI 2007a); MIKE (DHI 2007b); HBV (Abebe et al. 2010); XPSWMM (XPSoftware 2009) are some of the examples of commonly used surface runoff models. Some surface runoff models basically developed to model flood plains and large scale rural catchments and have been further developed as urban stormwater models (e.g. XPSWMM, XPSoftware 2009). Urban catchment modelling is significantly more complex to analyse with numerical models than rural catchments, because the modelling demands the consideration of urban features like fences, highly varying land-use, buildings and other structures, narrow flow paths and underground stormwater drainage (Syme et al. 2004). With regards to this argument, Mignot et al. (2006) has cited surface flood modelling of the urban environment is a challenge because of the presence of a large number of obstacles of varying shapes and length scales, water storage in the buildings, the complex geometry of the city, etc. all have to be represented within the model. However, it has been cited by Zoppou (2001) and Nourani (2009) that the representation of urban hydrology within numerical hydrological models is done by many approaches throughout recent history. Some include the coupling of models (Abebe et al. 2010) to find the combined effect of a few or more catchment characteristics. With the inadequacy of common runoff catchment models and approaches to analyse the urban catchment with its complexities, a combination of 1D and 2D models, different methods of representation of urban concepts such as dual drainage systems (Smith 2006), GIS and raster based flood modelling approaches by using LIDAR data and aerial photography (Chen et al. 2009; Fewtrell et al. 2011; Smith et al. 2010) have been studied in recent history.

However Fewtrell et al. (2011) cited that it is difficult to ensure that each model interpreted the model inputs and boundary conditions in the same way. This is the best reason to select the most suitable model according to its capacities and catchment characteristics to a specific urban catchment before modelling (Fewtrell et al. 2011). A number of studies have assessed the importance of model resolution for simulating surface water propagation, while others have investigated the necessary process representation.

There are several hydrological runoff routing methods have been used by different software to generate the surface runoff hydrographs. Loss rate methods such as the rational method and storage routing model are still widely used for calculating rainfall excess and used by many models (Mouri et al. 2011). Some models such as SWMM, XPSWMM and MIKE 21 are providing range of hydrological methods to model surface routing aspects. However the surface routing in urban catchments by using these methods are questionable due to above cited complexities of them. Most of the recent case studies and research related to urban runoff modelling are based on two-dimensional surface runoff routing methods instead of the usual hydrological methods.

2.8.1. Modelling applications by using recent technologies

Recent technologies such as GIS data availability, which increased the availability of digitised spatial data, LiDAR data and remote sensing techniques, have shifted the traditional hydrological models into a new stage. The availability and accuracy of data with the new technologies reduces the modelling time extensively, makes them more user-friendly, reduces the possible input data errors and provides a new base to use complex modelling techniques. With the launch of 2D surface water routing modelling techniques, spatial data becomes a vital factor. 2D models are based on surface topography converted into a digital form widely known as digital elevation model (DEM) or digital terrain model (DTM). The use of spatial distribution of terrain related to its topography is being used to estimate the flood paths in 2D surface routing models. Fewtrell et al. (2011) used the terrestrial LiDAR data to generate a DEM, which most of the modelling software is capable of doing once they have been fed with LiDAR data. The LiDAR segmentation separates returning ground laser hits from surface object using classification algorithms and filters the ground objects by mixing with surface topography. To analyse and work with high resolution LiDAR data, high capability and efficiency of computers is vital. The level of accuracy of models is mostly described by the level of resolution used and the running time of such models can be extensively long unless they are run in suitable computers. Flood inundation modelling on urbanised floodplains has become more feasible due to the increased availability of high resolution digital terrain data and computer power (Neal et al. 2009).

Among the recent researches by using different modelling applications, Schumann et al. (2011) have carried out a case study to find accuracy of sequential aerial photography and space-borne synthetic aperture radar (SAR) data for flood dynamics based on UK town of Tewkesbury. They have concluded that the aerial photography and SAR data are capable of representing the important floodplain dynamics without distinguishing with other data even in a built-up area. Honghai and Altinakar (2011) cited a research work that resulted a decision support system for integrated flood management based on GIS. Their GIS based decision support system has proven its versatile and reliable capabilities for estimating various flood damage, and greatly enhance decision making process for future design of the flood proofing facilities. Fewtrell et al. (2011) have used the terrestrial laser scanning system for gathering the ultra high resolution elevation data. This data has been used in the spatial flood modelling which has been processed by highly preformed computers enabling to keep the accuracy of data by using small grid sizes.

Capabilities of coupling the hydraulic and hydrological models together with GIS based spatial models have been cited throughout the literature. Aggett and Wilson (2009) have coupled a high-resolution DTM with a 1-D hydraulic model in a GIS for scenario-based assessment of avulsion hazard in a gravel-bed river. HEC-RAS was used as 1-D hydraulic model and the results from hydraulic analysis were fed in to the DTM to illustrate the results' spatial variation over the cross section of the river bed. Sarhadi, Soltani et al. (2012) have coupled the GIS technologies with the statistic analysis to emphasis the probabilistic flood mapping in their study. They have used regional flood frequency analysis to estimate flood quantiles in different return periods at ungauged reaches. The hydro-geomorphic characteristics and the land use properties of the catchments were then extracted using RS&GIS techniques to establish multivariate regional regression models between hydro-geomorphic characteristics and flood quantiles. HEC-RAS was used as the 1D flood model and the GIS-based HEC-Geo RAS pre- and post-processor were used for careful optimization of the geometry features for real visualization of the flood prone areas.

Apart from above researches, Priestnall et al. (2000); Brown (2006); Coveney, Stewart et al. (2010); Tarekegn et al. (2010); Cook and Merwade (2009); Aguilar et al. (2010); Hladik and Alber (2012) also cited recent LiDAR and GIS applications in hydrological and hydraulic modelling. The usage of these new technologies is speeding up among the modern researchers with the improvement of the computer processor capabilities. Other than the time saving and user friendly techniques and accuracy of the data, the quality of representation of results by using these new applications is one of the most important features of modern modelling by using new technologies.

2.8.2. **XP stormwater management model (XPSWMM)**

XPSWMM One-dimensional (1D) engine is based on the EPA's Stormwater Management Model (SWMM) engine, with some modifications to it. The 2D component of the software is based on TUFLOW, which was developed to route 2D unsteady flows. XPSWMM has the ability to combine TUFLOW with its 1D engine and run as a comprehensive 1D/2D combined model (XPSoftware, 2009). XPSWMM is capable of analysing urban stormwater drainage behaviour and its changes with varying factors like percentage of imperviousness of the land use, infiltration capacities of the soil in the pervious areas, roughness coefficient of the materials used to build roads, canals, and pavements, development of basins within these areas and their capacities, etc. The approach to a numerical model and its success will depend on the accountability of all water entering, leaving and being stored in a catchment (Boughton, 2005). Also, in relation to urban flooding, the availability of aerial laser scanning of flooding areas and aerial photographs, and adaption of the 2D numerical models (Phillips et al., 2005) has been further extended to an urban inundation model, combining a storm sewer model with stormwater management models, two-dimensional (2D) diffusive overland-flow model and the operations of pumping stations (Hsu et al., 2000).

First the model will be run in its 'hydrological runoff' mode with given rainfall, soil type, land use and topography data. Then it will be combined with a hydraulics model with given data of existing USWMS. Then the model will be calibrated by using field data and the availability of distributed analytical data (Giuliano et al., 2009) and will be refined by changing and revising the existing data and properties.

The whole catchment will be analysed by combining the network models which are divided according to the small sub–catchments, due to the number of nodes limitation of the software. Finally, improvements to the existing hydraulics USWMS components can be suggested accordingly. Also, flood inundation vulnerable maps will be created with the results of combining the 1D and 2D analysis of the model. In addition, the conceptual analytical model will be used to perform the modelling of climate change impacts on catchments.

The modelling application of XPSWMM has varied from rural and urban catchment modelling to groundwater coupling in the recent past. Urban modelling approaches launched by using the software have been cited by Syme et al. (2004), Phillips et al. (2004), Smith et al. (2006), Dey and Kamioka (2007) and Dey (2010). Syme et al. (2004) have modelled an urban catchment by using the TUFLOW engine, which was later used to enhance the XPSWMM's 2D surface flow routing. The coupled model was supported by the DTM and GIS data. They have modelled urban areas features such as fences, highly varying land-use, buildings, narrow flow paths and underground stormwater drainage within the coupled model. Phillips et al. (2004) carried out urban stormwater management analysis by directly using the XPSWMM's comprehensive 1D and 2D capabilities. The 1D flow paths were modelled as link flows in a 1D layer, and 2D urban catchment was modelled in the 2D component. The building of DTM has helped to generate the results that represent the urban flooding inundation of a road network. They were capable of modelling the building and other high elevation structures as they affect the urban flow paths. The results of these case studies have shown the XPSWMM's capabilities of representing the 1D and 2D urban stormwater management features efficiently (Phillips et al. 2004 and Smith et al. 2006).

2.9. Summary of literature review

Natural pervious land use has been transferred to impervious land use with urbanization, reducing possible infiltration. Infrastructure with less surface roughness and unnatural flow paths along road networks, together with manmade drainage bypassing the natural flow paths, have decreased the time of concentration of urban catchments. These changes have caused the intensifying of runoff hydrograph peaks, increased the runoff quantity and reduced the urban runoff quality.

Removal of grass cover has led to deduction reduction of evapotranspiration, again increasing the amount of runoff by the long duration rainfalls. It also increases the time to recover the inundated areas. Urban catchments with shallow groundwater tables can have their stormwater drainage systems affected by storm events. An elevated shallow groundwater table causes submerged underground stormwater and sewer drainage. Climate change has intensified storm events and increased the frequency of their occurrence. All those phenomena have caused most of the urban stormwater management systems in urban cities to become outdated.

To prevent disasters such as urban flooding when urban stormwater management systems faces adverse increases in urbanization, land use change and climatic effects, analysis of urban hydrology and stormwater management systems is important. Urban development guidelines and stormwater management strategies have been developed worldwide to minimize the threat of disasters such as urban flooding, protecting lives, properties and making a liveable urban environment. Treatment of urban runoff closer to the source as much as possible, keeping controlled peak flows and storage to retain and detain runoff volume within the catchments while slowing down the runoff flow, are a few examples to such urban stormwater management strategies. The strategies used by different authorities in different places can vary depending on the climate, existing urban stormwater management system, level of urbanization, available technology and funding, etc. Best management practices and water sensitive urban designs are more frequently used terms which describe the controlled and nature-friendly stormwater management guidelines. The modern developments and subdivision works of urban cities are based on these guidelines to prevent disasters. Australia has its own stormwater management guidelines and strategies which address the water scarcity problem as a part of this, together with its main consideration of flood controlling measures. These guidelines provide both structural and non-structural measures for managing stormwater runoff.

Analysis of the combined (or even isolated) effects of all above mentioned phenomena upon urban stormwater management is a quite complex process. Such studies have commonly depended on urban stormwater management models and statistical analyses. There are numerous urban stormwater management models concentrated on one or few phenomena that have been created in recent history.

Selection of such a model to analyse an urban catchment hydrology can depend on catchment characteristics, available data and technology and expected results and their accuracy. The catchment characteristics most sensitive towards analysing the urban hydrological effect can be unique to each catchment and the analysing effect itself. Urban stormwater management models can be categorised under urban runoff quantity and quality. However there can be more additions to basic models, covering features to analyse climate effects and groundwater mounding, etc. In the past few years, models have been developed to represent the surface routing of overland flows, and associated storm sewer interactions, supported by high resolution topographic data, for example from LIDAR airborne remote sensing systems (Djordjevi´ et al. 2004). Spatial data generated for topography, drainage systems, land use changes, urban infrastructure and groundwater contours can be directly used with hydrological models using advanced computational technology. Those models increase the accuracy of input data, reduce the modelling time and ease the ability to do research by changing input data extensively. XPSWMM is a comprehensive urban stormwater management model which is capable of analysing the urban hydrology by combined 1D and 2D capabilities. It also can analyse the groundwater mounding simultaneous to the catchment runoff routing. By analysing all the components including surface runoff flow, 1D pipe flows and groundwater base flows, XPSWMM gives a complete hydrological package for urban catchment modelling.

CHAPTER 3

3. METHODOLOGY

3.1. Introduction

The study aims to analyse urban catchment hydrology and needs to assess complex urban catchment characteristics by numerical modelling. The XPSWMM stormwater and wastewater management model was selected as the modelling tool after an extensive literature review on current models and their applications. To research the stresses such as land use change and urban development effects on urban hydrology, three case studies have been selected:

1. Canning Vale drainage assessment
2. Victoria Park stormwater sump capacity assessment
3. Assessment of use of water sensitive urban designs (WSUD) and best management practices (BMP) in urban developments.

An urban catchment at Canning Vale in the City of Gosnells has been selected to analyse the effects of urban land development and land use change on the existing stormwater management drainage. Existing USWMS of the selected Canning Vale Central catchment comprises of underground pit and pipe network, detention and retention sumps, weirs, siphons, vegetated swales and open channels etc. It has been modelled by using XPSWMM and calibrated against observational data. The case study of Canning Vale represents the major research work of this study. The case study of Victoria Park urban catchment's stormwater sumps capacity assessment was done to assess runoff generation of an un-gauged urban catchment by using 2D surface water modelling techniques. The assessment helped to find the required stormwater sump capacities and results will be used to develop a master plan for land development in the area. The case study of assessment of use of WSUD and BMPs in urban developments has been carried out to analyse how sub-divisions and land developments can be managed to comply with stormwater management government policies and guidelines in Western Australia by copping with WSUD concepts and BMPs. The modelling process included the modelling of proposed urban catchment together with stormwater BMPs.

Several tasks have been carried out during these case studies, including activities such as; sensitivity analysis of catchment characteristics, calibration and verification of data, finding suitable methods to model different urban catchments, finding the groundwater effect on submerged stormwater drainage, 2D surface water modelling, finding methods to model stormwater BMPs, etc. The overall research methodology used for the study can be given as:

1. Detailed literature review to understand the urban hydrology, land use change and other anthropogenic effects on hydrology, urban stormwater management and concepts of WSUD and stormwater BMPs.

2. Study and understand the existing USWMS of selected urban catchments as an initial overview to find out the data availability and the gaps between data (field visits and investigations of drainage maps and other literature/information).

3. Collect basic catchment properties using recorded secondary data (geology, geography, and topography data).

4. Study and understand the local governments' stormwater management policies and guidelines (including limitations to stormwater peak flows and regulations in using BMPs).

5. Literature review on different modelling techniques, different stormwater management models and recent model applications to select suitable numerical model for the study.

6. Study and further understanding of the selected numerical modelling tool (XPSWMM numerical model). Develop urban stormwater management models, taking catchments' properties into account.

7. Carry out field data collection campaign (i.e. to identify the suitable data monitoring locations), infiltration tests and telemetric hydrological observation data collection process.

8. Sensitivity analysis to understand the behaviour of hydrological parameters in the model.

9. Calibration of models to understand the impacts of urban land use changes on urban catchment hydrology.

10. Verification of the models using collected long-term hydrological data and data from previous studies.

11. Hydrological assessment of urban catchments and USWMSs using developed and verified models, including the analysis of different rainfall scenarios and frequency thresholds.

12. Extension of the numerical models to identify the flood inundation conditions for the flood prone areas.

13. Develop potential flood distribution and flood vulnerability maps for flood prone areas. Flood mapping for different scenarios such as urban land development and land use change scenarios, groundwater scenarios and rainfall scenarios.

14. Combine the analysed results to facilitate decision-making tools on improvements for existing stormwater drainage network and to develop/provide appropriate adaptation mechanisms, recommendations, and specifications and to recommend the necessary improvements and BMP for urban stormwater systems considering WSUD guidelines.

3.2. Numerical modelling by using XPSWMM

XPSWMM is a stormwater and wastewater management model which has been used in numerous recent studies related to urban hydrology. XPSWMM is capable of analysing urban stormwater drainage behaviour with changes to the urban catchment characteristics. It is based on the United States' Environmental Protection Agency (EPA)'s Stormwater Management Model (SWMM) engine, with some modifications. The surface runoff of a catchment can be modelled in XPSWMM by using its hydrology layer (component/engine). It provides various routing methods and infiltration methods. A suitable surface routing method can be selected according to the catchment characteristics and data availability. The hydraulic analytical engine of XPSWMM, which is used for the 1D flow simulation, is based on the EXTRAN engine (XP Software, 2009). 1D hydraulic structures and flow paths can be modelled by using the 1D hydraulic component of the software. Other than these two components, XPSWMM has its two-dimensional (2D) component which is based on TUFLOW engine. TUFLOW was developed to route 2D unsteady flows. Its capability of solving shallow water finite differential equations was used in the XPSWMM 2D engine (XP Software, 2009). In addition to modelling hydraulic features, the software has the capability to understand the user input spatial data and represent them spatially within a model. The coupling of spatial data which are modelled and run based on 2D hydraulic layer, pit and pipe drainage network with data modelled in a 1D hydraulic layer is the major advantage of using XPSWMM model to represent the complex features of urban catchments.

Several approaches and techniques within the model can be used to model an urban catchment. In this study, the following two approaches were used to rout the surface runoff:

1. Hydrological surface runoff routing. Demands the common catchment characteristics such as catchment area, width, land use type and percentage and soil properties. The catchment properties can be fed in as numerical forms. It was found that this approach is highly accurate for solving catchments without complex urban infrastructure, which deteriorate the flow paths. Some runoff routing methods which can be used in this layer allow one to couple groundwater mounding with the channels and storage areas. Both surface runoff routing and sub-surface flows can be modelled simultaneously. The groundwater mounding application identifies the infiltration and percolation, but demands a number of soil property data.

2. Hydraulic 2D surface runoff routing. Demands highly accurate topography data to generate a digital terrain model (DTM) which represents the catchment in a three-dimensional space. The major benefit of this method is its capability for spatial representation of urban catchment characteristics and stormwater management features. Channels, stormwater basins, road surfaces, roundabouts, footpaths, traffic islands and bridges, buildings and fences, etc.

can be modelled with their actual specific representation. This approach can be used to model overland runoff in an urban catchment where complex urban flow paths exist and the time of concentration is affected. However the approach demands a good understanding and experience of 2D modelling, accurate spatial and topographic data and high-speed computers. Time steps, boundary conditions and configuration parameters that can cause erroneous modelling results are more likely than the hydrological surface runoff routing approach. Further, 2D hydraulics layers can also be used to generate spatial data maps to represent the water elevation contours and inundation polygons which are used to generate spatial flood inundation maps.

Hydraulic structures such as underground pit and pipe drainage, open channels, swales, retention and detention storage areas and BMPs such as treatment pockets, bio-retaining swales, soakage wells, etc. can be modelled by using different methods. They can either be represented as 1D components, or 2D components:

1. 1D hydraulic components: Hydraulic structures can be modelled as a combination of 1D nodes and links. Links represent the 1D flow paths such as pipes, open channels and swales etc., while nodes represent the pits (i.e. manholes), outlets and storage areas. Also pumps, weirs and other structures can be represented as 1D links or a combination of links.

2. 2D hydraulic components: The road surfaces and overland flow paths, channels and stormwater basins can be modelled as 2D hydraulic components. The structural spatial representation can be done by generating the DTM (automatically) or by user input spatial coordinates.

XPSWMM has the ability to combine TUFLOW with its 1D engine and run as a comprehensive 1D/2D combined model (XPSoftware, 2009). This study used these approaches and methods as combinations, when they were found to be appropriate to analyse different urban hydrological catchments.

3.2.1. **1D hydraulic flow routing**

EXTRAN is a hydraulic flow routing model for both open channel and closed conduits in dendritic and looped networks. The XPSWMM model performs dynamic routing throughout the major storm drainage system to the outfall points of the receiving water system by using EXTRAN. The EXTRAN Model will simulate branched or looped networks, backwater due to tidal or non-tidal conditions, free-surface flow, pressure or surcharge flow, flow reversals, flow transfer by weirs, orifices and pumping facilities, and pond or lake storage (XPSoftware 2009). The EXTRAN concept, integrated within XPSWMM modelling of routing inlet hydrographs through the network of pipes, junctions, and flow diversion structures of the main stormwater system to the receiving water outfalls was used in this study. EXTRAN is based on the shallow water St. Venant equations for gradually varied one-dimensional flow. The conservation form of shallow water equations can be given as follows:

$$\frac{\partial \eta}{\partial t}+\frac{\partial (\eta u)}{\partial x}+\frac{\partial \eta v}{\partial y}=0 \tag{1}$$

$$\frac{\partial (u\eta)}{\partial t}+\frac{\partial}{\partial x}(\eta u^2+\frac{1}{2}g\eta^2)+\frac{\partial (\eta u v)}{\partial y}=0 \tag{2}$$

$$\frac{\partial (\eta v)}{\partial t}+\frac{\partial (\eta u v)}{\partial x}+\frac{\partial}{\partial y}(\eta v^2+\frac{1}{2}g\eta^2)=0 \tag{3}$$

Where, u is the velocity in the x direction, or zonal velocity; v is the velocity in the y direction; H is the mean height of the horizontal pressure surface; η is the deviation of the horizontal pressure surface from its mean; g is the acceleration due to gravity; f is the Coriolis coefficient associated with the Coriolis force, on Earth equal to $2\Omega\, sin(\varphi)$, where Ω is the angular rotation rate of the Earth ($\pi/12$ radians/hour), and φ is the latitude; b is the viscous drag coefficient.

It also uses the Manning equation and kinematic wave equation when there are special cases in 1D simulation. The Gauckler–Manning formula can be given as (Gioia and Bombardelli 2001);

$$V=\frac{k}{n}R_h^{\frac{2}{3}}.S^{\frac{1}{2}} \tag{4}$$

Where, V is the cross-sectional average velocity (L/T; ft/s, m/s); k is a conversion factor of 1 L1/3/T, m1/3/s for SI, or 1.4859 ft1/3/s; n is the Gauckler–Manning coefficient, which is unit-less; R_h is the hydraulic radius (L; ft, m); S is the slope of the water surface or the linear hydraulic head loss (L/L) (S = hf/L)

The non-linear kinematic wave for debris flow can be written as follows, with complex non-linear coefficients (Pudasaini, 2011):

$$\frac{\partial h}{\partial t} + C\frac{\partial h}{\partial x} + \frac{\partial^2 h}{\partial x^2} = 0 \qquad (5)$$

Where, *h* is the debris flow height, *t* is the time, *x* is the downstream channel position, *C* is the pressure gradient and the depth-dependent nonlinear variable wave speed, and *D* is a flow height and pressure gradient-dependent variable diffusion term.

The modern versions of EXTRAN integrated to XPSWMM are consistent with a combination of implicit and explicit finite difference formulations for solving the nodal continuity equation, combined conduit momentum and continuity equation, and the boundary conditions of the solved network. The drainage can be represented within the model as links transferring flow from node to node. Properties associated with the links, which are either user input to the model or calculated by the model by using the given inputs include roughness, length, cross-sectional area, hydraulic radius, conduit depth, and surface width. Velocity, hydraulic radius, and the cross-sectional area of flow, or depth, are variable in the link and computed at the upstream and downstream ends of the conduit (XPSoftware 2009).

The EXTRAN Model uses the momentum equation in the links and a special lumped continuity equation for the nodes. The basic unsteady flow continuity equation with lateral inflow is (Yen 1986):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (6)$$

In the equation, cross-sectional area *A* and flow *Q* exist as dependent variables and *q* is the lateral inflow. In EXTRAN, lateral inflow is zero and the inflows enter the network at the nodes. The conduit momentum equation may be written in several forms depending on the choice of dependent variables and by using dependent variables flow and hydraulic head *H*, the momentum equation is written as (XPSoftware 2009):

$$\frac{\partial Q}{\partial \mathrm{t}} + \partial\frac{\frac{\mathrm{Q}^2}{\mathrm{A}}}{\partial \mathrm{x}} + \mathrm{gA}\frac{\partial \mathrm{y}}{\partial \mathrm{x}} + \mathrm{gA}\left(\mathrm{S}_{\frac{\mathrm{e}}{\mathrm{L}}} + \mathrm{S_c} + \mathrm{S_f} + \mathrm{S_o}\right) = 0 \qquad (7)$$

The terms in the momentum equation are, respectively: Local inertia $\partial Q/\partial t$, Convective inertia $\partial(Q^2/A)/\partial x$, Pressure slope $g \cdot A \cdot \partial y/\partial x$, Entrance/exit loss $S_{e/L}$, Contraction/expansion loss S_c, Friction slope S_f, and Bed slope S_o, where L is distance along the conduit and A is conduit cross-sectional area. The same equation can be modified as:

$$\frac{\partial \mathrm{Q}}{\partial \mathrm{t}} + \frac{\mathrm{gkQ}\,|\mathrm{Q}|}{\mathrm{R}^{\frac{3}{4}}} - \frac{V\,\partial \mathrm{A}}{\partial \mathrm{t}} + \frac{\mathrm{Q}\,\partial \mathrm{V}}{\partial \mathrm{x}} + \frac{\mathrm{gA}\,\partial \mathrm{H}}{\partial \mathrm{x}} = 0 \qquad (8)$$

Where, R is the centre hydraulic radius, R_{up} and R_{dn} respectively the upstream and downstream hydraulic radius, $k = (n/1.49)^2$ for U.S. customary units and n^2 for metric units.

Expressing equation (9) in fully implicit finite difference form (i.e., all Q values are at the $t+\Delta t$ time step, or $\theta=1$) the final finite difference form of the fully implicit dynamic flow equation (excluding terms for Sc and Se) can be given as (XPSoftware, 2009):

$$Q_{t+\Delta t} = \frac{\left[Q_t + V\frac{\Delta A}{\Delta T}.\Delta t - \frac{gA\left(H_{dn} - H_{up}\right)}{L}.\Delta t\right]}{\frac{gK\,|V|\,\Delta t}{R^{\frac{4}{3}}} - \frac{\left[Q_t\left(\frac{1}{A_{up}} - \frac{1}{A_{dn}}\right)\Delta t\right]}{L}} \qquad (9)$$

Δ_A/Δ_T is the average area time derivative from time step n. The time step selected should not be greater than the minimum value for any channel (except non-inertial channels such as bridges, culverts, etc). Accuracy of the results is also influenced by time step. The limiting value adopted is usually a compromise between accuracy, stability and simulation time, and sensitivity checks are recommended. The occurrence of mass errors may indicate the use of too high a time step (TUFLOW 2010).

3.2.2. Hydrological surface runoff routing

There are number of methods facilitated within the XPSWMM hydrology layer for surface runoff routing, which include (XPSoftware 2009):

1. SWMM Runoff Non-linear Reservoir Method
2. Kinematic Wave Method
3. Laurenson Non-linear Method/Rafts
4. SCS Unit Hydrograph Method
5. Other Unit Hydrograph methods: Nash, Snyder (Alameda), Snyder, Rational Hydrograph, Time/area, and Santa Barbara Urban Hydrograph.
6. Rational Formula
7. UK Hydrology: New UK, Wallingford, ReFH, FEH, FSR

The methods' suitability for different catchments can be dependent on catchment behaviour and data availability. Two methods were considered during this study: the SWMM Runoff Non-linear Reservoir Method and the Laurenson Non-linear Method (Laurenson 1964). The SWMM Runoff method has to be used when groundwater mounding is expected to be run simultaneously to the runoff routing. The Laurenson Method can be used during urban surface runoff routing methods where specific land use data is present, but it cannot mound the groundwater simultaneously.

<u>SWMM Runoff Non-Linear Reservoir Method</u>

In this method, sub-catchments are modelled as idealized rectangular areas with the slope of the catchment perpendicular to the width. Each sub-catchment is classified into three (or four when one counts the snow melting) sub-areas, as indicated in the following table (XPSoftware 2009).

There are other factors, such as snow melting, during the routing process, but they are not described here since they were not used in this study. Flow from each sub-area moves directly to a node isolated from other sub-areas. The width of the pervious sub-area, *A2*, is the entire sub-catchment width, whereas the widths of the impervious sub-areas *A1* and *A3* are in proportion to the ratio of their area to the total impervious area (XPSoftware 2009). Sub-catchments are analysed as spatially lumped non-linear reservoirs (Rossman 2004). The routing is performed separately for each of the sub-areas within the sub-catchment (attached to a node).

Table 1. Catchment classification under SWMM runoff non-linear reservoir method.

SUB AREA	PERVIOUSNESS	DEPRESSION STORAGE
A1	Impervious	Yes
A2	Pervious	Yes
A3	Impervious	No

There are inflows coming from precipitation and any designated upstream sub-catchments and there are several outflows, including infiltration, evaporation, and surface runoff. The capacity of a sub-area or "reservoir" is the maximum depression storage d_p, which is the maximum surface storage provided by ponding, surface wetting, and interception. Surface runoff per unit area, Q, occurs only when the depth of water in the "reservoir" d exceeds the maximum depression storage, d_p, in which case the outflow is given by Manning's equation. Depth of water over the sub-catchment is continuously updated with time t by solving numerically a water balance equation over the sub-catchment (Rossman 2004). The sub-catchment routing Manning's equation can be given as:

$$Q = W\,\frac{1.49}{n}\left(d - d_p\right)^{\frac{5}{3}}.S^{\frac{1}{2}} \tag{10}$$

Where, Q is sub-catchment (or sub-area) outflow; W is sub-catchment width; n is Manning's roughness coefficient; d is water depth; d_p is depth of depression storage and S is slope.

Laurenson Method

The Laurenson method (Laurenson 1964), integrated in the software was used for the surface runoff routing of urban water sheds which are linked with a 1D pipe network, but not influenced by the groundwater. When using Laurenson hydrology the sub-catchment width is by default not used. The percentage of imperviousness of land uses were given by adding separate sub-areas of '0 per cent urbanized' for bare land use and ‘100 per cent urbanized’ for impervious land use. All the land use percentages were calculated manually to divide them into these two categories. Routing for a particular sub-catchment is carried out using the Muskingum procedure. The storage, however, is a non-linear function of the discharge:

$$s = K(q) \times q \tag{11}$$

Where, s is volume of storage, (hrs × m³/s), q is instantaneous rate of runoff, (m³/s), *K(q)* is storage delay time as a function of q (hours). Each sub-area is treated as a concentrated conceptual storage. Each storage point has a storage delay time described thus:

$$K(q) = B_q^n \tag{12}$$

Where, B is storage delay time coefficient and n is storage non-linearity exponent. Finally equations (13) and (14) can be written as:

$$s = B_q^{(n+1)} \tag{13}$$

The default value for the non-linearity exponent n is (-0.285) and this is used during this study.

3.2.3. Hydraulic surface runoff routing

The 2D hydraulic surface routing method in XPSWMM uses the shallow water equation to route the surface water runoff. TUFLOW is being used as an integrated engine. It is based on a fully 2D solution algorithm which solves the full two-dimensional, depth-averaged, momentum and continuity equations for free surface flow (XPSoftware 2009). The model has the capability of coupling the 1D hydraulic engine together with 2D surface runoff engine to act as a comprehensive stormwater management model. This capability is used in the study to represent the urban catchment’s properties such as pit and pipe networks, roads, infrastructure and buildings with different elevations, surface flow paths, etc.

To analyse the surface runoff by using a 2D hydraulic layer following data is required (XPSoftware 2009):

- A DTM with sufficient resolution and accuracy to depict the topography of all flow paths and storage areas in the 2D domain(s). The vertical accuracy depends on the modelling objectives and budget constraints. However, for large scale models ± 0.2 m is preferred, whilst for fine-scale urban models < ± 0.1 m is recommended.

- Cross-sections for any 1D flow paths.

- If bed resistance varies over the model, geo-corrected aerial photography or other GIS layer from which material (land-use) zones are digitized for calculating Manning’s n values.

- Boundary conditions (e.g. ocean water levels, catchment inflows, rainfall, evaporation, etc).

- Calibration data locations as points in a GIS layer. Peak levels should be attached as attributes to the calibration points.

- Surveys of key hydraulic controls such as levees / embankments (3D break-lines), culverts, bridges, etc.

The 2D shallow water equations which are solved during the 2D hydraulic surface runoff can be described in relation to the horizontal plane by the following partial differential equations of mass continuity and momentum conservation in the X and Y directions (TUFLOW 2010);

- 2D Continuity:

$$\frac{\partial \zeta}{\partial t} + \frac{\partial (Hu)}{\partial x} + \frac{\partial (Hv)}{\partial y} = 0 \qquad (14)$$

- X Momentum:

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} - c_f\, v + g\frac{\partial \zeta}{\partial x} + g\, u\left(\frac{n^2}{H^{4/3}} + \frac{f_l}{2g\Delta x}\right)\sqrt{u^2 + v^2} - \mu\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2}\right) + \frac{1}{\rho}\frac{\partial p}{\partial x} = F_x$$

(15)

- Y - Momentum:

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+c_f u+g\frac{\partial \zeta}{\partial y}+g\,v\left(\frac{n^2}{H^{4/3}}+\frac{f_l}{2g\Delta y}\right)\sqrt{u^2+v^2}-\mu\left(\frac{\partial^2 v}{\partial x^2}+\frac{\partial^2 v}{\partial y^2}\right)+\frac{1}{\rho}\frac{\partial p}{\partial y}=F_y \quad (16)$$

where

$\zeta = Water\ surface\ elevation$

$u\ and\ v = Depth\ averaged\ velocity\ components\ in\ X\ and\ Y\ directions$

$H = Depth\ of\ water$

$t = Time$

$x\ and\ y = Distance\ in\ X\ and\ Y\ directions$

$\Delta x\ and\ \Delta y = Cell\ Dimensions\ in\ X\ and\ Y\ directions$

$c_f = Coriolis\ force\ coefficient$

$n = Manning's\ n$

$f_l = Form\ (Energy)\ Loss\ coefficient$

$\mu = Horizontal\ diffusion\ of\ momentum\ coefficient$

$p = Atmospheric\ pressure$

$\rho = Density\ of\ water$

$F_x\ and\ F_y = Sum\ of\ components\ of\ external\ forces\ (eg.\ wind)\ in\ X\ and\ Y\ directions$

For the 2D scheme, the Courant number generally needs to be less than 10 and is typically around 5 for most real-world applications (Syme 1991). The computation time step in the 2D model should be given according to the used grid size to comply with the following equation:

$$C_r = \frac{\Delta t\sqrt{2gH}}{\Delta x} \quad (17)$$

Where, Δt is time step, Δx is length of model element, g is acceleration due to gravity, H is depth of water.

As a rule, the time step is typically half the cell size. For steep models with high Froude numbers and supercritical flow, smaller time steps may be required. It is strongly advised by TUFLOW (2010) and XPSoftware (2009) to not simply reduce the time step if the model is unstable, but rather to establish why it is unstable and, in most instances, correct or adjust the model topography, initial conditions or boundary conditions to remove the instability. If the model is operating at high

Courant numbers (>10), sensitivity testing with smaller time steps to demonstrate no measurable change in results should be carried out. The occurrence of high mass errors is also an indicator of using too high a time step (TUFLOW 2010). When coupling the 1D engine with the 2D engine it is highly preferable that the 1D domains do not control the time step, as 99 per cent of the computational effort is usually in solving the 2D domains (TUFLOW 2010).

The model determines the wet and dry cells by using the depth initiated by the user. To analyse urban flooding, a 0.002 m depth was assigned during the study, considering the model stability and required water depths (i.e. a water depth below 0.1 m was neglected during the flood inundation map generation). The method of keeping the Viscosity Formulation as a constant was used. This applies a constant value throughout the model, irrespective of velocity gradients and variations. This is generally satisfactory when the cell size is much greater than the depth or when other terms are dominant (e.g. high bed resistance). The recommended coefficient for the constant formulation is 1 m^2/s (TUFLOW 2010).

3.2.4. Modelling groundwater interaction

Groundwater drains through the road drainage network when the groundwater level reaches to the level of the drainage. Groundwater can be leaked to the drains from defective water-tightness of joints (Berthier et al. 2004) and the unsealed bottoms of manholes. The groundwater table is always dynamic and changing characteristics with seasonal variations, showing a quick response to heavy rains. When the groundwater level rises to the surface and infiltration is stopped and the drainage network is occupied by groundwater, this will lead to all the rainfall flowing as surface runoff. If it drops below the bottom elevation of the drainage network, groundwater outflow will cease (XP Software 2009). The Canning Vale catchment is water-logged in some areas, especially near the ponds and swales and the drainage network is submerged during the rainy season. The field results show that even if there was no rain for months, still some of the outlets show a water flow due to groundwater outflow to the drains. Therefore it was important to consider the groundwater impact and model it together with the surface flow.

The concept of three reservoirs, identified as root zone, percolation zone and saturated zone, was used widely in the literature to model groundwater interaction (Berendrecht et al. 2006). Figure 2 shows the concept of the three reservoirs method. In this study, the groundwater outflow to the submerged manholes and basins is modelled using a similar concept to the above by using the software's integrated option for groundwater flow mounding. In the root zone, the evapotranspiration was routed. The short durations of rainfall events, which were considered to evaluate the critical conditions for flooding events, led to neglecting the effect of evapotranspiration contribution to the final results. In the percolation zone, the water infiltrates to the unsaturated zone and then percolates to the saturated zone.

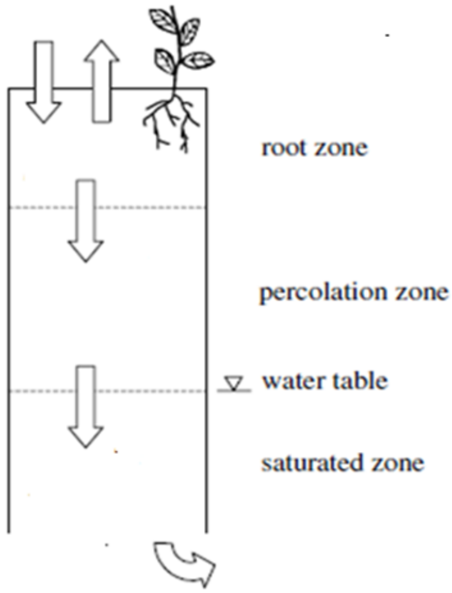


Figure 2 Three reservoirs counted for groundwater modelling

Finally the water from the saturated zone to the drainage was routed. As shown in equation 18, linear reservoir routing was used by only giving the groundwater flow coefficient *C*, and groundwater flow exponent *b* which is used by the software to rout the groundwater flow mounding from the saturated zone to the drainage network. The rest of the equation was neglected by letting the relevant coefficients be zero (XP Software 2009):

$$Q= C \times D \times b \tag{18}$$

Where, Q is groundwater inflow to the manholes, and D is the depth of groundwater table above the relevant manhole's bottom level. After the groundwater flowed to each sub-catchment node, it combines with the water flow in the drainage system, where routing was done by the normal hydraulic routing method mentioned above. The following assumptions were used in the modelling:

- There is no groundwater interaction with the stormwater pipe network through the pipe joints, and the only interaction is through manhole bottoms.
- Groundwater flow to the pipe network system will not be negative, and the only possibility is for the flow to become zero, when the groundwater level goes below the manhole invert level (XP Software 2009).
- There is no groundwater dissipation out of the system.

The levels of starting groundwater tables for the calibration and model run under different scenarios were selected by using observation data from the City of Gosnells and the groundwater atlas from the Department of Water. There can be a huge effect on groundwater levels when the groundwater is extracted manually by pumps for different usages. So, when considering the observation data, it was confirmed that there is no manual extraction that has happened during the related time period.

3.3. Infiltration rating curves

Infiltration was used during the surface runoff routing and the infiltration bio-retentions and retention basin modelling. XPSWMM integrating the initial and continual loss method was used during surface runoff routing. This method is based on the direct measurements of initial loss and continuous loss of the catchment's soil. The values were taken from the literature and sometimes from field tests. On the other hand, the infiltration loss due to ponding in storage devices was counted during modelling and they were incorporated as modelling components.

The storage capacities within the model were developed by giving rating curves for their areas (i.e. for the rectangular basins, varying the surface area against depth was used). The infiltration from those basins was given as an outflow which is lost from the model. The infiltration outflow was represented by using the rating curve which varies with water depth. To calculate the infiltration rate from a basin, Darcy's flux, as given in the following equation, was used (Chow et al. 1988):

$$q = -K\frac{\partial h}{\partial z} \quad (19)$$

Where, q is Darcy's flux, K is hydraulic conductivity of the soil, h is water head and the z is distance in z direction (vertical, in this case). The term of $\partial h/\partial z$ can be given as rate of head loss per unit length (it is negative because the head is decreasing towards the flow direction). The equation is one-dimensional and can be applied for the unsaturated porous media just below the surface (Chow et al. 1988). Considering the ponded condition of basins and the higher infiltration rates of soil in the basin bottoms, the soil diffusivity, which is a property of water suction head of soil mentioned in the Richards equation (Richards 1931), was neglected. The hydraulic conductivity was assumed to be constant with the water depth. The hydraulic conductivity for each soil type was taken from literature.

Stormwater runoff carries large amounts of contaminants and suspended solids that tend to accumulate in infiltration basins and form a sedimentary layer covering the soil surface and this may impact on water infiltration (Lassabatere et al. 2010). Some studies such as Urbonas and Stahre (1993) have considered the bottom of infiltration storage areas is impervious, which led to the assumption of that the effective infiltration area is equal to one-half the area of the vertical sides of the storage area. The justification for this was suggested as being that the bottom of the trench seals quickly by the accumulation of sediments. Also Cordery and Pilgrim (1983) cited the using of a factor of safety of 1.5 to reduce the final infiltration rate of the soil below the retention systems with to a minimum for design purposes. This study uses a clogging factor of 0.5 (50 per cent reduction of infiltration) for the bio–retention areas, while the clogging of stormwater retention and detention basins (depth is more than 1 m) was neglected. The increasing infiltration surface area of a storage device with its depth was considered (i.e. infiltration area = $A(h)$) and calculated by using the side slope of the basin. After these assumptions, the study used following equation, which is derived from equation (20) and represents the infiltration rate Q of a basin when its water depth is h. The hydraulic gradient $\partial h/\partial z$ is taken as 1, since the water head is decreasing vertically and head loss is equal to the water travelled distance:

$$Q = CKA(h) \tag{20}$$

The clogging factor was given as *C*, i.e. this study used *C* as a design safety factor and, for the infiltration basins (where depth is higher than 1 m), *C* = 1 and for bio–retention areas and other low depth retention basins *C* = 0.5. The vertical surface area was accounted for as infiltration when designing soak wells, together with a safety factor *S* for the infiltration and the above equation was modified as follows: where A_b is bottom surface area and A_v is vertical surface area. Value of *S* was given as 0.5 as a design rule of thumb.

$$Q = KA_b + SKA_v \tag{21}$$

3.4. IFD rainfall data and critical duration events

Engineers use Intensity Frequency Duration (IFD) data to estimate the design rainfalls which are used for purposes related to stormwater management (i.e. to determine the required flood capacity of a stormwater basin). IFD rainfall data represent rainfall statistics and can be generated by statistical analysis of past recorded rainfall data. The rainfall can be measured in terms of depth related to a period of time (duration). The duration can be 1 month, 1 year or several years and decided upon based on the required use of the rainfall data statistics. To compare the severity of different rainfall events (described in terms of a depth of rainfall over a certain duration), the frequency (Average Recurrence Interval or ARI) of an event is important. Intensity of a rainfall is calculated by dividing the depth by the duration. The rainfall data recorded as IFD statistical data might be needed in several standard durations. For example, 1 year (frequency) 1 hour (duration) ARI event rainfall data is needed to design water quality BMPs according to the Western Australian stormwater management guidelines. Therefore rainfall data for the catchment that describes several standard duration events with their possible occurring frequencies for one area is the key to stormwater management designs. The IFD rainfall data for this study was obtained from the Bureau of Meteorology (2012), which has stored the data on a 0.025^{o} latitude by 0.025^{o} longitude grid (approx 2.5km by 2.5km) covering Australia. These rainfall data were recorded over 20 years for Australia (BoM 2012). The following equation gives the rainfall intensity calculation procedure used by the BoM (2012):

$$log_e i = A + Blog_e T + C(log_e T)^2 + D(log_e T)^3 + E(log_e T)^4 + F(log_e T)^5 + G(log_e T)^6 \quad (22)$$

Where, *i* is rainfall intensity (mm/hr) and *T* is time in hours and *A, B, C, D, E, F, G* are coefficients calculated by an algorithm.

The critical duration for any ARI rainfall event should be found to design hydraulic structures (for example, channel cross-sections and stormwater storage capacities) and for any development. Usually the design critical duration for 5 year and 100 year ARI events for land developments are found by using the pre-developed catchment and then the same duration was applied to propose the hydraulic structures' design capacities for post-development catchment. The critical duration event for designing the volume of stormwater storage basins, for example, can be varied depending on the infiltration rate and outflow from the basin other than the catchment characteristics. Therefore the study suggests to use 11 standard durations that vary from 10 minutes to 72 hours in the modelling to create runoff hydrographs (either volume or flow–rate, depending on the purpose) to decide the critical duration for each ARI event. The critical duration was solely calculated for each scenario by using the model in this method.

CHAPTER 4

4. SENSITIVITY ANALYSIS, CALIBRATION, AND VERIFICATION OF THE NUMERICAL MODEL

4.1. Introduction

Understanding the sources of uncertainty in stormwater management models and their consequences for the model outputs is essential so that subsequent decisions are based on reliable information (Wagener et al. 2004). Also to run and generate the results effectively and accurately, sensitivity analysis of modelling parameters and catchment characteristics is important. Catchment characteristics such as surface roughness coefficients, infiltration values (both initial and continual), and characteristics that create a lag time for runoff and the groundwater effect can be changed according to the land use types. Various parameters used in a model, which the model results are sensitive to, can be estimated using different approaches including *a priori* estimates using look-up tables (e.g., for physically-based soil parameters), manual and/or automatic calibration using optimization algorithms, and using transfer functions between similar basins (Abebe et al. 2010). Also the Western Australian Planning Commission (2009) cited there are several methodologies to do sensitivity analysis and listed two popular approaches: the Generalized Likelihood Uncertainty Estimation (GLUE) methodology which is also known as pseudo-Bayesian or informal Bayesian, and the formal Bayesian methods, such as Monte Carlo Markov Chain (MCMC) methods. Since the major aim of this study is to find the overall effect of land use change on urban hydrology, the sensitivity of each of these parameters affects the final results of the study. Therefore sensitivity analysis of selected catchment characteristics was done for the urban catchment model. Also sensitivity analysis for model performances was carried out using two different modelling techniques.

The number of uncertainty parameters for several catchment characteristics involved in rainfall runoff modelling can make the model behaviour and results very variable from the actual conditions. The calibration of modelling parameters is required to make the model as close as possible to the catchment behaviour and to match the results as closely as possible to the observation data. The modelling parameter and catchment characteristics calibration of a conceptual model usually involves multiple criteria for judging the performance of observed data (Town of Kwinana 2005). Calibration after sensitivity was done by using observational data and model results treating selected parameters carefully. The sensitivity analysis helped the calibration process by reducing the number of variables during the process and reduced the needed effort and time spent during the calibration. The selected characteristics were changed according to their sensitivities in order to match the modelling results and observational data at a selected outflow location. The range of values for each catchment characteristic was derived from the literature. The values were changed until the model results were sensible compared to the observational data. The models were calibrated by using their reservoir water depths and out flows.

Groundwater was considered as a major parameter during the calibration of the shallow groundwater urban catchment models. Other than those, catchment parameters changes according to the land use categories such as surface roughness coefficients and infiltration values were treated as calibration parameters. Some parameters, such as downstream outlet water depth, were fixed by using the relevant observational data.

The calibrated model was verified by using a different time period and different rainfall (independent from the calibration rainfall) to make sure the model is suitable to assess urban catchments with similar characteristics.

4.1. Selecting the best modelling technique for urban flood modelling

Selection of the best suitable modelling technique to model urban catchment hydrology and USWMSs was important to get accurate results and to reduce the modelling time and effort. Avenues catchment in Canning Vale was selected as the gauged urban catchment. It was modelled under two different numerical modelling approaches for surface overland runoff routing; the hydrological surface routing approach and the hydraulic surface routing approach (introduced as the 2D surface routing in the software). Both methods shared a common 1D drainage flow routing method and a common linking method of surface runoff and 1D drainage flows. The pipe drainage network was modelled and surface runoff was combined with the drainage system through manholes for both cases. A 2D hydraulic layer was common to both cases. The link between the 2D hydraulic layer and the 1D drainage was initiated through manholes. Stormwater from the 2D hydraulic layer was allowed to inflow or overflow to the drainage, depending on relative water pressure of the two layers at any location in any time step. The water depth in the 2D layer at any location, at any time, represented the spatial flood inundation of the area. The drainage network was uploaded as spatial data while 2D land use categories were represented spatially. A DTM was created by using 1 m interval contour topography data. Grid size (6 m x 6 m in this case) and time step (2.5 seconds) were common to both methods.

The main difference between these two approaches was their routing method. The hydrological approach routed the surface runoff from catchments by using the Laurenson runoff routing method, while the hydraulic approach used 2D shallow water equations to rout the surface runoff (TUFLOW engine). In first method, the 2D engine was used only to rout excess water from manholes. The catchment was divided into small sub-areas and they were linked as sub-catchments to the manholes. Catchment characteristics such as area, width and slope were determined manually. Land use categories were fed in as numerical area percentages and catchment characteristics were given in the hydrology layer. Rainfall was given in the hydrology layer as a hydrograph and surface runoff was routed into the manholes (by the Laurenson hydrological routing method) before it overflowed into the 2D layer (if there was not enough space in the drainage system). The surface runoff routing and the excess water runoff routing were carried out simultaneously.

In the hydraulic method there was no hydrology layer used and both the catchment runoff routing and excess water routing was done in the 2D hydraulic layer. The catchment characteristics were given according to the spatially represented land use areas. Rainfall was given as a hydrograph to the 2D layer. The catchment characteristics such as area, width and slope of the catchment were derived automatically according to the DTM. The results for two approaches were calibrated and verified against observational data. The results are discussed under the calibration section. However, they show that the both methods are capable of representing the urban catchment, but the hydrological approach was more accurate than the hydraulic approach.

Flood vulnerability maps spatially indicate the level of possible flood inundation in a catchment by means of a variable colour code. The average recurrent interval flood events of 1 in 5 years for the drainage network design guidelines, 1 in 10 years for the public open spaces guidelines and 1 in 100 years for the flood vulnerability maps generation has been considered. The level of flood risk has been identified with relation to the flood water level. In this study the 0.1 m level was considered to be the maximum inundated water level that can exist within an urban area. It is proposed that areas where the flood inundation water levels exceed this be treated as sensitive areas during future developments.

The 1 in 100 year average recurrent interval flood event was modelled for the two approaches. The historical rainfall data were obtained from the intensity - frequency - duration curves (Pilgrim 1987). The results are shown in Figure 3 and Figure 4 for both cases. The flood vulnerable area for both cases is almost identical and rages from 0.10 m to 0.78 m flood depths. There are flood water heights showing above 0.782 m for some clusters. These can occur due to the coarseness of the topographical data and the 6 m x 6 m grid spacing may not be able to represent exact topographical variations of areas less than the grid size. The water depth of the basin shows as 0.283 m due to the initial water levels given. The hydraulic approach can be more suitable, since it counts water from surface runoff and excess water from manholes at the same time, when simulating the flood depths. In the method used in the hydrological approach the surface water is routed to the manholes first, and then the excess water from manholes enters the 2D network. This process has a lag time, and the surface runoff which is routing through the hydrological layer will not simulate the flood depths until they overflow from manholes. However, the coarseness of the topographical data may cause some inconsistencies for the 2D hydraulic routing process. The level of inconsistencies is higher in the hydraulic layer than the hydrology layer. The overall flood depth representation is adequate, however and further enhancement can be done by using topographical contours with finer topographical data.

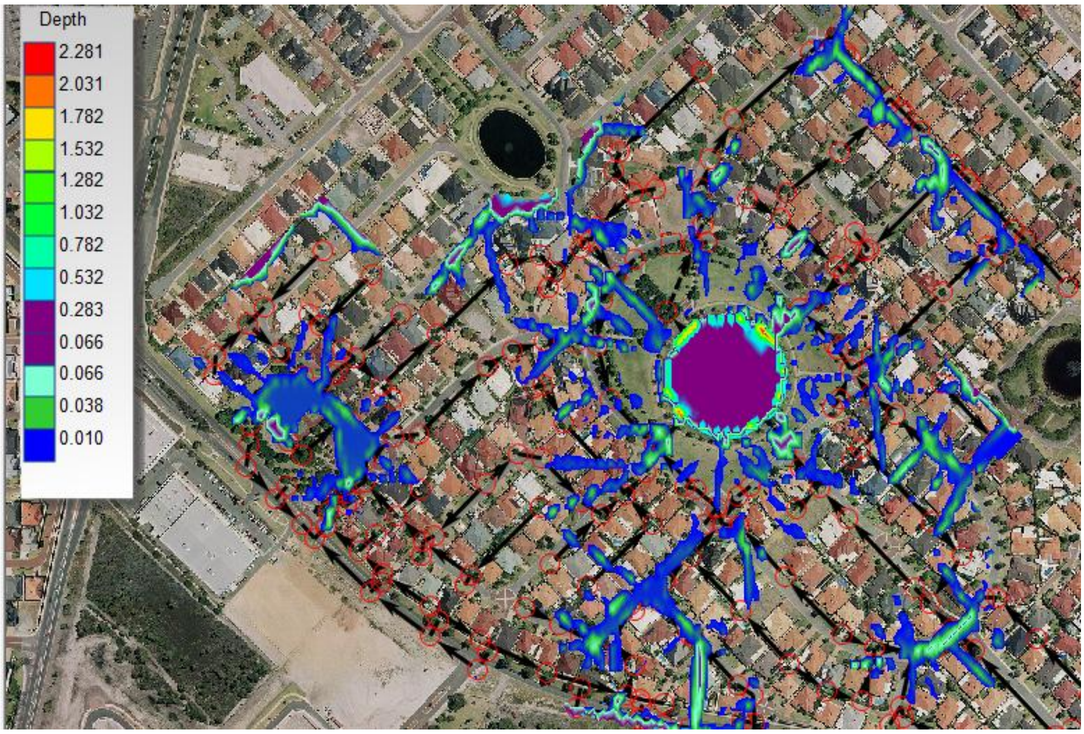


Figure 3. Flood inundation mapping for 1 in 100 year flood event: Hydrologic approach

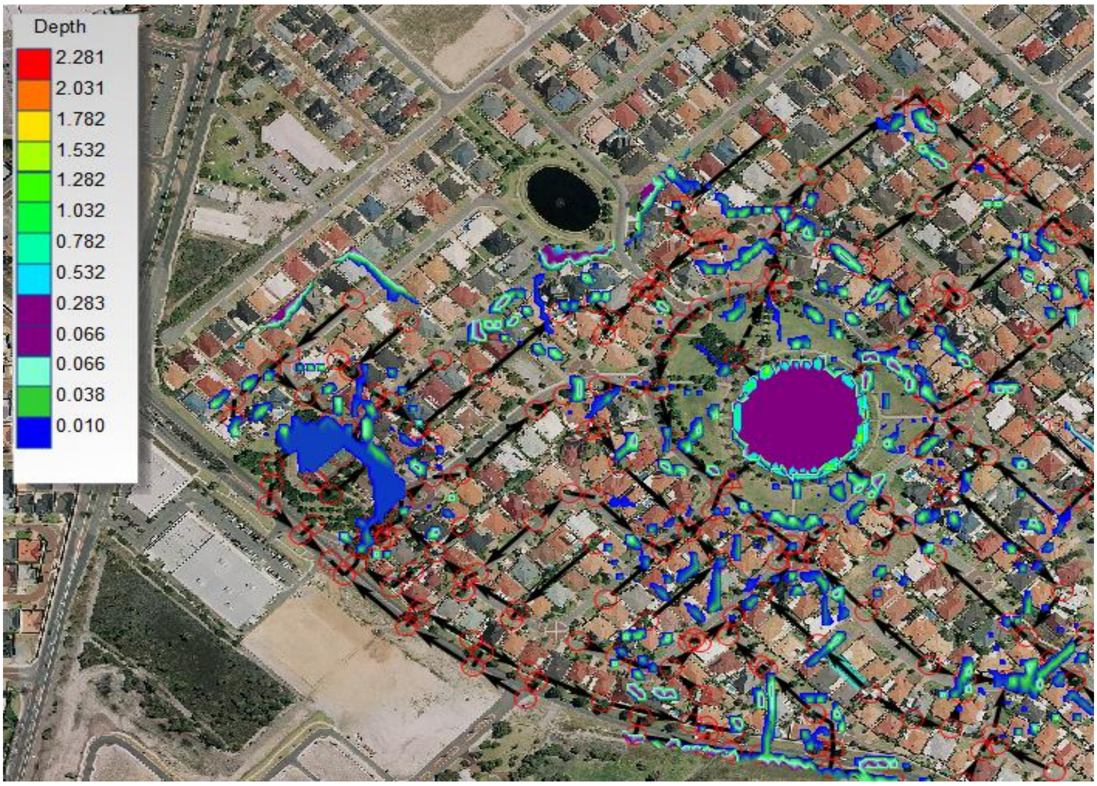


Figure 4 Flood inundation mapping for 1 in 100 year flood event: Hydraulic approach.

4.2. Sensitivity analysis of catchment characteristics

The sensitivity analysis has been carried out based on four catchment characteristics which are dependent on land use changes. One of the urban sub-catchments of Canning Vale, called Glenariff, sub-catchment was used as the modelling catchment. Urban watercourses and flood plains were modelled as a combination of 1D (watercourses) and 2D (floodplain) elements. Sub-catchments based on drainage manholes were used in the hydrology layer to count the surface runoff. Initial values for catchment characteristics such as area, percentage of imperviousness, slope, infiltration rate, depression storage, Manning's surface roughness and percentage of zero detention were fed into the model. The SWMM Runoff Non-linear Reservoir Method was used during the hydrological surface routing. The surface runoff routed into manholes was allowed to flow through the 1D pit and pipe network (underground stormwater drainage). The 1D flow analysis was done in the hydraulics layer. The excess water from the drainage system was allowed to overflow through the manholes into the 2D hydraulic layer. The overland flow due to excess water was routed through the 2D hydraulic layer until there was space in the drainage network again. The manholes were treated as existing as a door between the 1D drainage and the 2D surface. All the analytical iterations in between the three layers (hydrology, hydraulics 1D and hydraulic 2D) were run simultaneously.

The 2D grid size was used as 5 m x 5 m, which was accurate enough to represent the hydraulic features spatially and the time step was used was 2.5 seconds. Basic modelling parameters and catchment characteristics were fixed during the sensitivity analysis routing attempts, which were done by changing the catchment characteristics one at a time. The initially fixed variables and their values are given in Table 2. The following four catchment characteristics depend on their land use categories and were changed one at a time for the two rainfall scenarios to find the sensitivity of each characteristic to the peak stormwater outflow.

- Surface roughness values of impervious area
- Depression storage values of pervious area
- Infiltration loss of pervious area
- Zero detention percentage of impervious area

Table 2. Initial values of catchment characteristics and modeling parameters.

PARAMETER AND CATCHMENT CHARACTERISTIC	INITIAL VALUE		UNIT
	IMPERVIOUS AREA	PERVIOUS AREA	
2D grid size	5 x 5		m
2D time step	2.5~2.6		s
Wet/dry cell depth	0.002		m
Sub-catchment slope	0.001		
Impervious percentage of lots	75		%
Sub-catchment width	square route of sub-catchment area		m
Surface roughness	0.014	0.05	
Initial infiltration rate	0	22.5	mm
Continuous infiltration rate	0	2.5	mm/hr
Zero detention	100	25	%
Depression storage	0	2	mm

4.2.1. Sensitivity of parameters against minor and major rainfall events

The results of sensitivity of surface roughness of the impervious area (75 per cent of each sub-catchment in this case) to peak outflow for the 1 year and 100 year ARI events are given in Figure 5. These results show that the roughness values of the impervious areas have a great impact on the peak flow rates. The usual Manning's value of 0.014 (or 0.015) for road, roof and concrete surfaces (Chow, 1959) will be adapted to the pre-development bare land roughness value of 0.035-0.050 with the land use changes and it will increase the peak flow from 5.4-10.3 per cent in the 100 year ARI event and from 10.3-16.5 per cent in the 1 year ARI event (considering bare land roughness coefficients of 0.035-0.050). This shows the sensitivity of the peak flow rate to surface roughness will be higher when the ARI event is lower. However, this value again will increase when the percentage of imperviousness is increased.

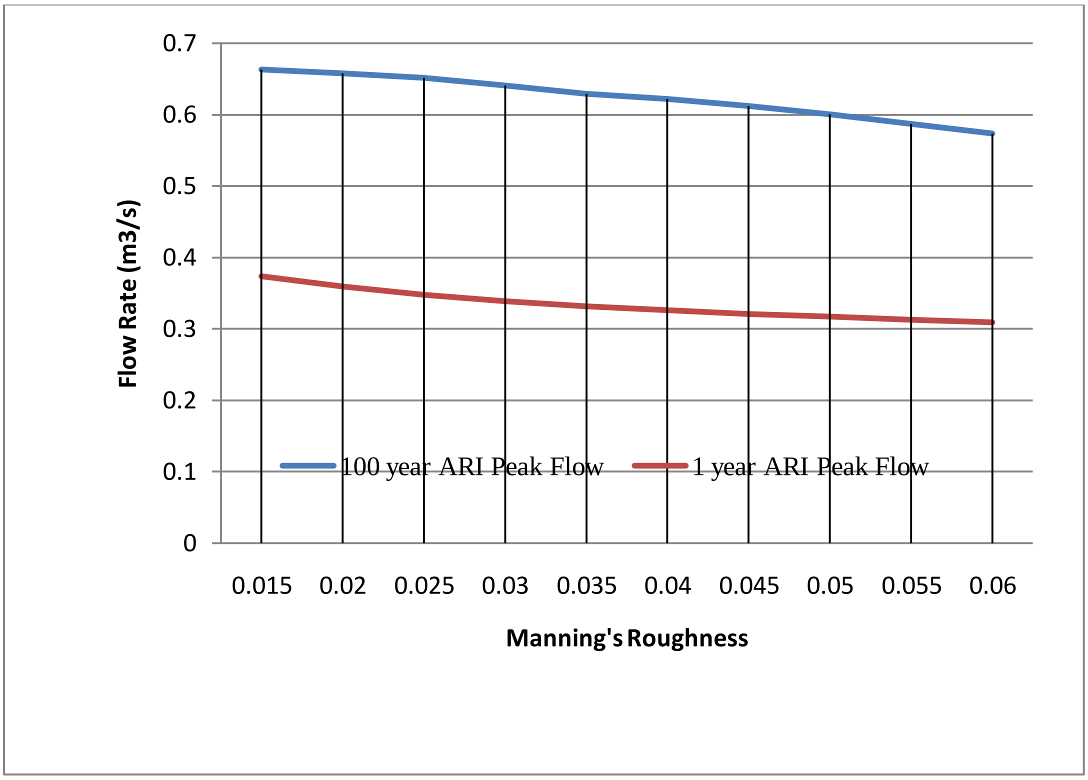


Figure 5. Change of peak flow rate with different surface roughness values

The results of the sensitivity of the combined effect of initial and continual infiltration losses of the pervious area to peak flow rate for the 1 year and 100 year ARI events are given in Figure 6. The infiltration rates were selected from a gradually reducing pattern for both initial and continual losses. The results show that there are 8.8 per cent and 0.3 per cent variations of 1 year ARI and 100 year ARI event peak flows from the fully pervious conditions to fully impervious conditions. The sensitivity of the infiltration values to peak flows is based on a 25 per cent value for the pervious portion of the land use. Therefore their effect is negligible in major rainfall events. However, the sensitivity of infiltration is considerable when it comes to minor rainfall events such as a 1 year ARI event. When the percentage of impervious land use is increasing, the sensitivity of infiltration towards the results will decrease further.

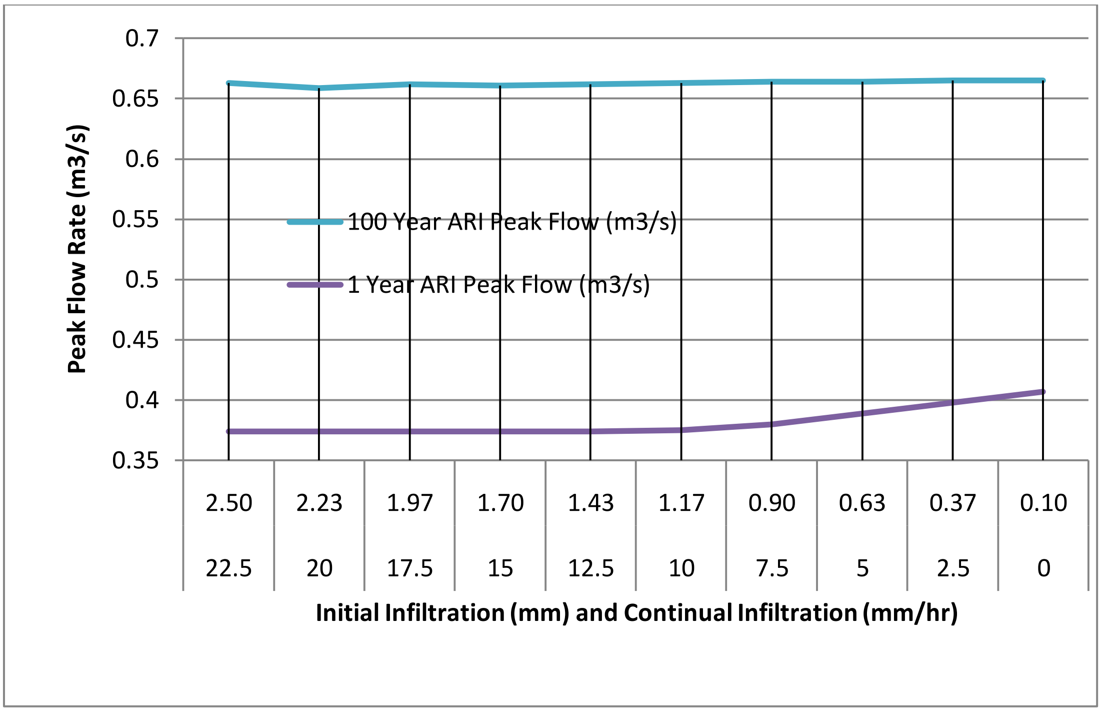


Figure 6. Change of peak flow rate with different infiltration values

The depression storage values for the pervious areas were changed from 0 mm to 4 mm and the sensitivity of depression storage towards the peak flow was less than 0.01 per cent for both rainfall scenarios. Again the effect of this catchment characteristic is dependent on the percentage of pervious area. Its effect on peak flow rate is negligible. The zero detention percentage for the impervious area was changed from 0 to 100 per cent, but the peak flow rate was changed only from 0.002 m^3/s in the 100 year scenario, whilst the peak flow variation remained constant for the 1 year event. Therefore sensitivity of percentage of zero detention of the impervious area towards the peak outflow is negligible for both minor and major rainfall scenarios. However this characteristic is again based on the percentage of impervious land use.

4.3. Calibration and verification

The mean squared error (MSE) and the related normalization, the Nash–Sutcliffe efficiency (NSE), are the two criteria most widely used for calibration and evaluation of hydrological models with observed data (Gozzard 1983). Equation (23) and (24) show MSE and NSE.

$$MSE = \frac{1}{n}\sum_{t=1}^{n}\left(x_{s,t} - x_{o,t}\right)^2 \tag{23}$$

$$NSE = 1 - \frac{\sum_{t=1}^{n}\left(x_{s,t} - x_{o,t}\right)^2}{\sum_{t=1}^{n}\left(x_{o,t} - \mu_o\right)^2} = 1 - \frac{MSE}{\sigma_o^2} \tag{24}$$

Where, n is the total number of time-steps, $x_{s,t}$ is the simulated value at time-step t, $x_{o,t}$ is the observed value at time-step t, and μ_o and σ_o are the mean and standard deviation of the observed values. In optimization, MSE is subject to minimization and NSE is subject to maximization (Gozzard 1983).

In this study, the models' hydrological performance based on multiple variables have been analysed by using NSE. The NSE is one of many ways to quantify the difference between values implied by an estimator and the true values of the quantity being estimated. The MSE measures the average of the squares of the errors. The error is the amount by which the value implied by the estimator differs from the quantity to be estimated. Instead, the NSE uses the MSE and also the observed mean as baseline. This coefficient of efficiency ranges from minus infinity to 1.0, with high values indicating better agreement.

4.3.1. Validation of modelling approaches

Calibration of the both hydrological and hydraulic approaches was carried out by using the observational data for the water depth of the Avenues basin for a 3-day rainfall event on 14 to 17 June 2010. The outflow backwater condition was one of the major parameters affecting the outflow from the catchment, and hence the water depth of the basin. The outfall backwater depth was taken from the observational data, and the length of the outflow pipe was considered to be the length of the main drainage line, while neglecting the inputs to it. This has reduced the effect of changes to tail-water conditions to the model calibration parameters. The groundwater impact was neglected during the initial model calibration process. The water depth of the Avenues basin was used as the calibration variable. The calibrated models' results for both approaches are shown in Figure 7. The figure shows the hydrological approach is closer to the observational data, while the hydraulic approach's results are just above the observational data all the time.

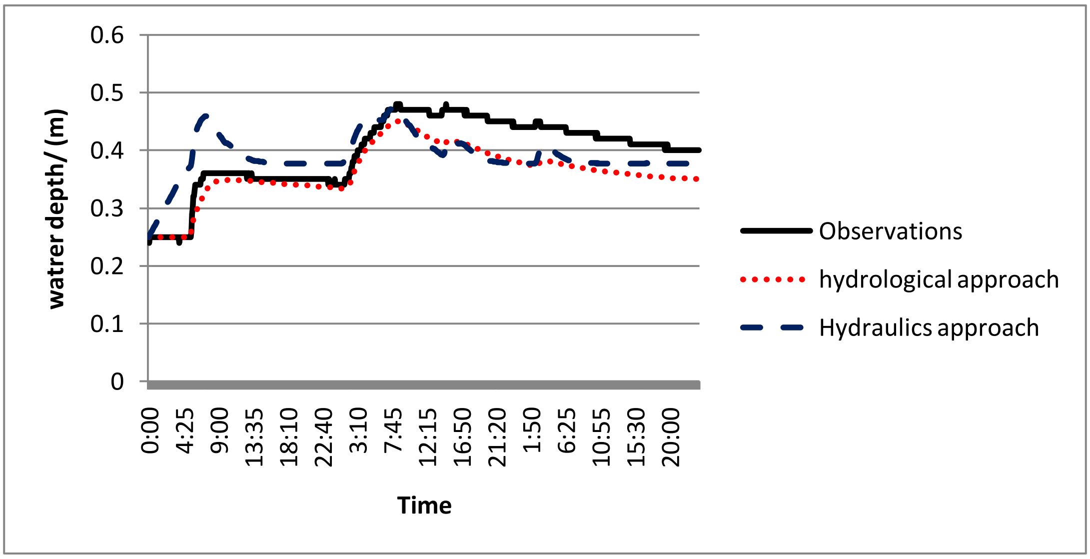


Figure 7. Calibration of hydrological and hydraulic models (using the rainfall event 14 to 17 June 2010)

The comparison was further analysed in Figure 8 by using the NSE for the model results and observational data. The NSE for the hydrological approach was 0.855 and for the hydraulic approach 0.513. As the NSE is maximized, the higher value closer to 1 gives better results. Therefore it confirms that the hydrological approach is most accurate and the hydraulic approach also still can be used, when hydrological approach cannot be used alone (the modelling approach selection is dependent on available data and their behaviour and the expected behaviour of the results).

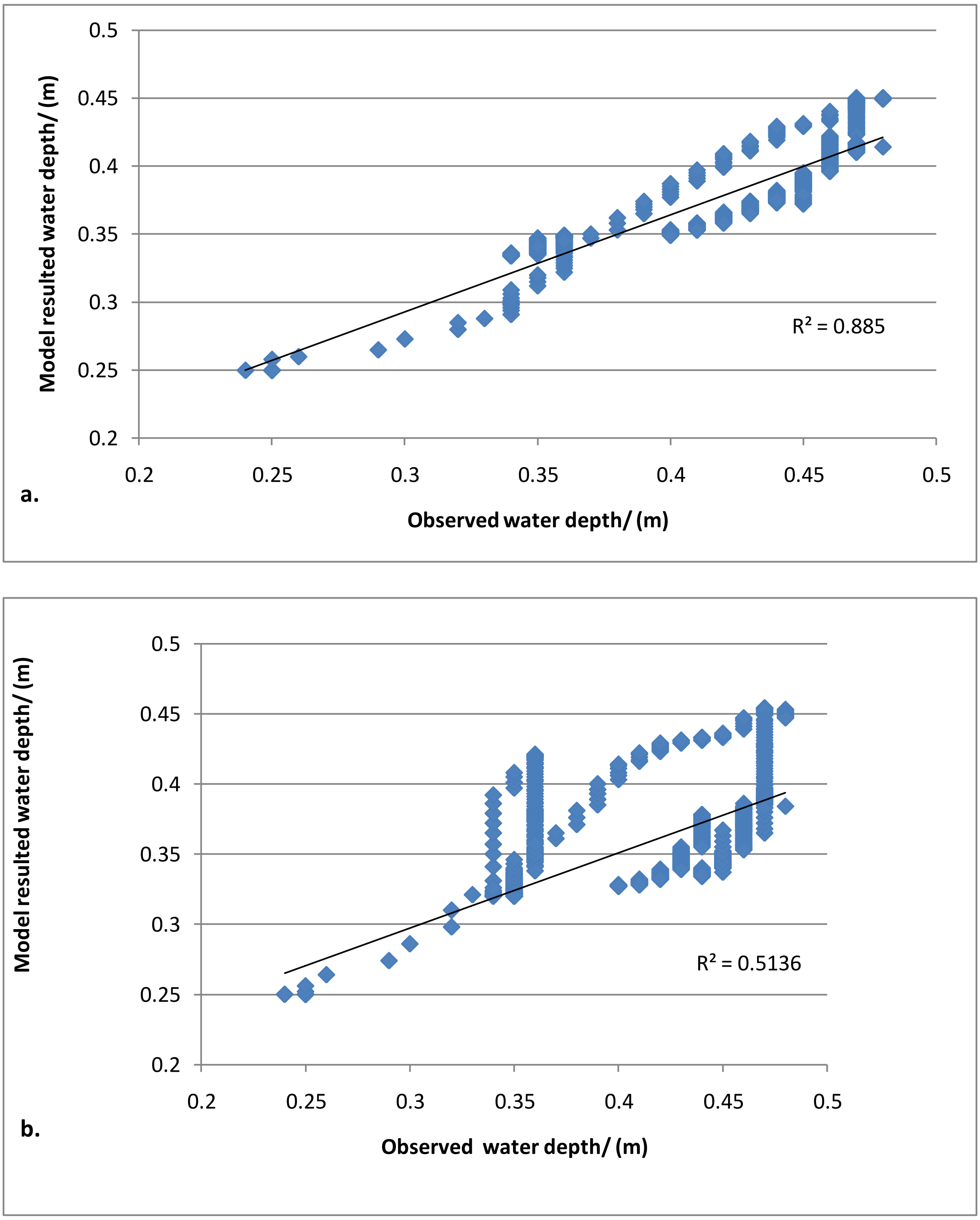


Figure 8. Model results against observation data: a. Hydrological approach and b. Hydraulic approach

Verification was done for another independent rainfall on 9 to 12 July 2010 and the results are shown in Figure9. It shows that the both approaches' results are just above the verification observational data initially and go below it at the end. This can be due to the modelled water levels of Avenues basin for both models having a similar effect from fixed tail-water conditions, which varies in the actual case. However the validation process of the two models shows that both modelling approaches are suitable to analyse urban catchments with similar characteristics.

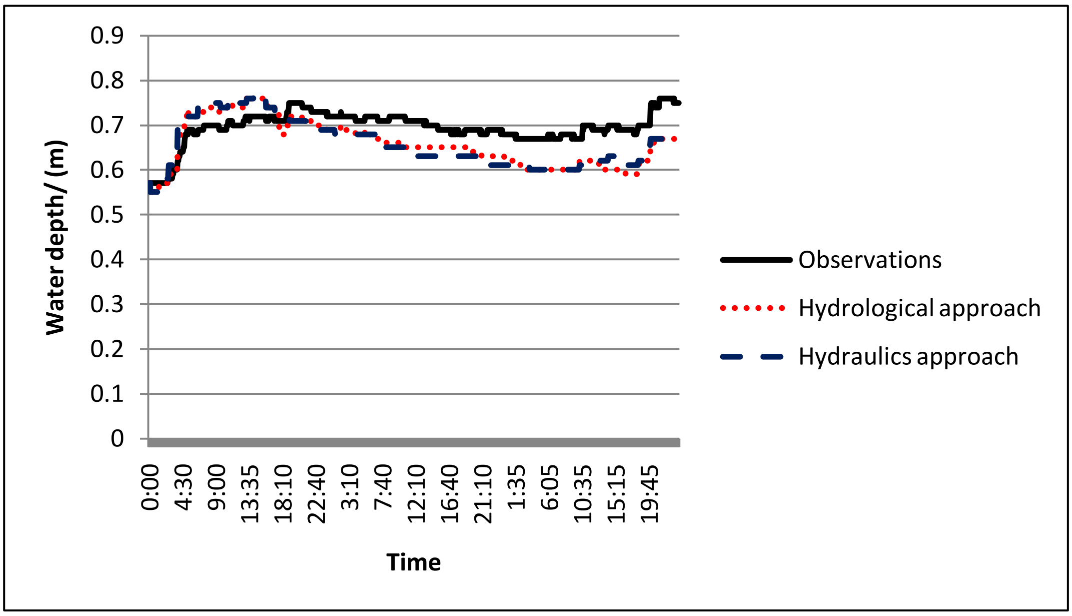


Figure 9. Verification of hydrological and hydraulic models. (Using the rainfall event 9 to 12 July 2010

4.3.2. **Validation of modelling of groundwater effect**

Shallow groundwater was one of the major reasons for the inadequacy of USWMs in Canning Vale. During field visits, it was observed that groundwater base-flow flows through the underground drainage, submerging and preventing them from conveying surface runoff downstream. Even when the above modelling approaches neglected the groundwater effect to simplify the selection and validation of modelling processes, it should have been given priority thereafter. Another calibration and verification process was carried out to find the groundwater impact and land use change effect on the catchment hydrology after validation of the modelling approaches. The same Avenues catchment was modelled by considering the land use changes, presence of shallow groundwater, and urban infrastructure on urban hydrology. The modelling of the urban drainage system and urban flood plain by using a combination of 1D (piped drainage) and 2D (overland flow) elements was used. The hydrological method, which was proven as the best method to represent urban catchment characteristics, was used. Instead of the Laurenson method, the SWMM nonlinear runoff routing method was used, since it facilitates the activation of simultaneous groundwater mounding analysis.

Calibration was carried out by using the observational data for the water outflow of the Avenues basin by using the same 3-day rainfall event on 14 to 17 June 2010. Calibration was done by changing the Manning's roughness values and infiltration rates for the different land uses. They were more influential on the results according to the sensitivity analysis. The zero detention percentage and the depressions storage were neglected, as the results were less sensitive to them according to the sensitivity analysis. The model's time steps and grid size were selected to be the same as the values derived from the sensitivity analysis. They were further tweaked as much as possible to minimize the iteration errors. The Manning's roughness values and infiltration rates for different land uses, finalized during the calibration process, are given in Table 3.

Table 3. The finalized Manning's roughness values and infiltration rates

LAND-USE TYPE	**MANNING'S ROUGHNESS VALUE**	**INFILTRATION RATES**	
		INITIAL (mm)	**CONTINUOUS (mm/h)**
Public open spaces and gardens	0.05	15	2
Roof	0.014	1	0.1
Ponds and swales	0.025	-	-
Roads	0.014	1	0.1
Car parks and other paved areas	0.025	1	0.1

The groundwater coefficient stated in equation (18) was varied during the calibration process and treated as one of the major calibration parameters. The outflow from the Avenues basin, instead of water depth, was considered as the data set that must be matched with the observational data. The calibrated models' results of two scenarios: the model run with the groundwater effect and the model run without the groundwater effect, and the observational data, are shown in Figure 10. Modelling results show that the groundwater impact on the catchment hydrology in the shallow groundwater catchment of Canning Vale, Avenues catchment is significant. The curve with groundwater effect is closer to the curve of observational data, while the curve of the results of flow without groundwater varies significantly from the observational data. The fixed tail-water condition given in the model causes the lower values at the end of this hydrograph. The consideration of the groundwater effect has stabilized this to some level. Also, the continuous groundwater base-flow (the same as observed during field visits), helps to keep the flow hydrograph above the flow rate of 0.01 m3/s, helping the results to match with the observational data. At the end of the rainfall event, there is a considerable variation between the simulated results and observations. Fixed tail-water conditions that differ from the actual varying tail-water conditions affects the results.

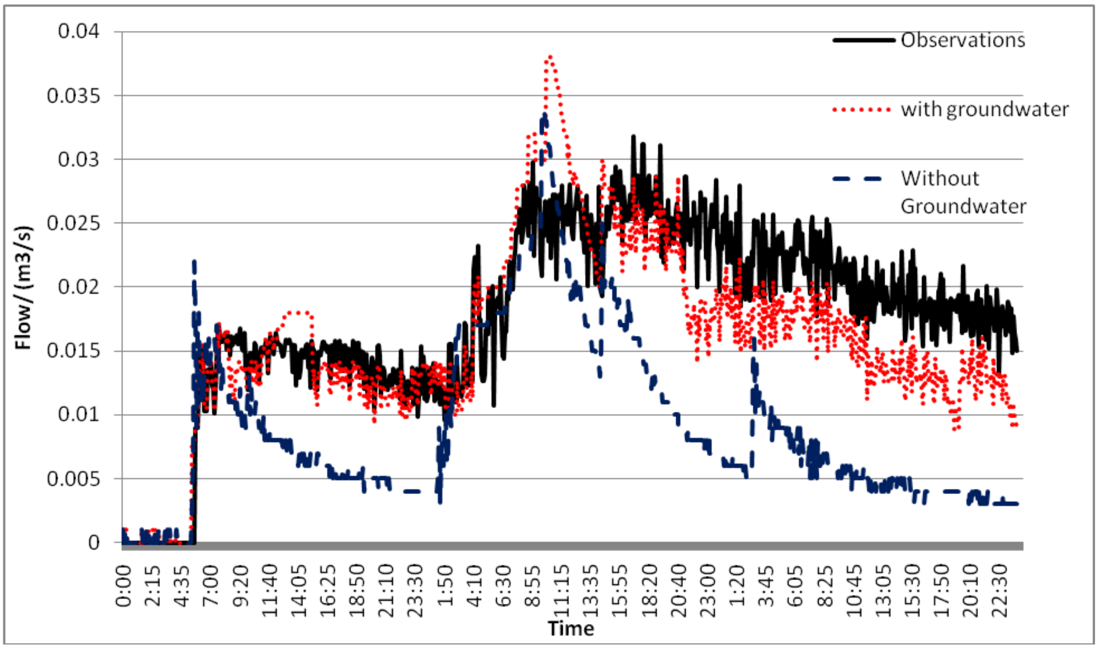


Figure 10. Calibration of Avenues outflow (using the rainfall event 14 to 17 June 2010)

The comparison was further analysed by using the NSE equation in the Figure 11. It confirms that the model with the groundwater effect, having an NSE value of 0.7347, can be used to represent the hydrology of the urban catchment.

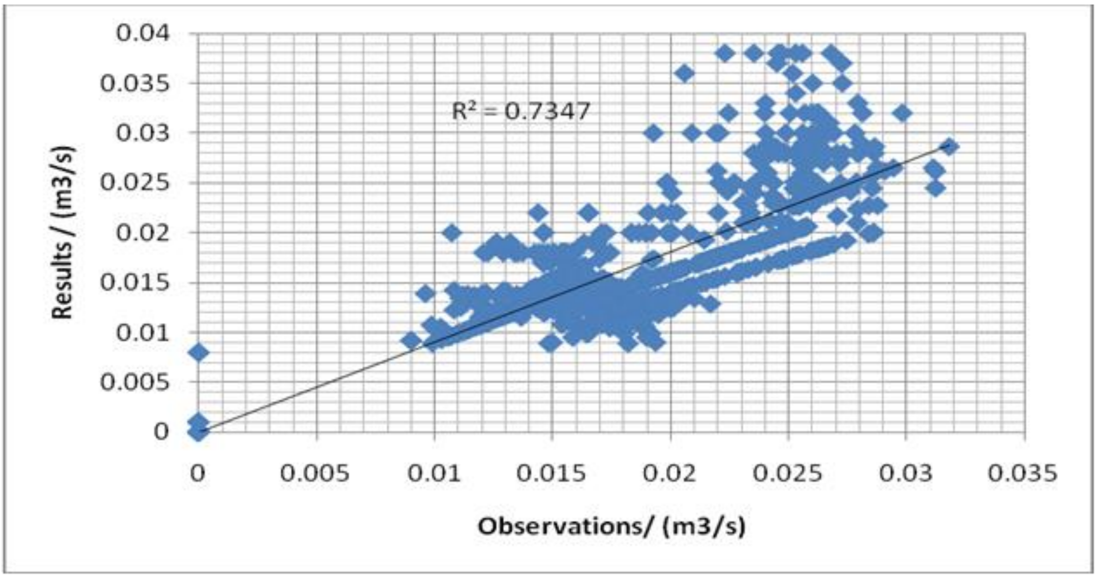


Figure 11. Model results against observational data

Verification was done for the model considering the groundwater effect by using an independent (i.e. independent from the rainfall used during the calibration, but the same rainfall used in the above verification process) rainfall event during the period of 9 to 12 July 2010. The resulting hydrograph of outflow from the Avenues basin against the observed outflow is given in Figure 12. The results show that again the groundwater effect is affecting the model results and that the model considering the groundwater mounding is more suitable for the urban catchment representation.

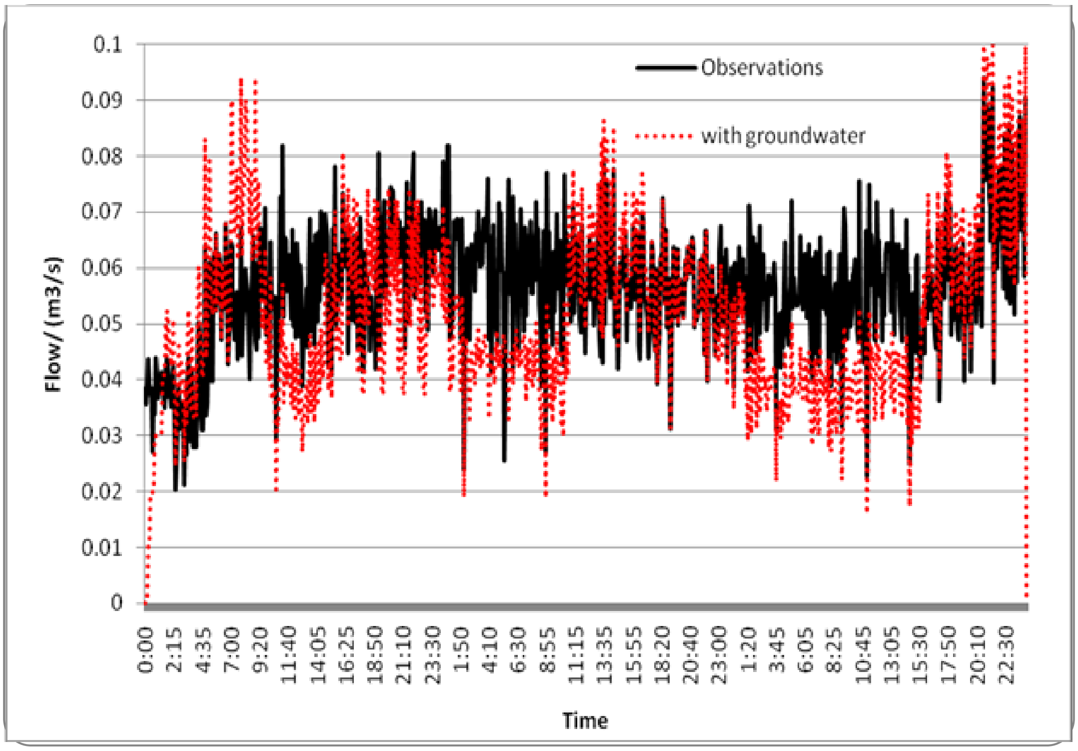


Figure 12. Verification of hydrological and hydraulic models (using the rainfall event 9 to 12 July 2010)

The model, after calibration, was used to generate the flood inundation of the area under a major ARI rainfall event. Checking the suitability of the verified model to generate and illustrate the flood vulnerability of the area was the aim. The results of mapping the flood vulnerability of the Avenues catchment for the 100 year ARI event are shown in the Figure 13. Maximum flood height for the catchment is around 1 m at the Avenues basin. This shows the whole basin and nearby public open space will be inundated under a 100 year rainfall event. This public open space has been designed to manage a 100 year event. Therefore this inundation level under a critical event is considered acceptable. The past evidence recorded in the City of Gosnells also proved that this is a flood prone area for major rainfall events. Other than that, no other critical flood vulnerable area was found. The inundation of the road network, as shown in the figure, is acceptable and it shows the model's capability for analysing urban flood inundation. The water depths are higher than the previous results generated without the groundwater consideration (Figure 3 and Figure 4), confirming that the analysis of groundwater contribution during the analysis of the shallow groundwater lodged urban Canning Vale catchment's hydrology and USWMS is significant.

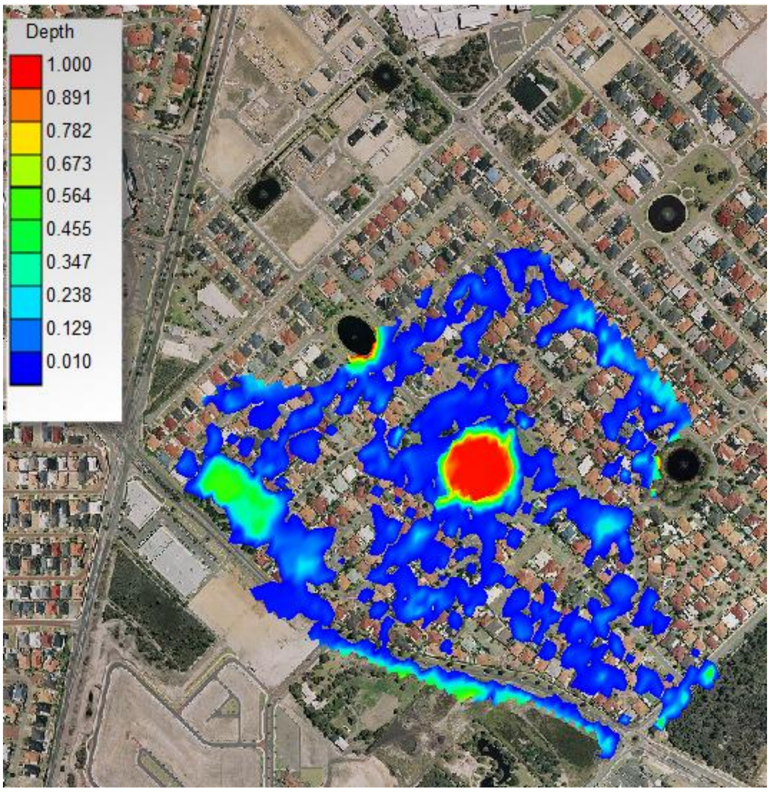


Figure 13. Flood inundation mapping for 1 in 100 year flood event

4.4. Conclusions

This chapter discusses the model's behavioural sensitivity analysis, catchment characteristics and the modelling parameter analysis, as well as model validation through calibration and verification. The whole sensitivity analysis and validation process was narrated step-by-step, reducing the variables affecting the results by deciding fixed or limited ranges of values for them.

A sensitivity analysis for selected catchment characteristics that depended on land use categories was done. The impact of land use change, even considering only surface roughness change, is considerable in its influence on the downstream peak flows. Results show that there can be and increment of 5.4-10.3 per cent in 100 year ARI event and 10.3-16.5 per cent in 1 year ARI event peak flow between the pre and post-development land use change. These show that the sensitivities of the surface roughness and the percentage of impervious land use are considerable, especially in the minor rainfall events. The sensitivity of infiltration loss values to the peak flows in terms of percentage is 8.8 per cent and 0.3 per cent respectively for the 1 year and 100 year ARI events. Therefore the sensitivity of infiltration losses can be neglected in major rainfall events in an urban catchment, considering the lower percentage of pervious land use portion, but it still needs to be accounted for in a minor rainfall event. During the modelling of similar urban catchments, percentage of impervious and pervious areas, surface roughness and the infiltration losses should be modelled with due care. Sensitivities of the depression storage depth and the percentage of zero detention to the peak flow are negligible according to the results for both 1 year and 100 year rainfall events. Therefore these characteristics can be neglected during urban catchment modelling.

The study used two approaches, the hydrological approach and the hydraulic approach, to simulate the flood inundation of an urban catchment. A comparison was made between both approaches for their capacity to represent an urban catchment most accurately. It was identified that both approaches are capable of representing the complex urban hydrological catchment, together with the 1D drainage network, but the coarseness of the topographical data might reduce the accuracy of the hydraulic approach. The results show that the hydrological approach is more accurate with the observational data having the NSE value of 0.855, whereas the hydraulic approach has an NSE of 0.5136 in the calibration process. Considering the flood inundation representation, both approaches show similar results for the inundated areas and flood depths.

An analysis of the effect of a shallow groundwater table on urban hydrology and USWMSs was carried out by using the Avenues catchment in Canning Vale. A hydrological approach with hydrological surface routing, 2D hydraulic excess water surface routing and 1D hydraulic drainage flow routing, which was proven as the best method capable of representing an urban catchment, was used. The effect of a shallow groundwater table on the catchment has been evaluated by comparing two scenarios: routing the surface water together with groundwater and routing surface water neglecting the effect of groundwater. The comparison shows that there can be a considerable effect from groundwater, when modelling a shallow groundwater urban catchment. The NSE value for the groundwater and surface water coupled model's result flow hydrograph against the observational hydrograph is 0.7347. This shows that the model accounting for the groundwater effect can be used as a tool to assess a shallow water urban catchment. The flood inundation map for the catchment, made by using best refined model and the selected catchment characteristics, was processed considering the 100 year ARI event. The results of inundation water depths which can be considered as reasonable compare to the recorded past experiences of flood inundation in the city during major rainfall events. This again shows that the model’s use of accounting for the groundwater effect is reliable. Also it can be concluded that more accurate topography data has improved the results. With the support of an adequate level of topography data, 2D surface runoff by using spatial data is a reliable hydrological approach to model urban catchments.

CHAPTER 5

5. CASE STUDY OF CANNING VALE CENTRAL CATCHMENT DRAINAGE ASSESSMENT

5.1. Introduction

This case study aims to assess the urban stormwater management system of Canning Vale Central catchment in the City of Gosnells. Urban areas, where much of the land surface is covered by impervious materials, are characterized by reduced infiltration and accelerated runoff, which has potential to result in localised flooding. Therefore traditionally the requirement for urban stormwater management of such areas was capturing runoff collected in the catchment and transporting it as quickly as possible downstream to avoid flooding. Therefore, assessments of stormwater characteristics represent a large investment for many urban communities, especially flood prone areas. The City of Gosnells was one of the first local authorities in the late nineties that embarked on a new approach, called water sensitive urban design (WSUD). WSUD strongly recommends land developments that incorporate infiltrating urban runoff as close to source as possible and high up in the catchment, to reduce the need for construction of major hard drainage infrastructure.

Canning Vale Central catchment has been developed rapidly throughout recent history, and currently most of the area is covered with urban land developments. Moreover, the study estimated that the current average land lots are about 75 per cent impervious. Canning Vale Central catchment has several storage basins constructed and some proposed basins currently act as natural flood storage areas. There are a number of public open spaces (POSs) within the Central catchment to facilitate runoff from major rainfall events. A multiple user corridor (MUC) at the catchment carries water from upper sub-catchments to downstream. There are open channels that convey stormwater from upstream basins to the MUC. The MUC has been designed to facilitate larger average recurrent interval (ARI) rainfall events above 1 in 10 years, but recent records cited that it was being inundated during minor rainfall events. The capacities of these drainage systems are currently not sufficient under the urbanization process, together with sub-division activities, which increase the percentage of impervious areas. The City of Gosnells has observed recent flooding in some of the areas within the catchment. Notably, flooding has occurred after even small storm events and water has been retained for a long time without infiltration. Observation has proved that the shallow groundwater level, which prevents infiltration, has increased the overland flow and tended to flood low areas. It was cited that most of the manholes near the storage basins, even in the upstream sub-catchments, converged with the groundwater table during minor rainfall events. Therefore the current situation urged a hydrological assessment of the catchment and its urban stormwater management system (USWMS).

In recent history a number of studies had been conducted by the City of Gosnells to assess rapidly urbanizing catchment behaviour and available drainage sufficiency in the Central Catchment. A total water management strategy for Canning Vale Central catchment had been prepared to address the stormwater issues (Jim Davis & Associates 1999). Subsequently a conceptual drainage design for the region was implemented to bring active stormwater and nutrient management within the scope of the total water management strategy (Wagner 2009). The review of the Central Catchment's drainage system has been conducted based on an observation of the lack of anticipated functioning of the drainage infrastructure (Wagner 2009).

This case study, assessing the hydrology of the catchment and its USWMS, was carried out as a collaboration between Curtin University and the City of Gosnells. The study has been performed using a numerical model based hydrological analysis of the Central catchment including the determination of flow characteristics, capacity of the stormwater drainage system and the hydrological behaviour of the stormwater catchment(s), based on available data. XPSWMM was used as the numerical model. The data collection, which was used to calibrate the model, was facilitated via real-time telemetric monitoring stations. The shallow groundwater table, which submerged the underground drainage, was treated with special care. Two scenarios have been considered during the modelling work:

- Drainage assessment considering the impact of shallow groundwater.
- Drainage assessment without considering the impact of shallow groundwater.

Other than that, one scenario using some recently proposed future developments within the catchment was assessed to find out the impact from them on the current

USWMS. Finally, several standard rainfall scenarios were used to generate results and to portray flood inundation maps.

The study facilitated a better understanding of the catchment characteristics, hydraulic and hydrodynamic behaviour of the drainage system and the performance of constructed drainage infrastructure. The results of the study led to develop recommendations that could address the existing flooding problems and would assist in reducing overall perceived risk of flood occurrences. Flood inundation maps have been portrayed for the worst-case scenario, which accounted for the shallow groundwater table's effect under standard rainfall events. The flood inundation maps would guide future land development and stormwater management by identifying the flood risk areas of the catchment.

5.2. Objectives and methodology

The main objective of this research was to assess the urban drainage system in Canning Vale Central catchment, including the impact of groundwater and urban land developments on the functionality of the drainage infrastructure. Therefore, the major objectives of the case study can be stated as being:

- To develop a numerical model of the catchment and drainage network, taking local catchment properties into account.
- To calibrate the model using the collected data and ongoing monitoring data (short-term data).
- To conduct a detailed hydrological assessment to evaluate the performance of the overall drainage network taking the groundwater effect into consideration.
- To compare the outcomes of hydraulic modelling with catchment topography and to develop flood risk/vulnerability maps.
- To assess the impact of proposed future development scenarios on USWMS.
- To provide recommendations and the required modification to the existing drainage system to reduce the risk of and /or avoid urban flooding.

To sustain the above objectives during the assessment, the following methodology was implemented:

- Literature review of past studies and data collection (including past study results, drainage details, hydraulic structure details, topographical, geographical and land use data, groundwater monitoring data and rainfall data).
- Conducting a series of field visits to assess the catchment features, flow observations and to verify drainage structure in digitized maps.
- Data analysis, including the monitoring and processing of telemetric data to use in model calibration.
- Processing of input data including identification of sub-catchments, processing of drainage data, assessment of land use types and other catchment characteristics and DTM building using topographical LiDAR data.
- Modelling of sub-catchments, their characteristics and the current USWMS.
- Performing a sensitivity analysis to identify the best modelling techniques, sensitivity of land use characteristics and the groundwater effect.
- Model validation by calibration and verification, by using observational data.
- Performing a series of model runs for standard ARI rainfall events.
- Assessment of catchment hydrology and performance of USWMS (by generating outflow hydrographs, storage water depths. etc).
- Portraying flood inundation/vulnerability maps under each rainfall scenario (under worst-case scenario accounting for the groundwater effect).
- Performing a post-development model run, to assess the impact of proposed future developments on the current USWMS.
- Giving recommendations to mitigate flood issues, increase the performance of current USWMS and to minimize the effect of future land developments on the USWMS.

5.2.1. **Available data and data collection**

- Historical rainfall data used to generate ARI events hydrographs (BoM 2012).

- The City of Gosnells provided as-constructed design drawings for the designs of major basins, some of other associated hydraulic structures and the pipe network.
- The City of Gosnells provided digitized drainage network (pipes and manholes).
- Cadastral map and the Central catchment boundary from previous studies were available in GIS and DWG formats.
- Topography of the area was derived initially from the 1 m interval contour maps and finally by the 0.2 m interval contour maps supplied by the City of Gosnells. These GIS contour maps have been used to build the Digital Terrain Model (DTM).
- Aerial photos available for the catchment have been used to model the spatial features in the model, i.e. the land use changes.
- Groundwater contours were obtained from the Department of Water's *Perth Groundwater Atlas* (DoW 2004).
- Water depths of some of the basins, groundwater levels and some other observations were obtained from telemetric data monitoring devices. There are 13 data monitoring locations identified within the Central Catchment and nearby Eastern Catchment and these are given in Figure 16 (Metermate 2011).

In addition to input and observational data, information on drainage features has been collected by several field visits. Flows at some locations in the stormwater pipe network were monitored by using a Starflow Ultrasonic Doppler Instrument (Unidata, 2007) with rainfall occurrences.

5.3. Catchment description

The Central Catchment of Canning Vale, comprising of area of approximately 333 ha, is bounded by Nicholson Road to the north, Ranford Road to the west, Campbell Road to the south and Gateway Boulevard to the east, and is shown in Figure 14. Figure 14 also gives the stormwater basins, MUCs and special drainage features of the catchment. Two major sub-catchments were identified within the Central catchment; Avenues sub-catchment and Main Drain sub-catchment. Two other upstream sub-catchments contribute excess runoff to the Central catchment and have been taken in to account for their stormwater contribution to the Central catchment. It is suggested that the runoff from these sub-catchments, especially in the rainfall events that exceed 10 year ARI events, can impact on the flooding along the Central catchment's downstream MUC. The Glenariff sub-catchment, south west of the Central catchment, has been modelled separately and feedback from its hydrographs has been taken into account in the Central catchment modelling process. Sub-catchments considered under this study and their areas are shown in Table 4.

Table 4. Sub-catchments, fully or partially related to the Central catchment

CATCHMENT NAME	AREA (ha)
Avenues Sub-catchment	33.4
Main Drain Sub-catchment	172.1
Sanctuary Lake Sub-catchment	64.1
Glenariff Sub-catchment	63.2
Total	332.8

Topography

According to the *Perth Groundwater Atlas* (DoW 2004) the Central catchment has flat grades throughout the area, and topography contours change gradually from 25.00 m AHD at its western boundary to 20.00 m AHD at the eastern boundary. The approximate slope is selected as 1:2000 from the previous studies and from current topography maps. Subsequently, the area was been developed with urban infrastructure and the original contours were modified accordingly, but the recent land development may not be represented in the topographical maps that were used. The 0.2 m interval topography data has been used to create the Digital Terrain Model (DTM).

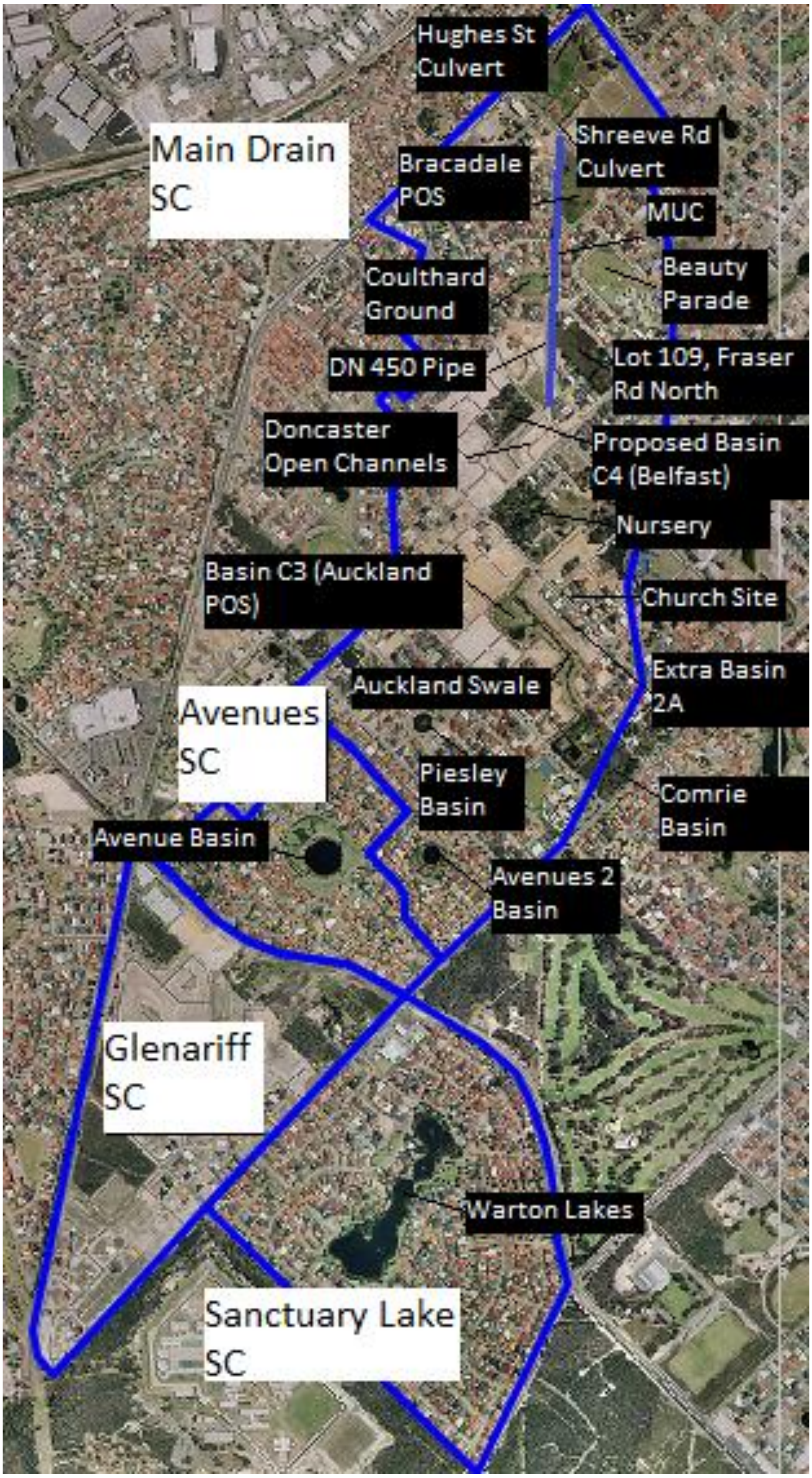


Figure 14. Canning Vale Central catchment and associated sub-catchments and special features (Note: SC = Sub-catchment).

Climate

The area experiences a dry Mediterranean climate of hot dry summers and cool wet winters. Long-term climatic averages indicate that the Central Catchment is located in an area of moderate to high rainfall, receiving 831.8 mm on average annually (BoM 2012) with the majority of rainfall received between May and August. The region experiences rainfall for 84 days annually (on average), however high evaporation rates and temperatures throughout the summer months drastically reduce flow within the MUC and main drain. But the possibility of area being subjected to the frequent storms, which can be categorized under 1 in 1 year ARI to 1 in 100 year ARI, is likely to be influenced by climate change effects.

Groundwater

The groundwater table of the area can be identified as shallow and near to the surface, which makes submerged stormwater drainage a possible condition. The level of the average annual maximum groundwater changes from 24.00 m AHD at the catchment's western boundary to 20.00 m AHD at its eastern boundary, giving it a roughly 2.5 m to 0.5 m depth from the surface (DoW 2004). It was found that the groundwater was visible at the surface level near the Avenue Basin and along the swales and multiple user corridors near the south west border of the catchment. Upstream groundwater is flowing continuously through the main drain to downstream areas until around November, even after the rainfall has stopped in August, according to the observational data. The areas around the major basins and MUC in the middle of the catchment are water-logged during most of the rainy season due to this high level of the groundwater table, and cause a cut-off of the infiltration. The previous studies have recommended the installation of subsoil drainage to lower the local groundwater table and to maintain pre-development groundwater levels during future developments. The groundwater contours for the area were obtained from the *Perth Groundwater Atlas* (DoW 2004) and are shown in Figure 15. Apart from that, to assess the groundwater levels, observational data from real time telemetric monitoring stations maintained by the City of Gosnells have been used.

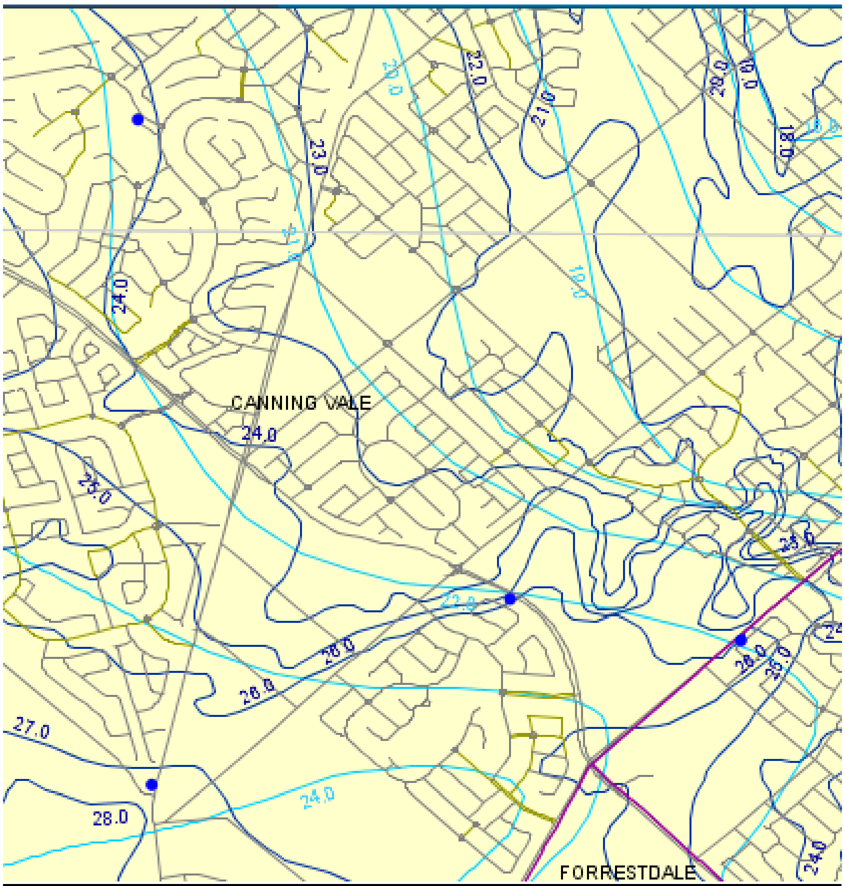


Figure 15. Groundwater contours for the central catchment area (DoW 2004)

Note: Dark blue contours – Average Annual Maximum Groundwater Levels (AAMGL)

Light blue contours– Groundwater Levels - March 2003

Water from Upstream Catchments

It has been noted in previous studies that two major pipelines carry the water from upstream catchments to Canning Vale drainage under the maximum allowable flow rate of 1.13 L/s/ha. Previous studies had not calculated the inputs from these sub-catchments to their models, assuming that only the events above a 10 year ARI would be contributing water from these catchments into Avenues. Runoff from both these catchments has been considered in this study and the upstream catchments were analysed for their contribution towards the increase of the Avenues basin water level during major rainfall events. The catchment located south west of the Central catchment has been introduced as Glenariff catchment and the sub-catchment south of the Central catchment has been introduced as Sanctuary Lake catchment.

Input Data

The City of Gosnells provided the as-constructed design drawings for the major basins, as well as some of other associated hydraulic structures and the pipe network. The City of Gosnells has digitized the majority of their as-constructed drawings into GIS based maps. As-constructed drawings have been referred to when developing all the hydrological models and they have been used to analyse the digitized drainage network (pipes and manholes). The Cadastral map and the Central catchment boundary from previous studies are available in GIS and DWG formats. Topography of the area has been derived initially from the 1 m interval contour maps and finally by the 0.2 m interval contour maps supplied by the City of Gosnells. These GIS contour maps have been used to build the DTM. The aerial photo available for the catchment has been used to model the spatial features in the model, i.e. the land use changes. Groundwater contours have been obtained from the Department of Water's *Perth Groundwater Atlas* (DoW 2004). In addition to that, water depths of some of the basins, groundwater levels and some other observations were taken from telemetric data monitoring devices.

Data for calibration and verification purposes were obtained from the available ongoing data from real-time telemetric monitoring stations, which have been managed by the City of Gosnells. There were 13 data monitoring locations identified within the Central Catchment and nearby Eastern Catchment and these are given in the Figure 16 (Metermate 2011). Later, one more location to monitor the Sanctuary Lakes outflow to Avenues has been installed, but this station has not recorded any data yet. Rainfall data, basin water levels, groundwater levels and flow data were obtained from these observational data and used as inputs to the model and also to calibrate the model.

No	Station Name	Location	Measurement
COG1	Admiralty Basin (Overflow)	Admiralty New Outlet Pipe	Q + WL
COG2	Avenue Basin 2	Baychester Lake	Q + WL
COG3	Central Catchment	Shreeve Rd Outlet pipe to open channel	Q + WL
COG4	Avenue Basin 1	Avenue Basin Outlet	Q + WL
COG5	Admiralty Basin (Main out)	Admiralty Old Outlet pipe	Q + WL
COG6	Comrie Basin Water Level	Comrie Basin Headwall	WL
COG7	Rain Gauge	Campbell Rd Primary School	RF
COG8	Piesley GW Level	Piesley Lake	GWL
COG9	Shreeve GW Level	Bracadale Way POS	GWL
COG10	Sandringham GW Level	Sandrigham porm POS	GWL
COG11	Waterperry GW Level	Waterperry POS	GWL
COG12	Belfast GW level	Belfast Cl	GWL
COG13	Central GW Level	Avenue Basin 1	GWL

Figure 16. Telemetric data monitoring locations for Canning Vale

Rainfall Scenarios and Storms

After the calibration, four major storm events were considered to obtain the flood vulnerable maps and required drainage assessment data. These were Average Recurrent Interval (ARI) storm events of 1 in 1 year, 1 in 5 year, 1 in 10 year and 1 in 100 year. The 1 year ARI event has been analysed to assess the adequacy of the existing lot wise stormwater drainage facilities and to propose more measures where required. The 5 year ARI event was analysed to assess the drainage network and to identify the areas where further modification might be required. The 10 year ARI and 100 year ARI events were assessed to find out the critical runoff flow paths and flood vulnerability in the catchment. Analysis of the flood issues along the main drain line was a major objective of the study, where the 100 year ARI event would be critical.

The rainfall intensity data were obtained from the Bureau of Meteorology web-based application (BoM 2012). The rainfall event hydrographs were generated by applying average rainfall data to the unit hydrographs (Pilgrim 1987).

Drainage network

The drainage network for the Central Catchment consists of an underground pipe network, manholes, open channels and other hydraulic features inclusive of off-line and on-line basins and swales. The pipe network consists of circular concrete pipes whose diameter varies from 225 mm to 1050 mm. Some of the drainage pipes and manholes, especially near the basins, are submerged by being below the groundwater table. It has been identified by field observations that they are submerged below the groundwater during the rainy session. The DN450 main drain line conveys groundwater and runoff from upstream, starting from Avenue basin #1 to the downstream ending at observation location COG 3 at Shreeve Road. There are number of culverts crossing the roads that are directly connected to the drainage system as well. Previous studies have identified that DN450 main drain pipe line is mostly occupied by groundwater. In fact it has been designed as a sub-soil drainage line to lower the groundwater table in the upstream part of Central catchment.

Major Basins, Swales and Open Channels

There were altogether ten major basins identified within the Central Catchment and shown in Figure 14. Auckland Rainwater Garden (Basin C3) has been constructed to act off-line with Auckland swale. Design Basin C4, which accommodates the space where Doncaster open channels are located, acts as an off-line flood storage area during major rainfall events. Basin C6 and C7 (ponds) are located in Bracadale POS. The proposed Basin C2 has been identified as the low elevation area which has been referred to as Warrendale Nursery. Auckland swale, two open channels at Doncaster and the open channel between Shreeve Road and Hughes Street are the open channels. The multiple user corridor (MUC) running along the DN450 pipe conveys stormwater downstream in major events. Warton Lake has been identified as the major basin within the upstream sub-catchment of Sanctuary Lake. Four proposed basins have been assumed for Glenariff upstream sub-catchment and those currently act as natural low elevation flood storage areas.

A summary of existing government regulations (guidelines) on the Central catchment stormwater management and the study's observations and assumptions are given in Table 5.

Table 5. Comparison of the initial site observations and assumptions during this study

RECOMMENDATION AND GUIDELINES	OBSERVATIONS AND ASSUMPTIONS
The area of the catchment is calculated as approximately 247.6 ha consist with total of 10 sub catchments integrated with one major basin for each.	The area of the catchment is calculated as approximately 327.2 ha consisting of 2 sub-catchments of Central catchment and two extra upstream catchments (outside of the Central catchment) called Glenariff and Sanctuary Lake.
There are two major storm pipe lines connected in to Central catchment near to the Avenue State carry water from upstream catchment (outside from the Central catchment) in a rate of approximately 1.13 L/s/ha for events larger than 1 in 10 year ARI.	There are two major storm pipe lines connected into Central catchment near the Avenue Estate that carry water from upstream catchments, outside the Central catchment (Sanctuary Lake catchment and Glenariff catchment) at a rate of approximately 1.13 L/s/ha for events larger than 1 in 10 year ARI.
Outflow to the Main drain (DN450 RCP) at Hughes St. is limited to 1.13 L/s/ha as per Water Corporation WA requirements.	However the maximum recorded outflow at Shreeve Road during last two winters (2010 and 2011) was 0.37 m^3/s, which is similar to 1.87 L/s/ha considering 205.5 ha Central catchment (except the upstream outside catchments).
Average Annual Maximum Groundwater Level (AAMGL) has been located 0.5 – 0.75 m below the natural surface.	AAMGL located below the natural surface is about 1 m in Glenariff, 1.0 – 0.5 m in Sanctuary Lake and Avenues and 0.5 – 0.75 m in Main Drain.
The Central catchment is approximately flat graded, with 1:2000 slope.	A 1:2000 gradient was assumed as slope of all the catchments, even where slight variations existed in some areas.
The catchment runoff is sensitive to the lot runoff and groundwater impacts.	Some parts of the drainage network in Central catchment and Sanctuary Lakes were found to be submerged during the winter.

Proposed basins were to be between the summer low water level (LWL) and AAMGL implying that they would be expected to be wet with limited infiltration and the existence of these criteria has been witnessed on site.	Two basins: Avenues basin and Piesley basin had water during the summer, but Comrie basin and all other wetlands, including swales and open channels, were dried during the summer. They all got waterlogged once winter started.
Some of the houses were directly connected to the drainage system.	There were some houses connected to the drainage directly, but in the process this has not been considered.
Major basins were designed to attenuate flow from respective sub-catchment prior to discharge into main MUC swale. Several of the basins have been constructed on-line and attenuate flow from both upstream and the local sub-catchment.	Two basins: Avenues basin and Piesley basin were functioning on-line with the drainage network, but they attenuate the runoff from the sub-catchments, prior to discharge. Comrie basin was functioning as an off-line basin. Also the Bracadale POS and some of other flood storage areas used in major rainfall events were off-line to the system rather than on-line.
Box culvert 1200 x 375 outlet to Hughes Street drain is at RL20.42 where the invert level of the downstream end of the Multiple User Corridor is approximately at RL20. This will cause stormwater backflow to basins C6 and C7.	This culvert is the outlet culvert at Shreeve Road and any back water pressure built up due to this culvert would affect adjacent Bracadale POS (proposed basin C6 and C7) and will increase the water depths of associated small ponds inside the POS. The invert level of the culvert was at 20.42 AHD. The lowest level of the MUC was about 0.3-0.4 m lower than this invert level. However the elevated invert level can be due to the limit of the maximum outflow to Hughes Street Drain (1.13 L/s/ha) and let the runoff to be attenuated to some extent within Bracadale POS before discharging it out of the catchment.
The Piesley Promenade basin on Stidwell Street caters for approximately 15 ha of catchment area originally intended for Basin C1.	This basin has been re-structured recently and promenade has been removed and land space has been leveled to cater for some excess flood volume for major events.

The basin at Birnam Road/ Philadelphia Parade caters for approximately 3.2 ha of catchment area originally intended for Basin C3.	This basin (Extra Basin #2A) has not been constructed yet. The Basin C3 was identified as the Auckland Parade rain garden, off-line with Auckland swale.
Maximum allowable discharge according to the Water Corporation's limitations: • Flows discharging into Hughes Street Main Drain to be limited to 1.13 L/s/ha (approximately 260 L/s) • Overflow discharged from Central Avenue #1 Basin to be limited to 1.13 L/s/ha (approximately 45 L/s) • Overflow discharged from Central Avenue #2 Basin to be limited to 1.13 L/s/ha (approximately 23 L/s)	Observed maximum discharges during two winters of 2010 and 2011; • Maximum flow discharging into Hughes Street Main Drain (only by DN450 pipe outlet) was 1.12 L/s/ha (approximately 370 L/s) • Maximum overflow discharged from Central Avenue #1 Basin was 0.62 L/s/ha (approximately 100 L/s) • Maximum overflow discharged from Central Avenue #2 Basin was 0.00 L/s/ha (0 L/s)

5.4. XPSWMM modelling

The numerical process has been based on XPSWMM. A combination of urban catchment modelling techniques for urban watercourses and flood plains modelled in 2D, urban watercourses and flood plains modelled by using a combination of 1D (watercourses) and 2D (floodplain) elements, urban drainage systems modelled by using a combination of 1D (piped drainage) and 2D (overland flow) elements, was used in this case study. The sub-catchments were used in the hydrology layer to count runoff and the groundwater interaction with the pipe network, when sufficient data could be obtained from the digital maps. The 2D hydraulic layer was used to analyse the runoff from areas where such detailed data of the drainage did not exist..

To represent the urban catchment as close as possible to its actual hydrological behaviour, different modelling techniques were used. The drainage network was modelled as a series of 1D hydraulic elements. The drainage details were fed directly from GIS files. To prevent model complexity that could result in longer running durations and instability of the model, some of the minor components within the drainage network have been neglected (i.e. drainage lines less than 10 m in length). The surface area of a typical manhole was considered as a default value of 1.2 m^2. All the manholes were connected to the 2D grid to prevent any losses of stormwater from flooding. The connection between the hydraulic and hydrology layer has been smoothly built to minimize possible errors. Roads, fences, roundabouts and other features influencing the rate of runoff were spatially represented in the 2D layers, where possible. Spatial representation was done by using a scaled aerial photo of the area. Footpaths and roads act as inland flow paths and convey water, while interconnecting with the drainage network by spill crests at the manholes. The MUC was modelled as a 2D flow path and the model was set to count its capacity and topography according to the DTM. The 0.2 m interval contour data was used to generate the DTM to represent the topography of the terrain. Basins and swales were represented either as 1D flow paths or 2D elements. The water levels of the storage areas and swales in the 1D layer were obtained from the available observational data and given as initial water levels. The cross-sectional data and slopes for swales and basins in 2D were generated by using the DTM. Catchments were represented in the hydrology layer and routed into manholes and basins. However, they were represented spatially in the 2D layer, when there is lack of drainage information.

The 2D engine's iteration time step of 6 seconds was used considering the grid size of 12 m. There were several hydraulic 1D and 2D boundary conditions have been used in the modelling process. The spill crest of the manholes (including storage nodes that act as basins) and inverts of culverts were given boundary conditions to link them into the 2D network to couple both the 1D and 2D hydraulic routing processes. Tail-water boundary conditions were introduced at drainage outlets by using observed average maximum water levels. The 2D head boundary conditions at catchment boundaries were taken according to the topography data. No flow boundaries have been given to open channels since they act as swales, but not as channels, at the beginning of rainfall events. The boundary conditions for 1D flow elements modelled in the 2D layer (i.e. open channels and storage areas) were given their spatial boundaries by an inactive layer.

To analyse the worst-case scenario, groundwater implementation of the catchment runoff was modelled. The starting levels of the groundwater table were selected from telemetric observation data and data from the *Perth Groundwater Atlas* (DoW, 2004). The effect of manual groundwater extraction at deep wells was neglected after analysing the observational data. Roughness and infiltration values for the catchments were assigned either in the hydrology layer or as 2D land use characteristics. The initial and continuous infiltration values for each land use category were given. Average infiltration values before the calibration process were selected after considering the shallow groundwater table of the area, sandy soil fills in the lots, saturated soil conditions of the basins and swales and the percentage of impervious surfaces. These values were further refined during the model calibration.

There were different and complex land use categories throughout the Central catchment, mostly mixed with pervious and impervious spots. There were five major identifiable land use categories defined to reduce the complexity of the model. Values for the surface roughness coefficient (Manning's number) were selected after proper literature review (Chow, 1959 and Pilgrim 1987). Comparison of land use category details cited in the literature with the land use shown in aerial photos was used when categorising the different land use types. Roofs and roads were given very low Manning's numbers considering the bitumen, concrete and/or roof materials. Paved areas and structures other than building roofs were assigned the same values. Gardens, POS and other pervious areas were considered as surfaces covered with vegetation and disturbed either with trees or structures. Ponds and water-logged swales were assigned an average of 0.025 considering vegetation cover at the banks. The values were refined by model calibration. It was cited that the values for the roughness coefficient are reasonable with respect to the large size of the modelled catchment, after sensitivity analysis. Concrete pipes, culverts and roads as 1D flow paths were given the Manning's roughness number of 0.014, while natural channels were assigned a value of 0.025 to represent the vegetated banks. Drainage roughness coefficients used in the model are given in Table 6.

The total modelling process was broken down to three major catchment models considering the software licenses (i.e. node limitation of the version used was 500 nodes). Therefore the Glenariff sub-catchment model, the Avenues sub-catchment model and the Main Drain sub-catchment model were separately built. The flow hydrographs from each upstream sub-catchment model were fed to downstream models for each and every scenario.

Table 6. Manning's roughness values used in the drainage network

ITEM	MANNING'S ROUGHNESS VALUE
Concrete underground pipes	0.014
Road network (carries the portion of the surface flow)	0.014
Open channel sections	0.025

A sensitivity analysis and model validation process was carried out for the study to select sensitive catchment characteristics and their values. The Glenariff catchment model was used to analyse the sensitivity of catchment characteristics. The Avenues sub-catchment model was used for calibration and verification. The results for the calibration and verification of models are given in Chapter 4. The infiltration values and surface roughness values finalized after the calibration are given in Table 6. These values were used for all the sub-catchment models.

5.4.1. Future development scenarios

The Warrendale Nursery subdivision site and the low elevation area of Fraser Road North development site are proposed for development in the future. This section discusses the special modelling considerations and the predicted stormwater scenarios for these sites before and after the developments. Also the Church subdivision site bounded by Philadelphia Parade, Norwich Road and Amherst Road, where there was low elevation bare land, is already developed. The impact of this subdivision work on the total Central catchment hydrology is also discussed here. This study considered these three sites as special cases and they have been remodelled to find the impact on the Central catchment under post-development conditions.

Warrendale Nursery Site is one of the proposed subdivisions. The model has been re-run for the post-development case assuming the site is 75 per cent impervious (including the proposed basin area). This land lot is being used as a flood storage area and also a proposed basin (Basin #2). Therefore the site development is assumed to be a development that would be in accordance with the best urban water management guidelines (WAPC 2008). During the post-development modelling, the basin (having area of 500 m^2 and depth of 0.9 m) has been assumed to cater for the runoff only from the site. The basin is acting as an off-line basin. Fraser Road North site has been proposed for development, which will lead to the loss of another flood storage area in the Central catchment and finally may lead to an increase in the peak flow and water levels in the MUC and in Bracadale POS. The model has been re-run for the post-development case assuming the site is 75 per cent impervious (including the proposed basin area). The land lot has been elevated to 22.00 m AHD to be level with adjacent lots. The flow from the site was directed to the MUC as a sheet flow in the hydrology layer. The Church sub-division area was bare land before development and the lowest elevation is at about 21.80 m AHD. It is filled to 22.7 m AHD assuming the same elevation as Amherst Road. Church subdivision site was modelled as per its current situation in the second scenario. The land area is considered as about 75 per cent paved, including the roads.

5.5. Results and discussions

Two scenarios have been considered during the study, as follows:

Scenario 1 – Without the effect of groundwater on stormwater runoff

Scenario 2 – With the effect of groundwater on stormwater runoff

Scenario 2 was considered as the worst-case scenario, which is closer to the actual situation when considering the shallow groundwater table of the area. Models were again run for four major rainfall events of 1 year, 5 year, 10 year and 100 year ARI events for both scenarios. The following modelling results are based on these model runs and they were compared against the previous studies' results, where possible.

5.5.1. **Outflow hydrographs**

The outflow hydrographs for the Glenariff sub-catchment for 1 year, 5 year, 10 year and 100 year ARI events are given in Figure 17. The controlled outflow can be seen clearly for all the events. Maximum outflows were just under 0.07 m^3/s, (approximately 1.10 L/s/ha) and less than the maximum outflow limitation of 1.13

L/s/ha. The groundwater effect is not been analysed in this sub-catchment because the AAMGL is below the drainage invert levels (there were a few exceptions, but these were neglected considering their sensitivity to downstream catchment hydrology under the controlled outflow from Glenariff). Previous studies noted that there is a stormwater contribution from this catchment to the Central Catchment only for events above the 10 year ARI. However the results show that there is an outflow from Glenariff even for a 1 year ARI event after the assumption of 75 per cent impervious land use.

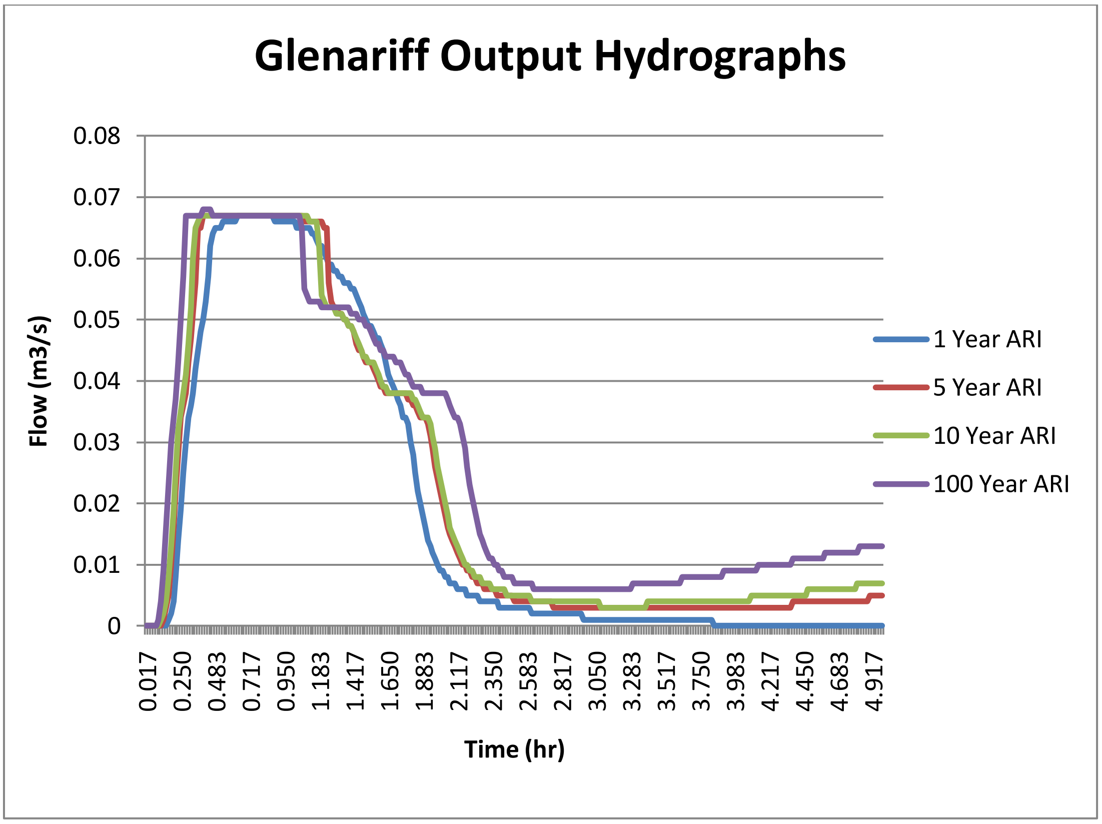


Figure 17. Glenariff outflow hydrographs for major ARI events

The outflow hydrographs for the Sanctuary Lake sub-catchment for 1 year, 5 year, 10 year and 100 year ARI events are given in Figure 18. The location at which that outflow hydrograph was obtained is the eastern end point of the Sanctuary Lake sub-catchment at Ranford Road. The results show the hydrographs for the worst-case scenario, which considers the groundwater effect. The results show that the 1 year, 5 year and 10 year ARI stormwater outflows from Sanctuary Lake sub-catchment to Avenues catchment are under the existing limitation of 1.13 L/s/ha. There is an outflow of 0.09 m3/s (approximately 1.21 L/s/ha) for the 100 year ARI event.

However no remediation was suggested in this study to control this flow, since it happens only for a 100 year ARI event.

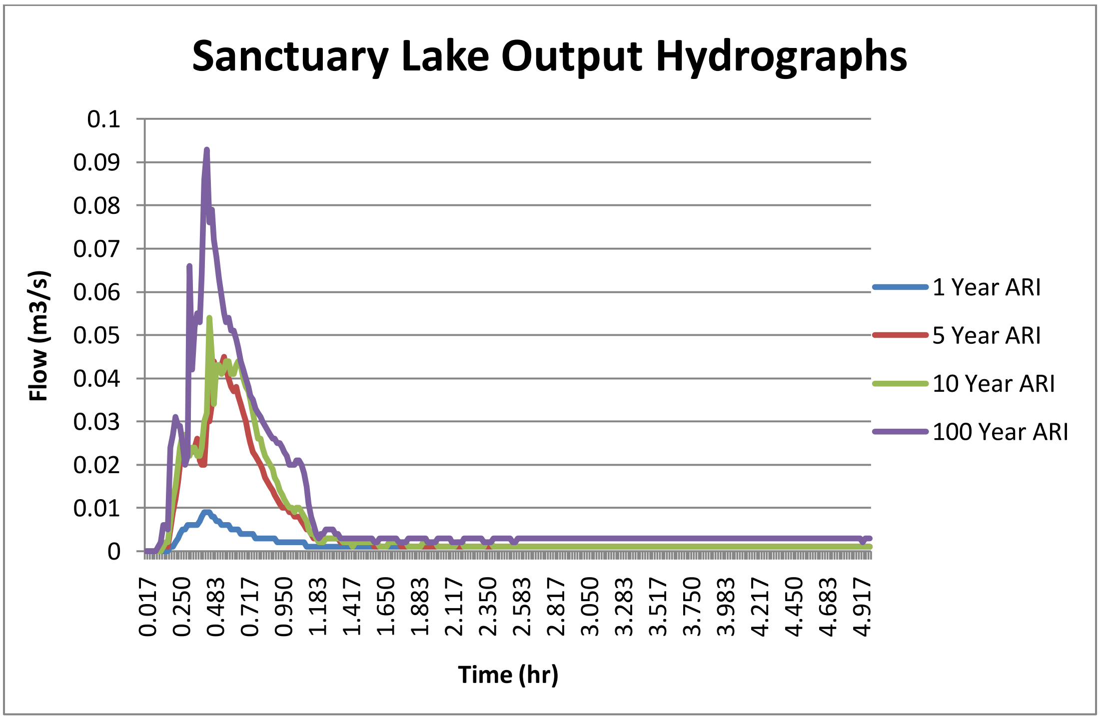


Figure 18. Sanctuary Lake outflow hydrographs for major ARI events

The maximum observed peak outflow from Central Catchment at DN450 pipe at the Shreeve Road observation location COG 3 was 0.084 m^3/s, while the XPSWMM model peak outflow was 0.12 m^3/s for the rainfall used in the model calibrating process. Even though the maximum outflow from the XPSWMM model is slightly higher than the observed maximum outflow, observed and modelled water levels of the Avenue Basin #1 tally well enough to predict the results of basin outflows, top water levels and finally the flood inundation areas based on this model. The outflow hydrographs from the Avenues sub-catchment at the outlet of Avenues basin 1 for 1 year, 5 year, 10 year and 100 year ARI events under both scenarios are shown in Figure 19.

The peak outflows from a 1 year ARI event for scenario 1 and 2 are 0.11 m^3/s and 0.12 m^3/s. The peak outflows for a 5 year ARI event for scenario 1 and 2 are again 0.16 m^3/s and 0.175 m^3/s. The peak outflows for both scenarios for 10 year ARI event are just under 0.2 m^3/s and for 100 year ARI event again just under 0.3 m^3/s. The hydrographs show that the flow decreases gradually after the one-hour rainfall event is finished. Results show that there are significant variations in the outflow hydrographs after the groundwater effect is introduced. Again the results obtained by using longer run-times show that the declining of the flow is delayed by the groundwater, which is closer to the actual observed scenario. This has suggested that there is a considerable effect of groundwater into the Avenues outflow.

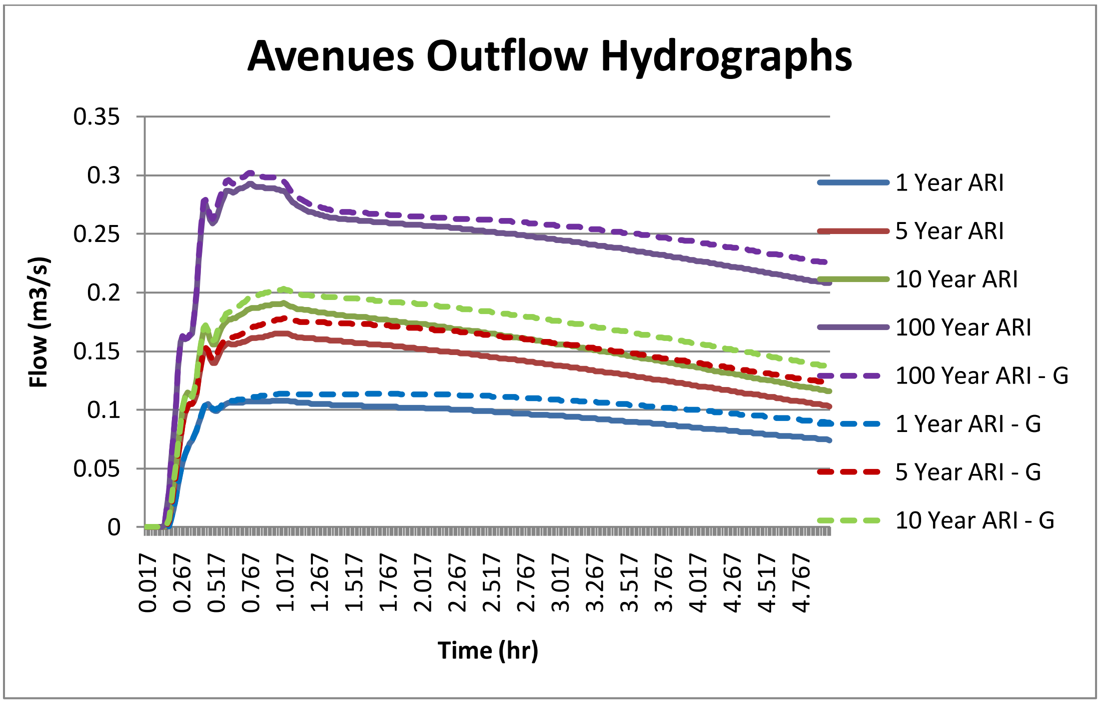


Figure 19. Avenues outflow hydrographs for major ARI events

The outflow hydrographs for the Main Drain sub-catchment (after the contributions from above mentioned upstream sub-catchments) for 1 year, 5 year, 10 year and 100 year ARI events under both scenarios are shown in Figure 20. These outflow hydrographs are based on DN450 pipe only, but do not consider the flow from the MUC through the box culvert at Shreeve Road. The results show that there is a sudden jump of all the hydrographs, from zero to above 0.1 m^3/s values, after about 45 minutes. The peak outflows for 1 year, 5 year and 10 year ARI events are around 0.12 m^3/s for both scenarios. This is because of the DN450 pipe is utilised fully for all the scenarios. The groundwater effect does not significantly affect the peak outflows, but as for the Avenues outflow hydrographs, it delays the declination of the outflow. The 100 year peak outflow for both scenarios are the same and about 0.14 m^3/s. This peak outflow exceeds other peak outflows because of the higher water head of the inundation areas along the MUC and also in the basins. However, the 100 year ARI hydrographs continue at same level for more than 5 hours.

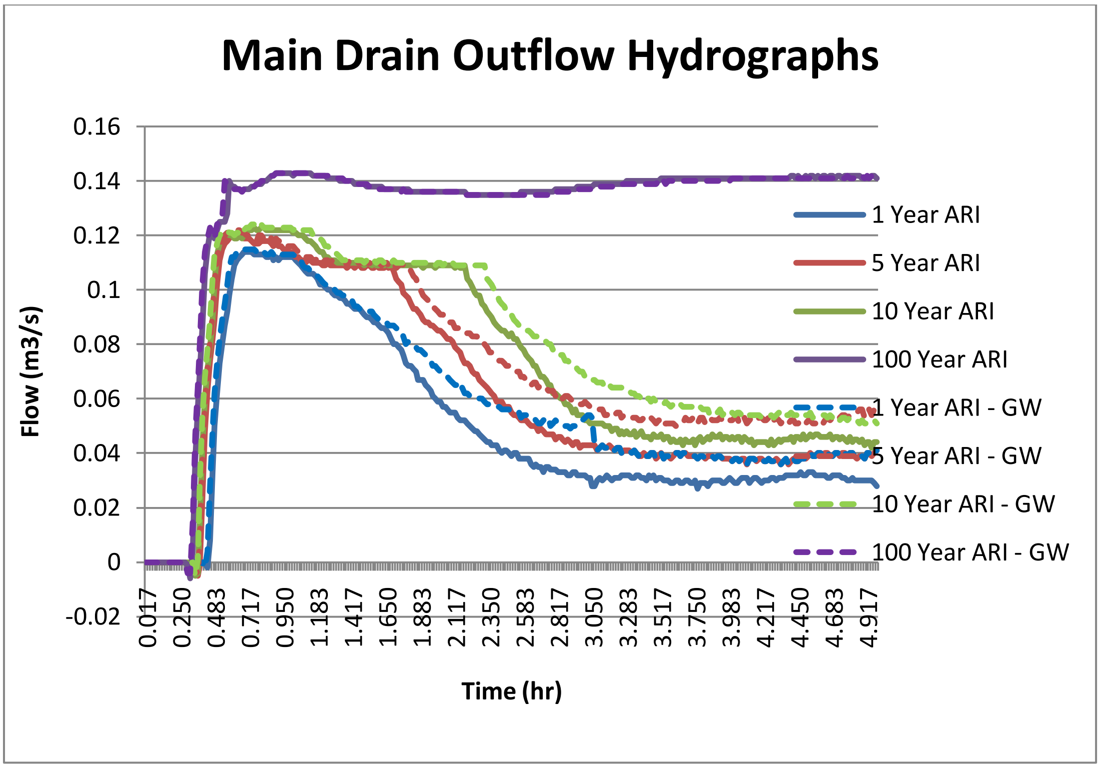


Figure 20. Main Drain outflow hydrographs for major ARI events

The total peak outflow hydrographs from whole Central Catchment across the Hughes Street open channel are shown in Figure 21. The hydrographs show the total outflow from the combined MUC and DN450 pipe outflows. The impact of groundwater on total outflow is clearly visible from the variation of the hydrographs under the two scenarios. The groundwater aids the continuation of flow, after the impact of the intense of rainfall is reduced with time. The peak out flow from the catchment for the 1 year, 5 year and 10 year ARI events are about 0.13 m^3/s, 0.18 m^3/s and 0.21 m^3/s respectively. The peak outflow for the 100 year ARI event is about 0.72 m^3/s. Therefore the total peak outflow for 100 year ARI event is 2.16 L/s/ha considering the total area of 332.8 m^2 for the Central catchment (including the upstream sub-catchments Glenariff and Sanctuary Lake). However the outflows from all the other ARI events below a 100 year event are within the allowable limit of 1.13 L/s/ha.

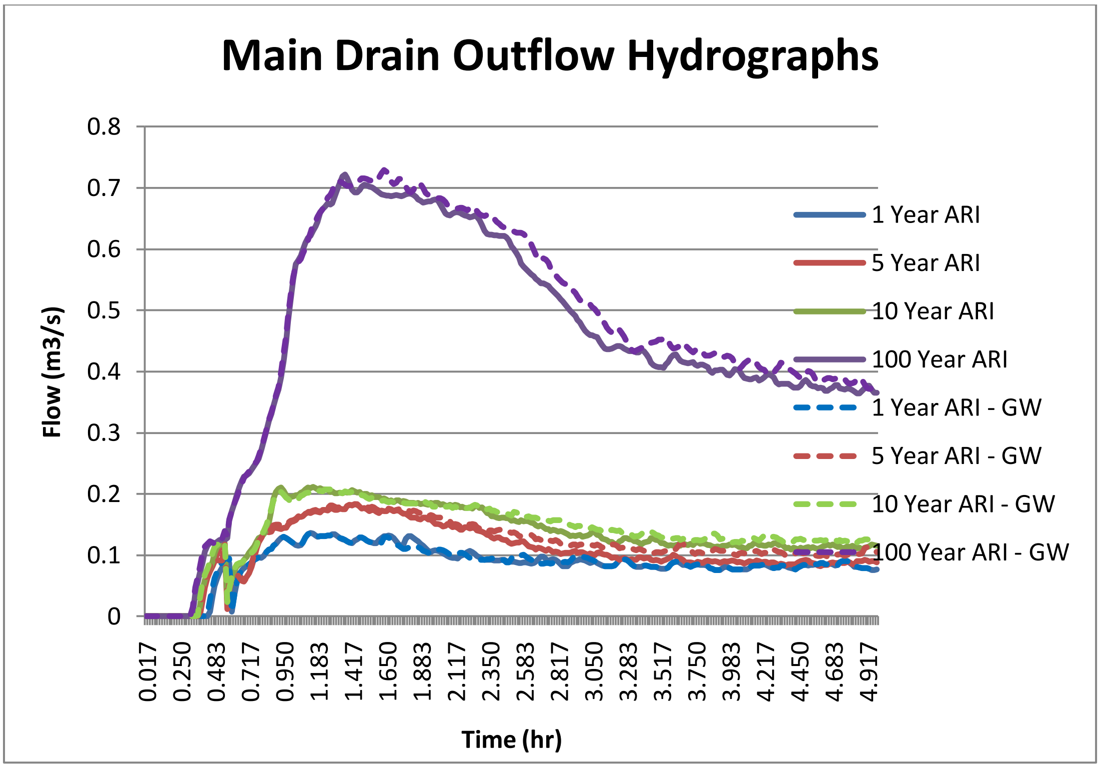


Figure 21. Central catchment total outflow hydrograph for major ARI events

5.5.2. Top water levels and peak flows

Basin top water levels and peak outflows model results, and the previous study results for the basin top water levels and peak outflows for 1 year, 5 year, 10 year and 100 year ARI events, are given in Tables 7 – 10. Basin design top water levels are also given to identify the overtopping basins. Avenues #1 top water level for a 100 year ARI event is around 22.67 m AHD for Wagner (2009) results as well as this study's results. The peak outflow for a 10 year event is zero, according to the past studies, while this study shows outflows even for a 1 year event from the basin. The peak outflow from the basin for a 100 year ARI event noted by Wagner (2009) is greater than this study's results. The basin's designed top water level of 23.45 m AHD was selected from the as-constructed drawings, but the contour shows it is around 22.40 m AHD which tallies with this study's top water level results.

Avenues #2 basin has no outflow for 1 year and 5 year ARI events. There would be a minor 0.01 m^3/s outflow for a 10 year ARI event and significant outflow for a 100 year ARI event, according to the results. However, the previous studies suggest that there are significant outflows for both these events from the basin. The recent modifications made by adding a weir in manhole CV 2281 near the Dumbarton Road and Packenham Road roundabout, blocking the outflow from the basin, might be the reason for this difference. However there is a significant flow into the basin from the eastern sub-catchments adjacent to the basin, according to the model.

Piesley Promenade has been modified recently to allow it to take a greater stormwater volume than it previously could from the part of the promenade north of the basin. These modifications are not represented in the contours, but the boundary levels at the roads adjacent to it have not been changed and are represented correctly in the model. The top water level of the basin for a 1 year ARI is 22.41 m AHD and from a 5 year ARI to a 100 year ARI the top water levels vary from 22.44 m AHD to 22.52 m AHD. The outlet line from the basin has a 22.13 m AHD high point at manhole CV2279 and the limitation of outflow from the basin seems to be controlled based on this elevation. The tail-water condition at Comrie basin causes the 600mm diameter Comrie inlet pipe to be fully equipped and causes to a back-flow. The outlet from Piesley combines with this back flow. Ultimately it impacts in the slow outflow from Piesley. Under these full capacity states of the outflow line, Piesley has same output flow rate of about 0.04 m^3/s for all the rainfall events.

The top water level of the Comrie basin was considered to be the lowest weir crest level at Auckland swale, which was 21.90 m AHD. The weir outflow was considered to be the Comrie outflow. A 1 year event outflow for Comrie basin was 0.0038 m^3/s and 0.004 m^3/s for both scenarios and there were small increments in flows under a 5 year ARI event. Previous studies do not provide any outflow value for this basin for the 5, 10 and 100 year ARI events as well, but there were about 0.08 m^3/s and 0.116 m^3/s outflows from this basin for the above events under both scenarios. The observational data suggests that the top water level is just above the 21.90 m AHD for minor rainfall events. The basin top water level remains at the bottom of the weir depth during minor rainfall events according to the model, but increases to 0.46 m in a 100 year ARI event.

The Auckland rainwater garden's top water level for the 1 year ARI event scenario 2 and all other major ARI events is higher than the basin's designed top water level, derived from the contour map. However the basin has its POS, which is noted by the City of Gosnells as a groundwater logged area during the winter, is also inundated up to adjacent road levels for all the events. The structure of the rainwater garden was modified and its boundaries were elevated. The modified structure can be filled up to a 22.26 m AHD level during a 100 year rainfall event without inundating the adjacent properties. However, the previous studies show lower top water level than the current contour elevations, except for the 100 year ARI event, probably based on the proposed basin level. This study's results for the peak outflow were over-topping flow from the POS to Auckland swale across its north eastern boundary.

There was a bubble-up outlet from Auckland swale to the Warrendale Nursery site. The average maximum top water level of the site for a 100 year event is 21.92 m AHD, according to the model results. Previous studies give this as 22.39 m AHD (JDA 1999) but the lowest level along the site boundary was found to be 22.22 m AHD. JDA (1999) had been based on design basin top water level and this study used the existing contours of the location. However this study's results suggest a higher outflow of 0.62 m^3/s than JDA's value of 0.02 m^3/s from the site into Doncaster open channel #2, while keeping the top water level under the lowest site boundary level.

The Extra Basin #2 was modelled as per the existing contours and having its lowest boundary level at 22.20 m AHD. The exact link from this basin to pipe drainage was not been found from the as-constructed design drawings of the drainage system. Also, the inlet drainage system has not been found according to the same source. Therefore the basin was modelled in the 2D layer. It has top water levels at 21.60 m AHD and 21.70 m AHD for 1 year and 100 year ARI events under a worst-case scenario. The Church subdivision site's stormwater flow and adjacent small catchment area may contribute to this basin, but results of the water levels and flow have been based on the contour map during this study. The outflow from the basin is calculated from the sheet flow across the basin.

The Belfast basin had been considered as a design basin in previous studies, but the present contours were used in this study. The basin top water level was under its lowest boundary level for all the scenarios. However the over-topping outflow from the site was measured from Belfast to the MUC across the road. The elevation of the culvert conveying the stormwater from Belfast to the MUC seems to be higher than the highest water level at the basin, according to the contour maps.

Table 7. Major basin top water levels and outflows for 1 year ARI event.

BASIN NAME	BASIN DESIGN TWL (mAHD)	RESULTS			
		WITHOUT GROUNDWATER		WITH GROUNDWATER	
		TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)
Avenues #1	23.45**	22.31	0.101	21.33	0.112
Avenues #2 (Baychester Lake)	22.53	22.08	0.000	22.10	0.000
Extra Basin #1A (Piesley)	22.80	22.41	0.038	22.41	0.039
Basin C1 (Comrie Basin)	21.90	21.90	0.02	21.90	0.022
Basin C2 (Warrendale Nursery Site)	22.20*	21.57	0.011	21.60	0.010
Extra Basin #2A	21.60*	21.72	0.000	21.60	0.000
Basin C3 (Auckland Rainwater Garden)	22.05*	21.90	0.020	22.05	0.020
Basin C4 (Belfast Basin)	21.45*	21.26	0.000	21.35	0.000
Basin C5 (Coulthard Crescent Ground)	21.50*	20.94	0.000	20.94	0.000
Basin C6 (Within Bracadale POS)	21.20*	20.65	0.000	20.66	0.000
Basin C7 (Within Bracadale POS)	21.20*	20.60	0.007	20.63	0.008

`TWL – Top Water Level
 * Basin design top water level was selected from the available contour maps
** The height was taken from the 'as-constructed drawings (100yr TWL)', but it was not the top water level according to the contours.
Note: Glenariff has more than one basin and they all are designed basins, so were not included in detail here.

The Coulthard Crescent Ground was considered to be Basin C5 in previous studies and given that design basin details, but here it was given the existing contour elevation. It acts as a basin for all the rainfall events. The elevated boundary separating the ground and the MUC prevents the water flow from the ground into the MUC for all the ARI events except for the 100 year ARI event. The 100 year ARI event peak outflow over-topping this boundary into the MUC is 0.39 m^3/s in this study and it is higher than the previous studies' peak flows. By having a higher outflow and probably more surface area than the design basin, the top water level of the ground is lower than the previous results and lower than the boundary levels separating the ground from the adjacent residences.

Table 8. Major basin top water levels and outflows for 5 year ARI event.

BASIN NAME	BASIN DESIGN TWL (mAHD)	RESULTS			
		WITHOUT GROUNDWATER		WITH GROUNDWATER	
		TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)
Avenues #1	23.45**	22.40	0.152	22.45	0.168
Avenues #2 (Baychester Lake)	22.53	22.20	0.000	22.25	0.000
Extra Basin #1A (Piesley)	22.80	22.44	0.039	22.44	0.039
Basin C1 (Comrie Basin)	21.90	21.90	0.05	21.92	0.06
Basin C2 (Warrendale Nursery Site)	22.20*	21.70	0.030	21.70	0.030
Extra Basin #2A	21.60*	21.69	0.002	21.69	0.002
Basin C3 (Auckland Rainwater Garden)	22.05*	20.14	0.025	22.15	0.025
Basin C4 (Belfast Basin)	21.45*	21.42	0.090	21.45	0.090
Basin C5 (Coulthard Crescent Ground)	21.50*	21.04	0.000	21.04	0.000
Basin C6 (Within Bracadale POS)	21.20*	20.76	0.004	20.80	0.004
Basin C7 (Within Bracadale POS)	21.20*	20.74	0.014	20.75	0.014

* Basin design top water level was selected from the available contour maps
** The height was taken from the 'as-constructed drawings (100yr TWL)', but it was not the top water level according to the contours.
Note: Glenariff has more than one basin and they all are designed basins, so were not included in detail here.

Previous design basins of Basin C6 and Basin C7 are located inside the Bracadale POS. This study used the existing contours to model them. The previous studies’ top water level for 10 year and 100 year ARI events exceed the existing POS’s lowest boundary elevations. However, the results of this study show the top water level for both basins (ponds) are lower than the existing lowest boundary levels of the POS. The 100 year ARI top water levels in this study for both basins are 20.92 m AHD, while the lowest boundary level of the POS is 21.13 m AHD. The outflow from

Basin C6 is measured across the boundary between Basin C6 and Basin C7. The Basin C7 outflow is the outflow from the culvert at Shreeve Road. This is the end-point of the MUC as well.

The low elevation land spot at Fraser Road North has been considered as a major flood storage area during all the rainfall events, but it has not been listed under these tables.

Table 9. Major basin top water levels and outflows for 10 year ARI event

BASIN NAME	BASIN DESIGN TWL (mAHD)	JDA RESULTS	WAGNER RESULTS		RESULTS OF THIS STUDY			
					WITHOUT GROUNDWATER		WITH GROUNDWATER	
		TWL (mAHD)	TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)
Avenues #1	23.45**	-	22.26	0.000	22.44	0.178	22.47	0.203
Avenues #2 (Baychester Lake)	22.53	-	22.91	0.282	22.24	0.010	22.26	0.010
Extra Basin #1A (Piesley)	22.80	-	22.31	0.005	22.45	0.039	22.45	0.040
Basin C1 (Comrie Basin)	21.90	22.20	22.05	0.000	21.91	0.120	21.91	0.130
Basin C2 (Warrendale Nursery Site)	22.20*	-	-	-	21.75	0.037	21.74	0.038
Extra Basin #2A	21.60*	-	21.62	0.340	21.69	0.007	21.69	0.023
Basin C3 (Auckland Rainwater Garden)	22.05*	21.85	22.21	0.060	22.15	0.025	22.16	0.027
Basin C4 (Belfast Basin)	21.45*	21.33	21.45	0.551	21.43	0.140	21.43	0.140
Basin C5 (Coulthard Crescent Ground)	21.50*	21.17	21.04	0.000	21.08	0.000	21.08	0.000
Basin C6 (Within Bracadale POS)	21.20*	20.94	21.06	0.000	20.80	0.004	20.82	0.012
Basin C7 (Within Bracadale POS)	21.20*	20.85	20.96	0.000	20.80	0.017	20.82	0.019

* Basin design top water level was selected from the available contour maps
** The height was taken from the 'as-constructed drawings (100yr TWL)', but it was not the top water level according to the contours.
Note: Glenariff has more than one basin and they all are designed basins, so were not included in detail here.
This table includes the results from JDA (1999) and Wagner (2009).

Table 10. Major basin top water levels and outflows for 100 year ARI event.

BASIN NAME	BASIN DESIGN TWL (AHD)	JDA RESULTS		WAGNER RESULTS		RESULTS OF THIS STUDY			
						WITHOUT GROUNDWATER		WITH GROUNDWATER	
		TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)	TWL (mAHD)	OUTFLOW (m^3/s)
Avenues #1	23.45**	N/A	0.045~	22.68	0.492	22.67	0.293	22.70	0.302
Avenues #2 (Baychester Lake)	22.53	N/A	N/A	23.70	0.262	22.43	0.029	22.43	0.029
Extra Basin #1A (Piesley)	22.80	N/A	N/A	22.40	-	22.50	0.040	22.52	0.040
Basin C1 (Comrie Basin)	21.90	22.62	0.090	22.44	-	21.95	0.175	21.97	0.190
Basin C2 (Warrendale Nursery Site)	22.20*	22.39	0.020		-	21.92	0.062	21.92	0.062
Extra Basin #2A	21.60*	N/A	N/A	21.71	-	21.71	0.035	21.71	0.035
Basin C3 (Auckland Rainwater Garden)	22.05*	22.17	0.060	22.55	-	22.26	0.410	22.27	0.430
Basin C4 (Belfast Basin)	21.45*	21.76	0.030	21.77	0.367	21.37	0.390	21.37	0.390
Basin C5 (Coulthard Crescent Ground)	21.50*	21.57	0.060	21.40	0.152	21.19	0.300	21.20	0.320
Basin C6 (Within Bracadale POS)	21.20*	21.30	0.070	21.44	0.084	20.92	0.240	20.92	0.240
Basin C7 (Within Bracadale POS)	21.20*	21.13	0.045~	21.32	0.079	20.92	0.530	20.92	0.550

* Basin design top water level was selected from the available contour maps.
** The height was taken from the 'as-constructed drawings (100yr TWL)', but it was not the top water level according to the contours.
Note: Glenariff had more than one basin and they all are designed basins, so were not included in-detail here.
This table includes the results from JDA (1999) and Wagner (2009).

5.5.3. **Flood inundation maps and flood vulnerability**

The flood inundation maps for the 1 year, 5 year, 10 year and 100 year ARI rainfall events are based on the worst-case scenario with the groundwater base-flow into the drainage system taken into account. The maps show the areas of possible risk of flooding and the predicted flood levels based on the existing contours. The 0.1 m interval was selected for the colour code to represent the flood levels. Results are displayed from the 0.01 m level, which is not being considered as inundated to improve the readability of the maps. The flood depths above 0.7 m, which are the basin flood depths, are not been re-distributed with a colour code since the areas other than the basins would have to be represented in detail. Both Avenues basins, as well as Piesley and Comrie basins were given initial water depths and base level within the 2D layer and show the flood depths accordingly. The flood depths in the basins change according to the side slopes, but this has not been considered during the flood mapping. The results are highly sensitive to the contour map. All ARI events generate flooding in most of the low elevation areas, but the flood levels slightly increase with the rainfall's intensity.

The flood map for the 1 year ARI event for Glenariff sub-catchment is given in Figure 29 in Appendix A. There are some areas that can be identified as water retention areas. If the model considered the best management practices which are supposed to be used in development sites, these water retaining areas would not be visible. Above a 5 year ARI event, flood mapping results are given in Figures 30-32 in Appendix A. All these scenarios have shown there are flood vulnerable areas. Most of the potential flooding areas are those where major basins were proposed. The flood depth from the ground elevation is shown as an index and the maximum possible flood depth in the reservation area would be 1.33 m. For all the other residential areas this value is less than 0.2 m. Further, there were many assumptions, especially on the design basins that were used to model this catchment. Also, the topography will change during future developments. The land use and lot yield values will be the key factors that will reduce infiltration. Results can further be tweaked by adding those values to the model in the future.

Flood vulnerability maps of Sanctuary Lake and Avenues sub-catchments for 1 year, 5 year, 10 year and 100 year ARI events are given in Figures 33 - 36 in Appendix A. There was no flood inundation risk identified for the 1 year ARI event. Bennett Drive, Yindana Entrance, Polaris Way and Bremner Circle, located at Sanctuary Lake sub-catchment, are shown to have a low flood risk (under 0.1 m flood depth) for the 5 year and 10 year ARI events. The playground at Lexington Avenue and Rushmore Avenue were inundated up to about 0.1 m flood depth for 5 and 10 year ARI events. The playground was inundated up to 0.2 m for a 100 year ARI event, but it will not pose a risk to the adjacent residential lots. The POS area in the Avenues #1 basin is almost inundated for major rainfall events. This can be expected when considering the level of AAMGL is almost at the surface level of the POS. Central Park Avenue might have some flooding during major rainfall events due to the fact that the POS is inundated above the road elevation of Central Park Avenue at its north western corner. Sanctuary Lake's water level can rise up to 0.85 m for a 100 year event.

Flood vulnerability maps of the Main Drain sub-catchment for 1 year, 5 year, 10 year and 100 year ARI events are given in Figures 37- 42 in Appendix A. Some areas downstream of this catchment near the MUC seem to have flood levels of 0.2 m even for a 1 year ARI event. The groundwater logged conditions during the winter, which prevents infiltration from major rainfall events, could be the reason for this condition. It has been identified that the DN450 pipe conveys groundwater from upstream catchments throughout the year. During the winter, the groundwater flow is higher than the summer and it may occupy the DN450 pipe fully.

There are some stormwater inundated clusters with 0.2 m depth, south of the Avenues #2 basin along the Engleswood Arc, but these may be showing because of older contour maps. It shows that the roundabout (where the weir structure controlling the Avenues #2 outflow is located) has a tendency to inundate up to the 0.2 m to 0.3 m level for a 100 year event. The POS area north of the Piesley basin and bounded by the Promenade has been modified into a rainwater garden and is flooded in all the events; the water depths and pattern may change when recently modified earthwork is uploaded to the model. All three crossroads, Stidwell Street, McKim Street and Gotch Crescent, can be inundated up to the 0.1 m level during major rainfall events. Houghton Street northwest of the Piesley basin can be inundated according to the maps. The playground south of Packenham Promenade is inundated only up to the 0.1 m level for all the ARI events and will not be an issue.

The Comrie basin has adequate capacity for all the ARI events. The flood boundaries of the basin expand slightly with the intensity of the ARI event while keeping the maximum flood depth of 0.46 m from its lowest weir crest level for the 100 year ARI event. The maps suggest that the bare land located south east of Comrie basin can be inundated up to 0.2 m to 0.3 m for 1 year to 100 year ARI events by extending the boundaries of the Comrie basin. The inundated area can extend up to about 0.6 ha for the 100 year event. The area to the north of Comrie basin up to Lausanne Way can be inundated up to about 0.4 m for a 100 year ARI event.

The maximum flood depths in Auckland rainwater garden is 0.6 m for a 1 year ARI event and above 0.7 m in a 100 year ARI event. However, the recent modification may change the flood levels once the recent contours are input to the model. The Auckland swale maximum water depth is about 0.6 m and 0.7 m for 1 and 100 year ARI events respectively. There are areas along the swale shown to be inundated up to 0.1 m to 0.2 m depths. However most of these areas should be reconsidered during an updating the model with accurate pipe drainage network data and land use details, when they become available. The comparison of the 10 year and 100 year ARI events' flood maps for the Auckland swale are given in Figure 43in Appendix A.

The Doncaster open channels and associated flood maps for 10 and 100 year ARI events are given in Figure 44 in Appendix A. The maps show the open channels are fully occupied for 1 and 100 year ARI events, but in the 100 year ARI the flooded area is widened. Two clusters at the south east of the open channels are shown in the maps as being inundated to more than 0.7 m, but they are roads. This should not be considered true as this is because the contour maps were not modified after the roads' construction. The stormwater from the Belfast proposed basin area is topping across Amherst Street as shown in the maps. This flow is a sheet flow with a low flow rate for minor rainfall events as given in the Table 7 and Table 8.

The starting point of the POS seems to be flooded for all the ARI events and the water depth can be up to 0.4 m. The flooded area widens with increased rainfall intensities. This is as expected, as the MUC was designed to act as a flood corridor for major rainfall events. The DN450 pipe is linked to the surface level along the MUC and the flow varies according to the water pressure, when the pipe is modelled with groundwater. The areas north of the Belfast basin are shown as 0.1 m flood inundated areas along the most of the roads. However, the new developments and pipe connections from the lots are not included in this study, as they are still being processed.

The bare land area, south to the Fraser road and west to the Cannich Boulevard is inundated in its lower elevation areas for all the major rainfall events. The flood maps of 10 year and 100 year ARI events for the MUC are shown in Figure 45 in Appendix A. Coulthard Crescent Ground is inundated for the 1 year ARI and 100 year ARI events up to depths of 0.1 m and 0.4 m. The ground acts as a flood storage area due to its elevated boundary that separates it from the MUC. The over-topping water from the flood storage area for a 100 year ARI passes into the MUC and the basin then acts as an off-line basin. Bracadale ponds are filled up to 0.6 m and over 0.7 m for 1 and 100 year ARI events. The POS area is almost inundated in the 100 year event, yet no flooding extends into the adjacent properties. The Hughes street open channel is clearly filled to its capacity under a 100 year ARI event. There are some clusters with low elevations in the bare land north of the open channel.

5.5.4. **Results for future development scenarios**

Warrendale site's post-development outflows were matched with pre-development, and allowed some capacity within the site to cater the increased runoff due to post-development infrastructure. Therefore no significant impact to downstream flows from the development has been resulted. Pre-development and post-development flood inundation maps for 10 year and 100 year ARI events are given in Figure 46 and Figure 47 in Appendix A. Loss of pre-development storage area has increased the water depth in the Doncaster open channels. The figures show that the 100 year post-development flood depth of channel #1 has been increased about 0.2 m from pre-development flood depth. A comparison of flood maps for current and future development scenarios of Fraser road for 10 year and 100 year ARI events is given in Figure 48 and Figure 49 in Appendix A. The land lot is a flood storage area and acts as an off-line basin to the MUC, which has a maximum flood depth of about 0.4

m at its northern boundary. The future development (after elevation of the surface level) shows only a negligible shallow water depth all over the site. This is due to the equally elevated surface. There is a thin water layer remaining under those conditions. It is recommended that proper survey data be used to increase the accuracy of the flood maps. The downstream peak flow in the MUC has increased from 0.4 m^3/s to 0.6 m^3/s in the future scenario. Also, the flood elevation of the Bracadale POS and in the location of proposed Basin #6 has been increased from 22.94 m AHD to 22.98 m AHD. The impact of losing the flood storage area again shows up in Pentland Street, which floods. The drainage along the road does not seems to have adequate capacity even in minor storm events under the future scenario. The 100 year ARI event's pre and post-development results for flood inundation for the Church subdivision site are shown in Figure 47 in Appendix A. The bare land spot south of the site, which has low elevation, seems to be filled up to about 0.3 m level. The pre and post-development flood inundation comparisons for Main drain sub-catchment are given in Figures 50 - 55 in Appendix A.

5.6. Summary of the results

The Central catchment has been modelled considering two scenarios: with the groundwater effect and without the groundwater effect. Four major ARI events of 1 year, 5 year, 10 year and 100 year have been considered to obtain the results. The results have been discussed with relation to peak outflows and the maximum water levels of the basins. The overall peak flows and runoff volumes increased from the anticipated levels, i.e. those used to design the stormwater drainage. Land use change has been found to be the reason for the incremental increase of flood levels in the basins and the cause to the localised flood inundation in low elevation areas. The sensitivity of catchment characteristics based on land use change, such as surface roughness and infiltration values, has been found to be significant.

Groundwater was found to have a major impact on the outflow from Avenues sub-catchment during initial field visits. Model results show that the groundwater impact is to keep a continuous base-flow from the Avenues basin and finally from the total Central catchment. The groundwater impact on peak outflows from Avenues basin is at a considerable level for all the rainfall scenarios. There is less impact in terms of peak outflows from the total Central catchment when considering the final outflow from DN450 pipe at Shreeve Road, and the combined total outflow at Hughes Street open channel. The effect of groundwater is to cause low infiltration on bare lands and this has been considered when deciding the infiltration loss component under both scenarios. Therefore, the shallow groundwater has an equal effect on both models when considering infiltration losses from pervious areas. The groundwater mounding through the drainage system has brought the peak flow of the second scenario slightly higher than the first, especially in later parts of the flow curves. In the meantime, it was found that the top water levels of the basins did not change much, nor did the depth and boundaries of the flood prone area due to the high intensities of the rainfall scenarios. It is clear that the groundwater effect on peak flow as well as surface water levels of the basins will be increased when the rainfall duration increases.

The peak outflows from Glenariff and Sanctuary Lake are within the limit of the Water Corporation's standards for maximum outflow, which is to be kept under 1.13 L/s/ha for all cases. Total peak outflow to the Hughes Street open channel from Central catchment is also under this limit for all the scenarios except for the 100 year ARI event, for which the outflow is 2.16 L/s/ha. Avenues #2, Piesley and Comrie basins have been modified either in their capacities or in controlling the outflows. These three basins and Extra Basin #2 act as off-line basins. The water depths of each basin have been analysed together with their peak outflows using available data.

Flood vulnerability maps for the total catchment have been produced for the above rainfall events. Localised flooding along the roads and its causes has been discussed. However these results are based on the available topographical contours, which are outdated for some of the recently developed areas. Localised flooding is highly sensitive to the topography and land use data, which it is suggested needs to be updated. However, the total flood depths shown in the maps are accurate enough to predict the flood risk area. The Avenues and two other upstream sub-catchments are not considerably challenged by flood risks except where there were a few locations identified as low level, where a maximum of 0.2 m flood depth can exist for major rainfall events. Downstream of the Central Catchment is more vulnerable to flooding as per the results. In particular, the land lots along the MUC may experience some inundations due to water from upstream catchments and from groundwater. The basins that were proposed but not yet constructed still have the capacity to cater for the excess water from the drainage system by having low elevations. Auckland swale and Auckland rainwater garden are functioning appropriately. The water level of the Auckland POS has been identified as being affected by the shallow groundwater at its surface level. Doncaster open channels are capable of handling major rainfall events and may inundate the POS area alongside them. Belfast basin is acting as a flood storage area, but major event excess runoff from the basin will overtop the Amherst Road. The culvert structure located there is not working as an appropriate outflow structure due to its higher elevation.

Future developments proposed in the centre of the Central catchment will occupy areas currently use as flood storage areas. Such developments (Warrendale Nursery site and Fraser Road North subdivision) are analysed against a 100 year ARI event and the impacts downstream have been discussed. The Warrendale Nursery site subdivision is proposed to have a basin to keep a portion of the runoff from upstream sub-catchments as it is currently acting as the proposed Basin #2 (JDA 1999). There is no considerable impact that has been identified by developing this site, provided there is an adequate basin size to match the pre and post-development flow for a 100 year ARI event. Fraser Road North proposed subdivision will impact by increasing the downstream peak flow at the MUC and water levels in the Bracadale POS. Also, the development of this land tends to flood the Dornoch Way road. Church subdivision site has been developed, removing the low elevation bare land which was acting as an extra flood storage area. The stormwater from this site has been assumed to be routed to the adjacent drainage system. No severe impact on the downstream flood levels from this development has been identified. However, the bare land lot at south west of the site will be flooded by extra amount of water under the current situation. All these modelling works are based on the assumed elevated fills for these areas and it is suggested that the model is updated when the earthwork plans for the developments are available.

5.7. Recommendations for stormwater management

It is recommended that the possible utilising of stormwater drainage capacities by groundwater, especially near Avenues Basin #1, is considered when preparing the stormwater management guidelines for future subdivision works and new constructions.

- Glenariff, Sanctuary Lakes and Avenues basins show outflows for the minor rainfall events below a 10 year ARI, but the peak outflows from basins for all the ARI events are within the limit of 1.13 L/s/ha. Maximum outflow from Glenariff should be maintained by using appropriate structural measures. It is recommended that the 1 year ARI event's runoff be kept within lots in future subdivisions.

- Upstream sub-catchments of Glenariff, Avenues and Sanctuary Lake have not been identified as areas where possible flood risk exists. However, there are some low level roads (below than 0.2m) where stormwater inundation is a possibility, as discussed under the flood mapping. These locations might be treated to careful consideration, checking drainage details and any major deviations of contours being used with current topography of the area.

- DN 450 pipe is flowing at its full capacity for all the rainfall events when considering the impacts of groundwater. Periodical monitoring of the sediment collection in the pipe is recommended as a maintenance activity.

- Comrie basin, Avenues #2, Piesley, Auckland Rainwater Garden, Coulthard Ground and Extra basin #2 were constructed as designed and are functioning well. They are acting as off-line basins as expected, due to some measures recently taken by the City of Gosnells, (i.e. constructing weir structures in Avenues #2 outflow line and a second weir structure at Auckland swale). The increasing of the capacities of Piesley, Auckland swale and Auckland Rainwater Garden have been acknowledged, but have not been modelled in detail due to a lack of survey contours. They should be modified in the model to obtain its best performance.

- The pipe outlet from Lausanne Way to the Comrie basin should be re-assessed for its length, and the location at which it connects to the basin.

- The proposed Basin #2 (Warrendale Nursery site) should be addressed properly during the subdivision work and the current peak outflow in a 100 year ARI event should be matched with the post-development stage to ensure the existing storage capacity of the site will mean it acts as an off-line basin during major rainfall events.

- The culvert structure at the outlet of Belfast basin should be re-assessed with its elevations to ensure its functionality as an outflow structure during storm events. The basin has not been constructed yet, but the area has some considerable capacity to attenuate the runoff from the area to its north. However, with the lack of proper operation of the outlet culvert, the model shows that there is an over-topping flow across Amherst Street and this should be prevented by installing a proper outlet.

- The Church subdivision site is already developed, which led to the loss of flood storage capacity of this low elevation area. It is suggested that the flow from this site and the original capacity of the flood storage area needs to be assessed. Inlet and outlet conditions of Extra Basin #2 should be assessed at the same time.

- Fraser Road subdivision will remove another flood storage area downstream of the Central Catchment. It is suggested that some flood storage capacity be allowed within this site in the post-development stage, to minimize development's adverse effect downstream of the MUC.

- The bare land north of the Hughes Street open channel seems to be inundated to some level during the major ARI events. It is suggested that future developments have adequate lot levels, above the 100 year flood levels of the open drain.

- The end of the catchment outlet at Nicholson Road should be re-checked for existing conditions, since this will slow down the flow and in the meantime increase flood depths along the Main Drain MUCs. The topography data used and the drainage details do not tally with each other.

- It is highly recommended that the latest topography data be used to produce much more accurate results from such a sophisticated model. The use of such data will increase the accuracy of predicting localised flooding, but will not impact significantly on the outflows and basin water levels.

CHAPTER 6

6. CASE STUDY OF VICTORIA PARK STORMWATER SUMP CAPACITY ASSESMENT

6.1. Introduction

The effects of urbanization and climatic change are outdating the rapidly developing Australian urban cities' stormwater management systems. Elevating an urban environment to a level that assures the quality and safety of the urban lifestyle by protecting the natural hydrology of the urban catchment, while facing changed weather patterns and increasing demand is very challenging. In order to do this, implementation of additional stormwater management structures—other than re-structuring the existing systems—is inevitable. Economic factors such as the value of urban land space and the cost of implementation should be considered when designing and constructing such additional stormwater management structures. Induced infiltration of urban stormwater into the ground is increasingly used as an alternative to its direct disposal to streams (SoSJ 2003). This can be done by introducing infiltration stormwater basins. Detention/retention basins are considered an effective tool for stormwater quantity and quality control in many urban areas and there are different views on the selection of optimum detention volume (Cordery and Pilgrim 1983). Infiltration basins are designed with the aim of attenuating major storm event runoff at the end of catchments, while letting a portion of stormwater to infiltrate into the groundwater table.

Victoria Park catchment has more than 100 stormwater retention basins. The majority of these basins were designed and implemented several years ago. The town has been growing in population and has urbanized rapidly. The urbanization caused changes in land use by removing most of the bare lands and pervious surfaces which were present when the stormwater basins were implemented. Therefore, some of the basins are not adequate for the current stormwater demand. Also, the basin capacities have been reduced due to continuous sediment collection on the basin bottoms over the years. This situation tended to produce localized urban flooding during storm events in the recent past. Also, some of the old stormwater basins were constructed without considering the actual required volume and the volume they process may be larger than the requirement. The land value of the town has increased extensively over the past couple of years due to its close location to the CBD of Perth. While keeping these facts in mind, the Town of Victoria Park proposed to assess the city's stormwater detention basins and develop a stormwater management and integrated land development master plan.

There are several methods to calculate the required capacities for the stormwater basins. For example, a study by Cordery and Pilgrim (1983) cited two such volume-based methods: the Federal Aviation Administration's method for stormwater detention designs and the capture volume method for stormwater retention designs. However, this study is based on the numerical modelling process to assess the catchment runoff generation and hence the required stormwater basin capacities. It also facilitated a series of infiltration tests to determine the infiltration conditions of the basins, to optimize the modelling results for the required basin capacities. Research was undertaken with an analysis of various stormwater sub-catchments within the town, in order to find their possible runoff generation under selected average recurrent interval (ARI) rainfall events. This study has identified the possible maximum capacities of stormwater basins and potential flood distribution within low elevation areas. The comparison of the model generated and optimized required top water level of the basins under different storm events, as against the actual top water levels of the basins, determined the requirements for basin modifications. Flood inundation maps for the town have been produced considering each major ARI rainfall event. Results of this assessment will help to increase the sizes of underestimated or inadequate stormwater basins to match with current demand and/or to oversee the construction of additional basins. They will also help to increase the land value and the beauty of the town by reducing the oversized stormwater basin capacities. Identification of localized flooding and flood illustration maps will lead to necessary flood mitigation actions. Ultimately the study will support the town's land development master plan.

6.2. Objectives and methodology

To assess the Victoria Park urban catchment's hydrology and stormwater sump (basins) capacities, a numerical model based on a 2D hydraulic surface water routing approach was developed. It identified potential top water levels of stormwater sumps and localized urban flood distribution in the area under selected major ARI events. A series of infiltration tests within the stormwater sumps was carried out to find the infiltration rates of these sumps. The hydrological analysis (flow characteristics) was combined with infiltration analysis to optimize the required sump capacities by determining the top water levels of the sumps. Finally the results were used to prepare flood inundation maps for the urban catchment under selected major ARI events. To achieve the above objectives, the research followed the following steps. The assessment mainly includes two sections, hydrological (modelling) assessment and infiltration analysis.

Hydrological Assessment

- Collect the secondary data on catchment properties (geography, topography and land use data).
- Define the sub-catchments using the topography maps.
- Develop numerical model to assess the urban catchment’s hydrology, maximum possible runoff generation under selected major rainfall events and stormwater basin top water levels and time taken to reach the top water level (TWL).
- Simulate the sub-catchments under different ARI rainfall events to produce urban flood inundation maps.
- Identify the high flood risk areas for further investigation.

Infiltration Analysis

- Identify the potential (high priority) stormwater sumps to conduct the infiltration tests (based on the recommendations from Town of Victoria Park).
- Conduct on site infiltration tests using Guelph Permeameter Kit (DEC 2011).
- Identify the infiltration rate of the basins and combine the infiltration values with modelling results (considering the time taken to reach TWL of the basins) to optimize the basin TWL under each ARI event.

6.2.1. **Available data and data collection**

- Historical rainfall data used to generate ARI events hydrographs (BoM 2012).
- Town of Victoria Park provided list of sumps and their locations
- Town of Victoria Park provided cadastral map of the catchment geographic information systems (GIS) and DWG formats.
- Town of Victoria Park provided 1 m interval contour maps. These GIS contour maps have been used to build the digital terrain model (DTM).
- Aerial photo available for the catchment has been used to model the spatial features in the model, i.e. the land use changes.

6.3. Catchment description

Town of Victoria Park catchment, comprising of an area approximately 11.71 km^2, is bounded to its north by the Swan River, to the south by Kent Street, to the west by Berwick Street and to the east by Rutland Avenue. It is located about 3 km south of the Perth CBD. The catchment is highly urbanized and land use is about 70 - 80 per cent impervious with the presence of infrastructure and buildings. Due to its closeness to the city centre, land value in the area is comparatively high and the cost of destruction of properties is high. Also, the space required for additional infrastructure and stormwater management structures can be an expensive loss to the city. During the study, Victoria Park watershed was divided into major 13 sub-catchments and another 72 minor sub-catchments within them, based on the topography of the terrain. The major and minor sub-catchments are shown in Figure 22. Each minor sub-catchment has one or more major stormwater sumps and is named according to the sump numbers. A list of sub-catchments (0 to 12) with their hydrological configurations and 5 year, 20 year, 50 year and 100 year ARI event rainfall data are shown in Table 11..

Topography

In Victoria Park, catchment topography contours are changing gradually from 5.00 m AHD at its northern boundary to 25.00 m AHD on its southern boundary. However, the grades throughout the area change irregularly. The approximate slope for each sub-catchment was selected considering the gradient along the longest flow path of the sub-catchment. Subsequently, the area was being developed with urban infrastructures and the original contours were modified accordingly. Minor changes to the terrain by recent land developments are not represented in the topographical maps that were used. Average annual maximum groundwater level contours are distributed variously from 3 m to 10 m running north to south across the terrain. Therefore, it was considered that there is no significant effect from groundwater on the infiltration rates of land surfaces and stormwater basins.

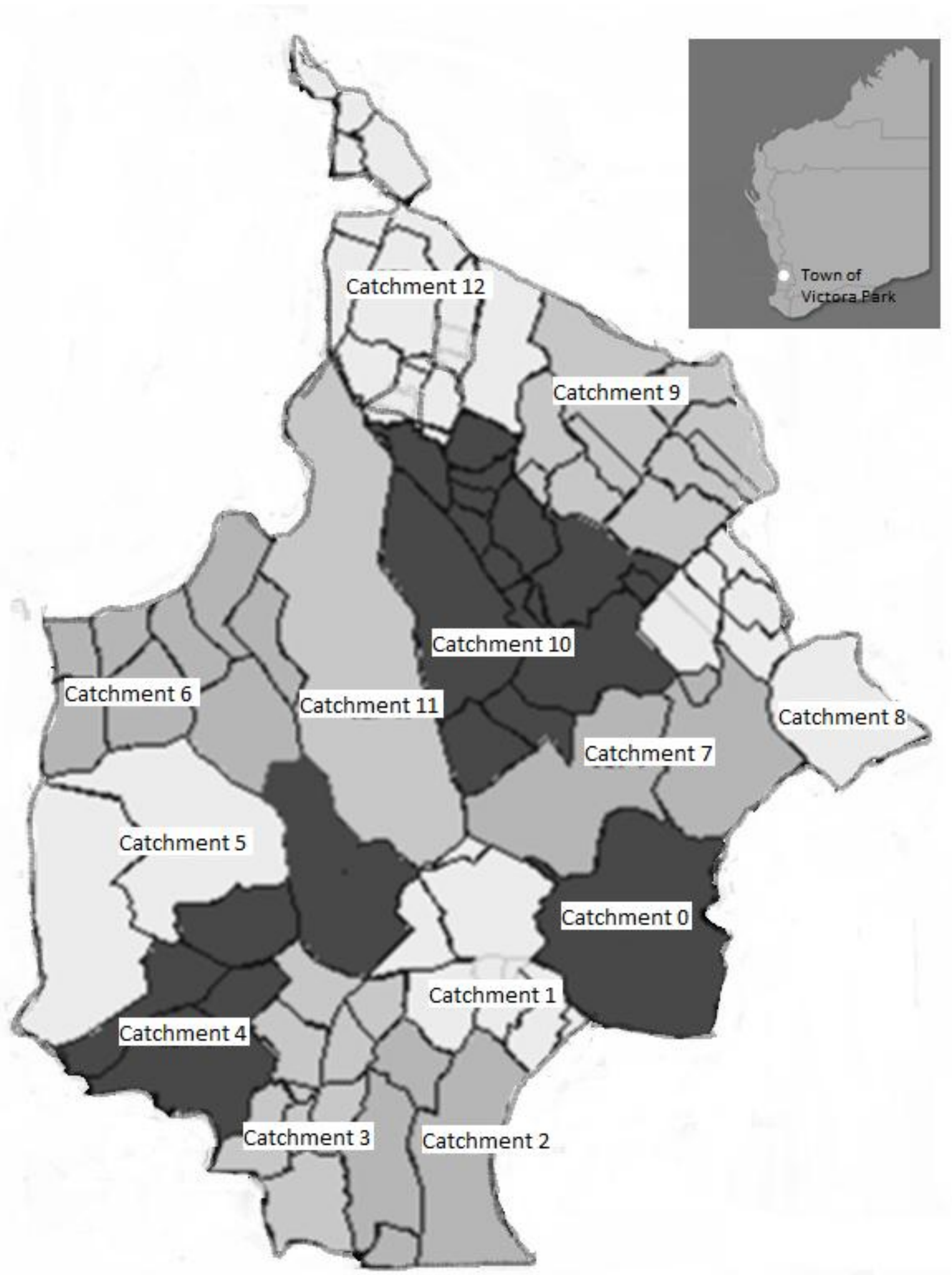


Figure 22 Town of Victoria Park sub-catchments, used for the hydrological analysis

Table 11 List of sub-catchments with their hydrological configurations and rainfall values

MAJOR SUB CATCHMENT NO	SUMP	SUB CATCHMENT	AREA (ha)	Tc (min)	AVERAGE RAINFALL (mm/hr)			
					1/5	1/20	1/50	1/100
CATCHMENT 0	S064	SC 064 - 00A	75.2	118.8	16	22	26	29
	S00A							
CATCHMENT 1	S003	SC 003 – 029 - 083	9.642	39.2	34	46	52	63
	S029							
	S083							
	S013	SC013 - 066	24.209	64.4	25	34	41	47
	S066							
	S034	SC034	15.622	50.9	28	37	44	50
	S085	SC085	5.44	28.8	41	55	67	77
	S086	SC086	9.227	38.3	34	46	52	63
	S087	SC087	1.88	16.2	60	85	105	120
CATCHMENT 2	S084	SC 084	9.76	39.5	34	46	52	63
	S088	SC 088 - 089	44.73	89.8	20	26	32	36
	S089							
	S090	SC 090	23.35	63.2	25	34	41	47
CATCHMENT 3	S001	SC 001	6.9	32.7	38	54	65	75
	S002	SC 002	6.29	31.1	38	54	65	75
	S037	SC 037	6.56	31.8	38	54	65	75
	S069	SC 069	10	40	34	46	52	63
	S070	SC 070	9.96	39.9	34	46	52	63
	S071	SC 071	9.15	38.1	34	46	52	63
	S03A	SC 03A	17.04	53.3	28	37	45	52
CATCHMENT 4	S005	SC 005 - 006 - 036	18.42	55.6	28	37	44	50
	S006							
	S036							
	S004	SC 004 - 035	24.44	64.8	25	34	41	47
	S035							
	S04A	SC 04A	31.39	74.1	23	30	36	41
CATCHMENT 5	S072	SC 072	56.3	101.6	18	24	29	33
	S05A	SC 05A	56.29	101.6	18	24	29	33
CATCHMENT 6	S007	SC 007 - 008 - 074	23	62.7	25	34	44	50
	S008							
	S074							
	S009	SC 009	10.4	40.8	34	46	52	63
	S010	SC 010 - 010A	8.32	36.2	36	50	60	70
	S010A							

MAJOR SUB CATCHMENT NO	SUMP	SUB CATCHMENT	AREA (ha)	Tc (min)	AVERAGE RAINFALL (mm/hr)			
					1/5	1/20	1/50	1/100
	S011	SC 011	10.37	40.8	34	46	52	63
	S076	SC 076	17.4	53.9	28	37	45	52
	S06A	SC 06A	12.62	45.3	31	42	50	58
	S06B	SC 06B	16.72	52.8	28	38	46	52
CATCHMENT 7	S07A	SC 07A	50.34	95.7	19	25	30	34
	S07B	SC 07B	41.65	86.4	20	27	32	37
CATCHMENT 8	S021	SC 021	13.6	47.2	30	38	45	52
	S023	SC 023	6.7	32.3	38	54	65	75
	S058	SC 058	4.3	25.3	45	61	74	86
	S059	SC 059	3.2	21.6	49	67	82	95
	S060	SC 060	5.9	30.1	38	54	65	75
	S061 S062 S08A	SC 061 - 062 - 08A	36	79.9	22	28	34	39
CATCHMENT 9	S019 S052	SC 019 -052	34.86	78.5	22	27	34	39
	S030 S049	SC 030 - 049	10.28	40.6	34	46	52	63
	S050	SC 050	7.94	35.3	35	48	65	75
	S051	SC 051	5.28	28.3	41	55	67	77
	S053	SC 053	9.18	38.2	34	46	52	63
	S054	SC 054	11.14	42.4	32	42	50	60
	S055	SC 055	7.56	34.4	35	48	58	68
	S056 S057	SC 056 - 057	15.46	50.6	28	37	44	50
CATCHMENT 10	S016	SC 016	3.24	21.7	48	66	85	96
	S017	SC 017	9.85	39.6	34	46	52	75
	S018	SC 018	1.26	13.1	66	95	125	135
	S022	SC 022	2.11	17.3	53	66	82	110
	S047	SC 047	8.26	36.1	35	48	65	75
	S048	SC 048	6.66	32.1	38	54	65	75
	S063 S10A	SC 063 - 10A	30.48	73	23	30	36	42
	S10B	SC 10B	19.26	56.9	27	36	43	50
	S10C	SC 10C	44.58	89.6	20	26	32	36
CATCHMENT 11	S012 S020	SC 012 -020	13.76	47.5	29	28	46	52
	S031	SC 031	14.84	49.5	28	37	44	50
	S032	SC 032 - 033	19.28	57	26	34	42	48

MAJOR SUB CATCHMENT NO	SUMP	SUB CATCHMENT	AREA (ha)	Tc (min)	AVERAGE RAINFALL (mm/hr)			
					1/5	1/20	1/50	1/100
	S033							
	S068	SC 068	7.66	34.6	37	50	61	70
	S073	SC 073	5.14	27.9	42	57	70	80
	S075	SC 075	3.18	21.5	50	67	83	96
	S077 S078	SC 077 - 078	13.1	46.2	31	41	50	57
	S079	SC 079	18.68	56	27	36	44	50
	S080	SC 080	8.4	36.4	36	48	59	67
	S081 S082	SC 081 - 082	18.42	55.6	27	36	44	50
CATCHMENT 12	S014	SC 014	3.56	22.9	46	62	75	85
	S015	SC 015	2.87	20.4	50	70	88	100
	S038	SC 038	23.61	63.6	25	34	41	47
	S040	SC 040	6.37	31.3	38	54	65	75
	S041 S046	SC 041 - 046	19.8	57.8	26	34	42	48
	S042	SC 042	8.25	36	35	48	65	75
	S043 S044	SC 043 - 044	6.75	32.3	38	54	65	75
	S045 S025	SC 025 -045	7.8	35	35	48	58	68
	S12A S12B	SC 12A - SC 12B	9.94	39.8	34	46	52	63

Climate

The seasonal weather and climatic data for the catchment was obtained from the nearest meteorology station, designated 'Perth Metro', which is 6.5 km away from Victoria Park. The catchment experiences a dry Mediterranean climate of hot dry summers and cool wet winters. Long-term climatic averages indicate that Victoria Park catchment is located in an area of moderate to high rainfall, receiving 741.1mm on average annually, with the majority of rainfall received between May and August. Monthly mean rainfall for July, the month having maximum rainfall of the year, was 152.7 mm during the period 1993 to 2011. However, the maximum recorded rainfall for the same period was 278.6 mm in 1995. The region experiences rainfall for 81 days annually (on average) (BoM 2012). Climatic changes have reduced the average rainfall distributed throughout the year, but increased the intensities of single storm events.

6.3.1. **Data availability**

Interval contours of 1 m are available for the Victoria Park catchment in GIS formats. These contour maps were used to generate the DTM, which was used during the hydraulic surface routing. There were no digitised drainage maps or specific details about the hydraulic features in the urban stormwater management system (USWMS). An aerial photo for the catchment was available. The details of stormwater basins were obtained from digitised maps. The rainfall data for 5 year, 20 year, 50 year and 100 year ARI events for the Victoria Park catchment were extracted with the given coordinates of -31.975S, 115.900E by using the Bureau of Metrology web-based application for generating intensity duration frequency (IDF) curves, (BoM 2012). The average rainfall values for each sub-catchment are given in Table 11. They were applied to the temporal hydrographs specific for the region, which are inbuilt as templates in XPSWMM.

6.4. Assessing the sump capacities

The sump capacity assessment was carried out in two steps, a numerical modelling assessment and a series of infiltration tests. The numerical model was run for several major rainfall events to identify the catchment runoff behaviour and sumps' top water levels (TWL) for selected major rainfall events. This has been done for all the sub-catchments and sumps within the Victoria Park catchment to select critical sub-catchments with higher flood risks. Then 25 sumps were selected in several suburbs and field-based infiltration tests were carried out for each of them. The infiltration tests were carried out to find the permeability of the soil and its influence on reducing sump top water levels during a storm event. The infiltration rate was multiplied by the average time it takes to reach the top water level of a sump, to calculate the infiltrated water depth. The effect of the infiltration from the sides of sumps and the volume of rain that falls directly on the surface of the sump can be neglected due to its small value when compared to the design volume of the system (Cordery and Pilgrim 1983). This depth was deducted from the sump top water level generated from numerical modelling to obtain the possible maximum top water level of the sump. The adequacy of the sump capacity was decided by comparing its available depth and the maximum water depth. The infiltration test values also helped to find out the time that may be taken to empty a sump after the top water level is reached.

6.4.1. **Hydrological modelling**

XPSWMM was used as the modelling tool for this study and its hydraulic routing option was directly applied with some initial settings. A 2D surface runoff modelling was used to analyse the whole catchment, in the absence of details for the hydraulic drainage network. A DTM was generated by using the 1 m interval contours. The initial time step for the 2D model was given as 2 seconds, to increase the accuracy. The wet/dry step was given as 0.004 m and a constant viscosity for water was used during the iterations. The spatial resolution was kept as low as possible, to increase the accuracy of the modelling results and used a 10 m by 10 m grid size. Considering the expected accuracy level of the results, an average of 0.025 roughness coefficient (Manning's number) was used across the areas. The lower Manning's number was used because of the highly impervious urban surfaces which make up 75 – 80 per cent of the land area of the catchment. The conditions at the boundaries were given according to the average topography along these boundaries. The infiltration loss was taken to be 5 mm/hr initial and 4.1 mm/hr continual for the terrain, considering the higher percentage of impervious land use.

The time of concentration, which was calculated manually for each of the sub-catchments, was used as the critical duration for the ARI events. Time of concentration t_c was calculated by using following equation, which is recommended for use in Western Australia by Pilgrim (1987), where A is the area of the sub-catchment:

$$t_c = 2.31A^{0.54} \tag{25}$$

The average land area of the selected sub-catchments within one major catchment was used to calculate the time of concentration.

6.4.2. **Infiltration tests**

Infiltration tests were carried out by using the Guelph Permeameter Kit (DEC 2011). The permeameter was capable of measuring the soil permeability allowing for multiple depths and multiple head heights, all in the same borehole. Measurements are based on both vertical (gravity) and lateral (capillary) flow from a point source of known head height forming saturated flow patterns, as would be the case in nature. The following testing procedure was used to measure the infiltration of the sumps, see Figure 23.

- A hole was drilled in the selected sump's bottom by using an auger, to a depth of 1 m. A bottled end piece, called a sizing auger, was used to retain a certain borehole size all the way through the borehole. Usually the hole was 5 cm in diameter. The soil auger was used to excavate until most of the hand auger was underground, which provided an excavated borehole of around 1 m.

- The permeameter was placed in the borehole and water was filled into the outer and inner reservoirs to generate the water head.

- Dial was set, so that either the inner reservoir or both reservoirs could be used. As the inner reservoir is smaller in diameter and holds less water, it was used for locations with low permeability soils such as clay.

- The valve was released, so that the water could enter into the soil. The time versus water level drop was recorded. Time intervals were selected according to the speed the water level was reducing. (If water seemed to be running quite quickly, a shorter time interval of 30 seconds was selected. Otherwise, longer time intervals of a couple of minutes were selected). Recording was continued until water level variation become uniform.

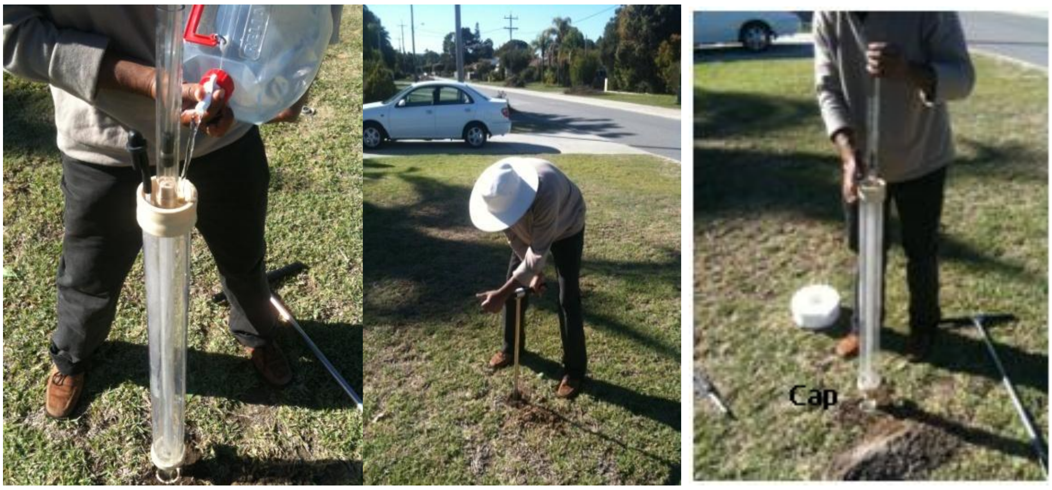


Figure 23. Field testing procedure using Guelph Permeameter

6.5. Potential water depths and flood inundation in stormwater sumps

Results for maximum water levels in the stormwater sumps for 5 year, 20 year, 50 year and 100 year rainfall scenarios are given in Table 12. Water level distribution maps for each rainfall scenario for each sub-catchment have been generated. Flood inundation results for sub-catchment SC004-035 in Catchment 04 for 5 year, 20 year, 50 year and 100 year ARI events are given in Figure 25 and the water depth is in metres. Similar figures demonstrating water level and inundation area were developed for all 13 sub-catchments. The stormwater sump #004 can be seen in the centre of the flooded area and sump #035 is located left of it in the figure.

Results show that some of the sumps are not capable of retaining runoff for any of the rainfall events. Some sumps are still capable of retaining 5 year and 20 year rainfalls, but are not adequate for greater events such as 50 year and 100 year ARI events. However, it was noticed that the sump top water levels are quite similar in some cases for 50 year and 100 year events. This happened due to localised flood inundation after exceeding the sumps' capacities. Therefore, in some sumps excess runoff caused an increase in the flooded surrounding area, rather than increasing the water depth. Results show that in some sumps top water level is exceeded at way over the maximum sump height. This happened when the surrounding area of the sump is still steeply graded down towards the sump. The infiltration rate varies from sump to sump and some sumps have higher infiltration rates which can empty the sump less than three days. The usual design criteria for sump emptying time in most of Western Australia's local government authorities is that it should be less than three days and then the sump should be ready for the next possible storm event. Also, the sumps are designed to cater for a 100 year ARI event volume. Therefore, the results concluded that the sumps that were not of adequate capacity should be enlarged, or the catchments should be provided with extra sumps. In addition, the water level elevation variations for sumps are given in Figure 24. Both sumps were inundated to about 1.25 m above the sump top water levels (TWL) for 100 year ARI events. They were not adequate in volume even to attenuate the 5 year ARI event runoff. This analytical assessment shows that both sumps are not sufficient for the current runoff demand, which could have been increased by the recent urban developments in the area. Maximum flood depths for 20-year, 50-year and 100 year ARI events were higher than the flood depth of the 5 year ARI event. The flood inundation area broadened when the intensity of the storm event was higher. Flood inundation maps show some water clusters remained in areas other than the sumps. This happened because of the coarseness of the topography data which were used to generate the DTM. However, considering their low water depths, they were not contributing significantly to the water levels of the sumps.

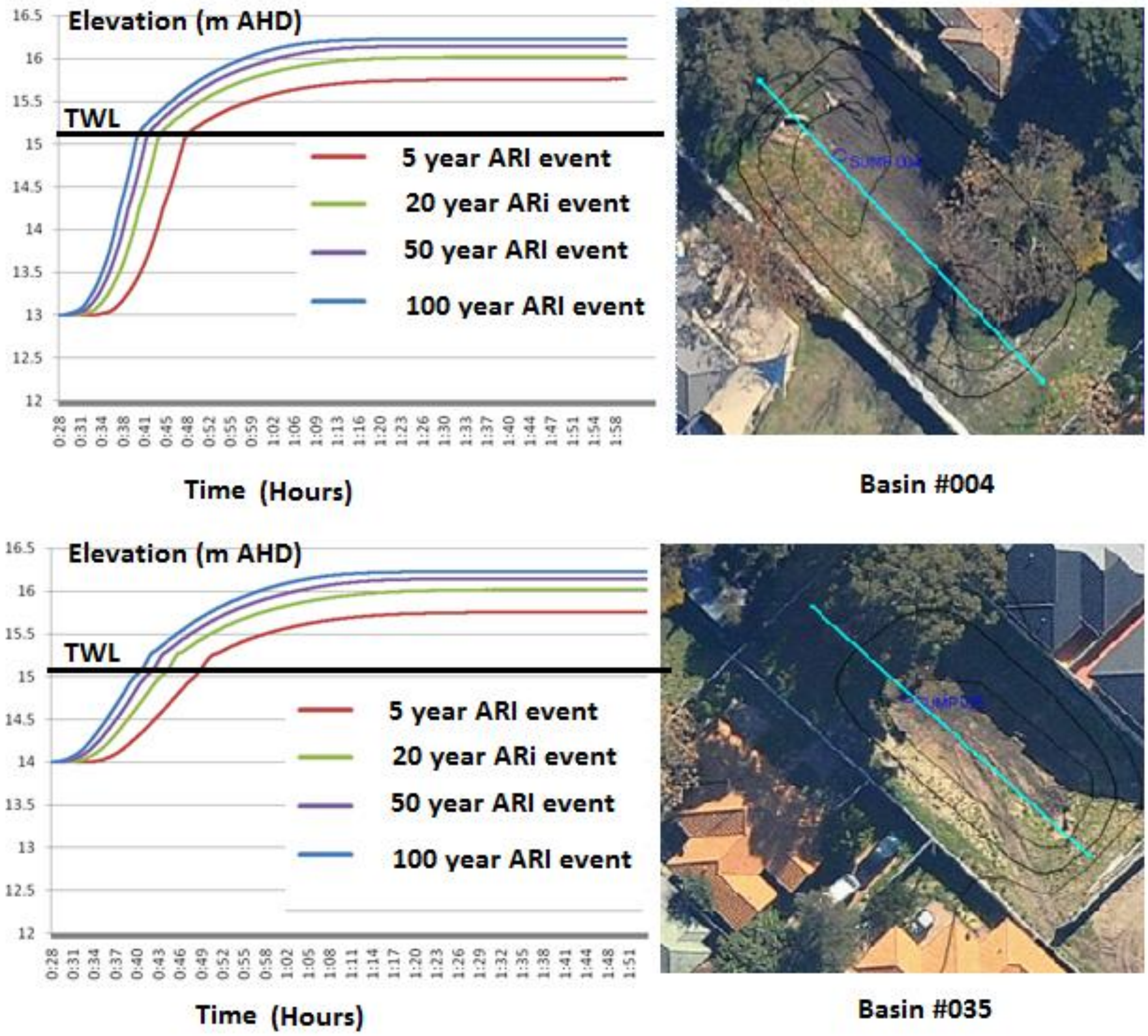


Figure 24. Water level variation at stormwater sump #004 and sump #035 for major rainfall events

Table 12. Maximum water levels of the stormwater sumps for major rainfall events.

SUMP	SUBURB	STREET NAME	AV. TIME TO PEAK (min)	SUMP TWL (mAHD)	MODEL RESULTS - TWL (mAHD)				INFILTR-ATION RATE (m/min)	TIME TO EMPTYING (Days)	AFTER INFILTRATION RATE IS APPLIED – TWL (m AHD)			
					5 YR	20 YR	50 YR	100 YR			5 YR	20 YR	50 YR	100 YR
S004	EVP	61 Camberwell Street	73	15.00	15.75	16.00	16.15	16.25	0.0030	3.73	15.53	15.78	15.93	16.03
S005	EVP	76 Canterbury Terrace	56	17.20	17.75	18.00	18.00	18.10	0.0045	2.75	17.50	17.75	17.75	17.85
S009	VP	59 Manchester Street	53	19.15	19.60	19.75	19.75	19.80	0.0061	2.21	19.28	19.43	19.43	19.48
S015	BW	16 Stiles Avenue	68	11.80	11.82	12.19	12.30	12.40	0.0052	1.62	11.47	11.84	11.95	12.05
S016	LT	34 Goddard Street	110	15.50	13.50	13.90	14.46	14.75	0.0002	47.50	13.48	13.88	14.44	14.73
S017	CL	26 Raleigh Street	105	13.40	13.41	13.62	13.80	13.82	0.0008	11.84	13.33	13.54	13.72	13.74
S023	CL	140 Mars Street	45	16.90	16.59	16.80	17.00	17.02	0.0000	N/A	16.59	16.80	17.00	17.02
S029	EVP	47 Dane Street	52	20.98	20.68	20.88	21.00	21.10	0.0031	4.76	20.52	20.72	20.84	20.94
S032	EVP	3 Swansea Street	57	14.15	14.20	14.50	14.70	15.00	0.0033	3.15	14.01	14.31	14.51	14.81
S034	StJ	7 Blechynden Street	51	13.30	13.49	13.51	13.51	13.52	0.0018	5.23	13.40	13.42	13.42	13.43
S035	EVP	61 Camberwell Street	68	15.00	15.75	16.00	16.15	16.25	0.0030	3.73	15.55	15.80	15.95	16.05
S03A	LT	51	53	16.00	13.5	14.52	14.56	15.10	0.0030	3.41	13.3	14.36	14.40	14.94

SUMP	SUBURB	STREET NAME	AV. TIME TO PEAK (min)	SUMP TWL (mAHD)	MODEL RESULTS - TWL (mAHD)				INFILTR-ATION RATE (m/min)	TIME TO EMPTYING (Days)	AFTER INFILTRATION RATE IS APPLIED – TWL (m AHD)			
					5 YR	20 YR	50 YR	100 YR			5 YR	20 YR	50 YR	100 YR
		Cornwall Street			5						9			
S045	LT	22 Gallipoli Street	35	20.00	20.10	20.20	20.20	20.20	0.0034	4.09	19.98	20.08	20.08	20.08
S058	CL	53 Solar Way	94	18.02	18.20	18.30	18.40	18.50	0.0113	1.07	17.14	17.24	17.34	17.44
S062	CL	195 Planet Street	116	16.00	15.48	15.51	15.55	15.56	0.0034	3.11	15.09	15.12	15.16	15.17
S063	CL	8 Lion Street	73	15.00	14.53	14.57	15.00	15.00	0.0008	12.88	14.47	14.51	14.94	14.94
S066	EVP	1 Patricia Street	52	14.20	14.58	15.10	15.20	15.25	0.0017	6.34	14.49	15.01	15.11	15.16
S069	EVP	16 Creaton Street	52	15.00	15.80	15.90	16.10	16.20	0.0038	2.96	15.60	15.70	15.90	16.00
S072	EVP	146A Sussex Street	102	10.00	11.70	12.00	12.00	12.00	0.0047	1.71	11.22	11.52	11.52	11.52
S073	EVP	11 Esperance Street	72	20.10	20.20	20.50	20.51	20.52	0.0059	2.36	19.77	20.07	20.08	20.09
S078	VP	21 Lichfield Street	62	15.20	14.30	15.00	15.25	15.28	0.0014	7.41	14.21	14.91	15.16	15.19
S080	VP	6 Sunbury Road	55	20.20	19.20	19.80	20.00	20.10	0.0075	1.84	18.79	19.39	19.59	19.69
S083	EVP	359 Berwick Street	56	20.30	20.20	20.30	20.40	20.60	0.0018	7.82	20.10	20.20	20.30	20.50
S087	StJ	25 Boundary	54	16.00	15.40	15.53	15.60	15.72	0.0050	2.16	15.13	15.26	15.33	15.45

SUMP	SUBURB	STREET NAME	AV. TIME TO PEAK (min)	SUMP TWL (mAHD)	MODEL RESULTS - TWL (mAHD)				INFILTR-ATION RATE (m/min)	TIME TO EMPTYING (Days)	AFTER INFILTRATION RATE IS APPLIED – TWL (m AHD)			
					5 YR	20 YR	50 YR	100 YR			5 YR	20 YR	50 YR	100 YR
		Road												
S090	StJ	119 Hillview Terrace	73	14.00	13.40	13.61	13.80	13.82	0.0051	1.82	13.02	13.23	13.42	13.44

Note: BW - Burswood, CL - Carlisle, EVP - East Victoria Park, LT - Lathlain , StJ - St James, VP - Victoria Park

Sump capacity is adequate

Sump capacity is not adequate

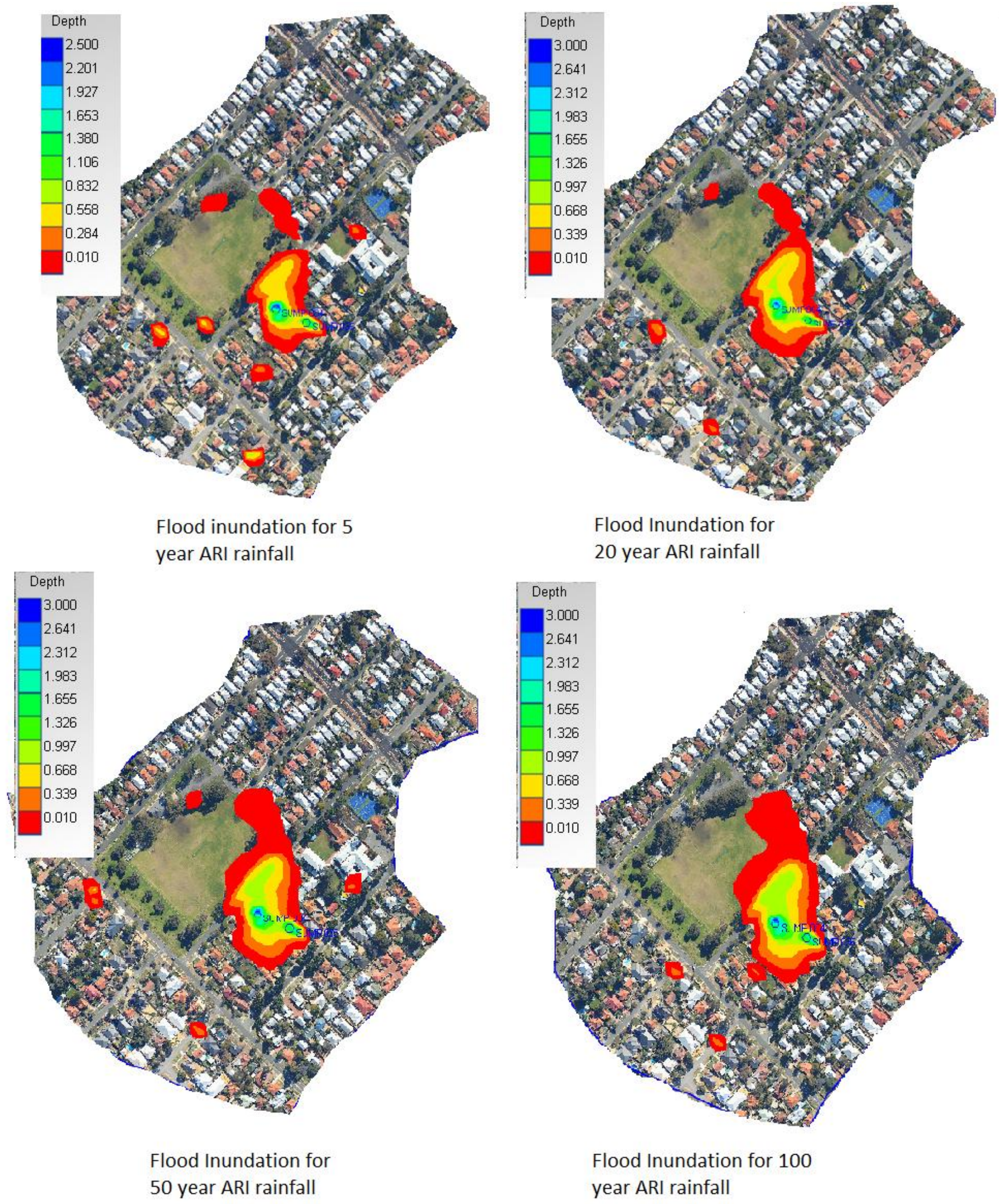


Figure 25. Flood distribution for 5 year, 20 year, 50 year and 100 year ARI event rainfalls for sub-catchment SC 004 – 035.

6.6. Conclusions

The case study was carried out to assess the stormwater sump capacities within Victoria Park catchment. About 98 sumps, together with their catchments, have been modelled by using 2D surface water routing numerical modelling techniques. The sump capacities against 5 year, 20 year, 50 year and 100 year ARI events have been assessed by modelling results for top water levels of the sumps for each rainfall event. Results of infiltration tests carried out for about 25 selected sumps were used to estimate the actual possible top water levels of the sumps after infiltration. The results show that there were some stormwater sumps that require additional capacity to facilitate the 100 year ARI event runoff volume. Some sumps were not even capable of catering for a 5 year ARI event runoff. The mud and debris that has collected over the years could be a reason for reduced sump volumes. Also, the increase in runoff generation by recent urban land use is another reason. The sump emptying time was calculated and the sumps with an emptying time of more than three days should be carefully treated to make them meet the level of local authority regulations. The flood maps were generated for the sumps and the catchments. They show the flood distribution in areas near the expelling sumps. These maps are helpful to analyse the flood potential of the catchment and to prevent hazardous situations. The sumps with a lack of capacity should be checked and appropriate measures should to be taken to protect surrounding residential areas from flooding.

CHAPTER 7

7. CASE STUDY OF ASSESSMENT OF WATER SENSITIVE URBAN DESIGNS

7.1. Introduction

Land development is demanded by the increasing population. Among several development guidelines such as those protecting the environment, there should be a strategy to direct land development which protects the natural hydrology of the developing catchment during and after development. This should describe the implementation of water sensitive urban designs (WSUD) within the development site. Therefore, several State Government policies and published guidelines and standards are available that provide direction regarding the water discharge characteristics that the urban development should aim to achieve. Preparation of a local water management strategy (LWMS) and an urban water management plan (UWMP) in the initial and development stages of a land development, which tend to describe how the development is going to follow those guidelines and policies which are mandated by the Western Australia Planning Council (WAPC 2008) is recommended. This case study was carried out to analyse the catchment hydrology obtained by a numerical modelling process to suggest the planning steps needed for an urban stormwater management strategy for an urban development site. The post-development hydrological modelling process was targeted to analyse the effects of land use change and the change of natural flow paths to urban stormwater management systems (USWMSs). It also analysed the mitigation of adverse effects of developments on catchment hydrology by using best management practices (BMPs) which are designed to cope with WSUD. Results of the study can be used to support the stormwater management section of an UWMP for development sites. The stormwater management section of the UWMP provides a full description of the approach to surface water management.

7.2. Methodology

The modelling work on which this study has based was carried out to support the stormwater management section of an UWMP. Only the stormwater management strategies related to quantity have been analysed during this case study.

- Literature review and understanding of government stormwater management policies and recommendations for the stormwater management of site development by local authorities.
- Data collection process (topographical data, geographical data, rainfall data, previous modelling results, proposed plans and drawings such as structural plans and earthwork together with landscape architectural plan).
- Identification of the stormwater management criteria which will guide the modelling process and which the final results should comply with.
- Analysis of previous pre-development modelling results (i.e. peak flow limitations)
- Analysis of design criteria of the local structure plan and landscape architectural drawings.
- Identification of the BMPs to be used.
- Post-development catchments and identification of their characteristics.
- Post-development 1D hydrology and hydraulic modelling. (BMPs were modelled in this stage by using several modelling assumptions and techniques.)
- The comparison of results with LWMS results and processing of results to comply with the stormwater management criteria.

7.2.1. Available data and data collection

- Historical rainfall data used to generate ARI events hydrographs (BoM 2012).
- Pre-development peak flows and modelling results from LWMS (Cardno 2010).
- Landscape architectural drawings and development structure plan.
- Guidelines from Western Australia Planning Council (WAPC 2006a, 2006b, 2007, 2008, 2009) and Town of Kwinana (ToK 2005).

- 0.2 m interval earthwork maps for post-development and 1 m interval pre-development contour maps.

7.3. Background of the study

This case study aims to address the development criteria at Wellard East for residential land development. The site is currently zoned as 'Urban' under the Metropolitan Region Scheme (WAPC 2009). Its development strategies followed the Town Planning Scheme #2 of the Town of Kwinana (ToK 2005). The case study was carried out to support the hydrological modelling process required to support the stormwater management system of the UWMP and it analysed the effects of land use change, several BMPs and USWMS.

Previous studies of the Jandakot district water management plan (DWMP) were carried out to fulfil the Western Australian planning policies by presenting a guide for developers and stakeholders within the area (DoW 2009). The Jandakot DWMP provided guidance on protection of environmental assets and groundwater management and implementation, other than the stormwater management within the Peel Main Drain catchment, which the site is located within. The key objectives related to stormwater management proposed in the Jandakot DWMP include:

- Protect wetlands and waterways from the impacts of urban runoff.
- Protect infrastructure and assets from flooding and inundation by:
    - Retaining and or detaining the 1 year 1 hour ARI event at source.
    - Maximising infiltration at source via soakwells, swales, sumps and other structures.
    - Using detention storage areas dispersed throughout urban areas to attenuate peak runoff rates.
    - Avoiding modification of existing channels unless it is to ensure continuation of flows.
    - Using revegetation and strategic channel stabilisation.

- Providing protection from 100 year ARI event levels by achieving 500 mm clearance for lot levels.

- Ensuring major arterial roads remain passable in a 100 year ARI event.

- Minimise changes to hydrology to prevent impacts on receiving environments by:

    - Maintaining post-development peak discharges to pre-development levels for the 1 year critical duration ARI event.

    - Managing catchment runoff such that the critical 10 year and 100 year ARI event peak flows are consistent with the pre-development peak flows.

    - Promoting Water Sensitive Urban Design (WSUD) and Best Management Practices (BMPs) which promote on-site retention of events up to the 1 year 1 hour ARI event.

The local water management strategy (LWMS) followed the DWMP in the preliminary stage of the site development proposal (Cardno 2010). It outlined the proposed stormwater management strategies to achieve compliance with DWMP strategies within the development site and summarised these as:

- Retain the 1 year 1 hour ARI event at source or as close as practicable.

- The post-development critical 5 year and 100 year ARI event peak flows and volumes shall be generally consistent with the pre-development environment at the discharge points into waterways and at the discharge points from each sub-catchment.

- Design the pipe network to cater for the 5 year ARI event.

- Ensure that the 100 year ARI event conveyance can be contained within road reserves.

- Development areas along the Peel Sub N Drain will have a finished floor level with a minimum of 500 mm clearance above the 100 year ARI event flood level described in the Jandakot DWMP.

- The invert of flood storage areas should have a minimum clearance of 300 mm from the CGL.

- Apply appropriate structural and non-structural measures to reduce applied nutrient loads.

- Bio-pockets will have a maximum water depth no greater than approximately 300 mm.

7.3.1. **Catchment description**

The Wellard catchment within Peel Main Drain catchment is located approximately 35 km south of the Perth CBD adjacent to the Kwinana Freeway and consists of approximately 38 ha. Total catchment size within the post-development structural plan boundary was 26.9 ha. A significant portion of land along the eastern and north eastern boundaries is dedicated to an easement for Western Power transmission lines. Some minor portions of the easement are proposed to be used for major flood event mitigation storage. A natural channel called Peel Sub North (N) Drain runs along the south eastern boundary of the site and a conservation category wetland is located at the north eastern boundary of the site. Surface water flows from the north and east of the site will discharge to the Peel Sub North (N) Drain, which discharges to the Peel Main Drain to the west of the Kwinana Freeway. The natural vegetation condition of the site currently consists of open paddocks and some remnant bushland. An aerial photograph illustrating the current condition and sub-division cadastral boundaries of the site is provided in Figure 26.

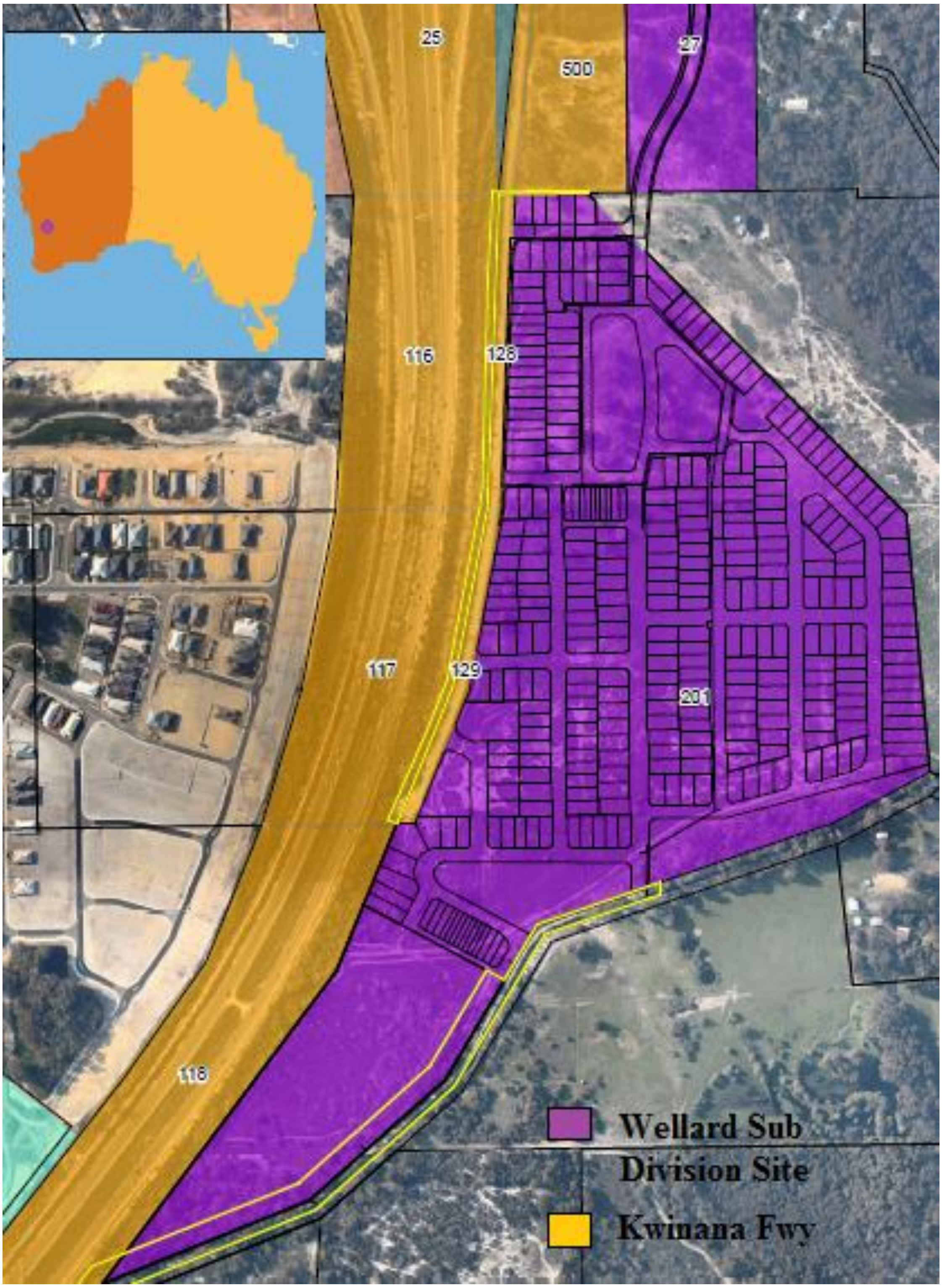


Figure 26. Wellard Residential land development site

Topography and Soil

The site is generally undulating and slopes gently from an elevation of 21 m AHD near the centre of the site close to the western border to an elevation of 9 m AHD within the southern portion of site. The portion within the site covering the northern low elevation land ranges in elevation from 13 m AHD to 10 m AHD adjacent to the Peel Sub N Drain (Emerge 2012). Gozzard (1983) indicates that the development site predominantly consists of Bassendean Sand (212Bs). Bassendean Sand is described as very light grey at the surface, yellow at depth, fine to medium-grained, sub-rounded quartz, moderately well sorted and of Eolian origin. The permeability of the Bassendean Sand is classified as high (Gozzard 1983).

Climate

The site experiences a dry Mediterranean climate of hot dry summers and cool wet winters. Long term climatic averages indicate an average of 745 mm annually (BoM 2012), predominantly received between June and August, occurring over 82 days per year. High evaporation rates and temperatures throughout the summer months drastically reduce flow within the Peel Branch Drain (DoW 2009).

Post-Development Catchment Characteristics

The proposed subdivision structural plan for the area has a mixture of residential housing densities and POS areas. The residential lots consist of 328 low density and medium density dwelling types which range from sizes of 160 m^2 to 2313 m^2 with an average lot size of 400 m^2. A mixture of road and laneways are proposed to be used within the development with 15 m wide reserves utilised for most of the development. Medium density lots were assumed to have 85 per cent impermeable area and low density lots 75 per cent impermeable area. Roads with 58 per cent impervious surfaces and public open spaces (POSs) having 100% permeability was the assumed land use for rest of the land. Ten per cent of the total land being developed has been kept for public open spaces (POS) according to the guidelines provided by WAPC (2009). Retention sumps for large ARI events have been located in these POSs.

7.4. Stormwater management criteria

The LWMS carried out hydrological and hydraulic modelling to determine pre-development peak flows leaving the site (Cardno 2010). The model was calibrated using the modelling results provided within the Jandakot DWMP (DoW 2009). An 'initial loss - proportional loss' infiltration method was adopted to generate stormwater runoff hydrographs in the XPSWMM model for the LWMS (Cardno

2010). This modelling work aimed to confirm that the post-development flows leaving the site are consistent with the pre-development flows provided in the LWMS. Therefore pre-development modelling parameters and peak flows were examined from previous studies of DWMP and LWMS. The post-development catchments and their characteristics were identified and matched with catchments described in the LWMS. The XPSWMM numerical model was created to represent the proposed post-development catchment and its hydrology. BMPs were selected to comply with stormwater management guidelines and represented in the model. The model was run for the 1 year 1 hour ARI event and critical duration rainfall events of 5 year and 100 year ARI events to assess the BMPs, inclusive of bio-pockets, retention and detention sumps design parameters and their compliance to the WAPC policies. The 100 year ARI retention sumps and their outlets were adjusted to match the pre and post-development peak flows. The effects of implementation of WSUDs and BMPs in urban developments were ultimately analysed. The design criteria for surface water management which the modelling work was based on and carried out to find are listed in Table 13.

Table 13. Stormwater management guideline criteria (Emerge 2012).

CRITERIA NUMBER	CRITERIA DESCRIPTION	MANNER IN WHICH COMPLIANCE HAS BEEN ACHIEVED	RESPONSIBILITY FOR IMPLEMENTATION	WHEN IMPLEMENTED
SW1	Retain the 1 year 1 hour ARI event at source or as close as practicable	Lot scale runoff from 1 year 1 hour ARI event retained within lot soakwells	Proponent	Detailed civil design
			Civil contractor	During civil works
			Lot owner	During house construction
		Road and verge runoff stored within sub-surface storage cells, swales and bio-retention sumps	Proponent	Detailed civil design
			Civil contractor	During civil works
SW2	Post-development 5	Storage sumps designed to detain	Proponent	Detailed civil design

	year and 100 year ARI event peak flows shall be generally consistent with pre-development environment at discharge points into waterways	runoff up to the 100 year ARI event with weirs designed to produce peak flows to be consistent with pre-development flows	Civil contractor	During civil works
SW3	Central spine roads shall be designed to convey the 5 year ARI event within the concrete pipe network	The pipe network has been designed to convey the 5 year ARI event within the spine roads of the development	Proponent	Detailed civil design
			Civil contractor	During civil works
SW4	All road reserves will be adequately sized to convey the 100 year ARI event within the road reserve	Road reserves have been designed to convey 100 year ARI event flows	Proponent	Detailed civil design
			Civil contractor	During civil works
SW5	Finished floor levels shall have at least 500 mm clearance to the 100 year ARI event flood level within flood storage areas (FSA) and the Peel N Sub drain	Detailed drainage designs confirm that lot floor levels have a minimum 500 mm clearance from top water level (TWL) within adjacent infiltration sumps and TWL within the Peel North (N) Sub Drain	Proponent	Detailed civil design
			Civil contractor	During civil works
SW6	Bio retention system, FSAs and drainage inverts should have minimum clearance of 300mm from commenced ground level (CGL)	Detailed design drawings confirm that sumps have a minimum clearance of 300 mm from CGL	Proponent	Detailed civil design
			Civil contractor	During civil works
SW7	Bio retention areas will have maximum water depth of 300 mm	Bio retention sumps designed with maximum depth of 300 mm	Proponent	Detailed civil design
			Civil contractor	During civil works
SW8	Bio retention areas will be	Connected impervious area of	Proponent	Detailed civil design

	sized to (at least) 2% of the connected impervious area	3.14 ha, bio retention area of 0.31 ha giving 9% connected road pavement area	Civil contractor	During civil works
SW9	Appropriate structural and non-structural measures shall be applied to reduce nutrient loads infiltrating to groundwater	Stormwater system maintenance	Maintenance contractor	Ongoing following construction
		Provision of educational material to residents at point of sale	Proponent	Point of sale
		Street sweeping to reduce particulate and sediment loads	Maintenance contractor	Ongoing post construction
		Waterwise garden practices to be implemented	Landscape contractor	Ongoing post construction
		Initial application of nutrients restricted to manufacturers recommendations	Landscape contractor	Landscape implementation
		Ongoing application of nutrients based on leaf tissue analysis	Landscape contractor	Ongoing post construction
		Ongoing groundwater monitoring to inform POS management	Proponent	Ongoing post construction

7.5. Pre and post-development modelling

The LWMS completed pre-development modelling and calibrated results for the Jandakot DWMP modelling. The UWMP has used the LWMS modelling results, adjusted to be comparable to the UWMP sub-division area. Table 14 gives the parameters used within the pre-development model (Cardno 2010). The proportional loss model had been used for the most of the catchments in the pre-development model to comply with the DWMP modelling. There were several catchments across site and the model was created according to the assumed proposed development earthworks.

Table 14. Pre-development catchment characteristics

LAND TYPE	INITIAL LOSS (mm)	PROPORTIONAL LOSS	ABSOLUTE LOSS (mm/hr)	ROUGHNESS
Sand-Sparsely-Vegetated	15	0.8	N/A	0.2
Sand-Medium-Vegetated	25	0.9	N/A	0.3
Wetland-Dry	25	0.1	N/A	0.2
Wetland-Wet	2	N/A	0.5	0.2

XPSWMM was used as the numerical model in post-development modelling. The surface routing was done by using the Laurenson Method in the XPSWMM hydrology layer. Catchments were divided according to the post-development earthwork and structural plan orientation. Initial and continual losses were used to count the infiltration during the post-development modelling, instead of proportional loss which was used during the pre-development modelling. The post-development land use types and their characteristics assumed in the model are given in Table 15.

The assumptions used in the modelling comprise of road reserves assumed to consist of 40 per cent road pavement and 60 per cent road verge from total roads and lots assumed to consists of 50 per cent roof area which is impervious, 25 per cent other impervious areas and 25 per cent gardens out of the total lot area. Lots were designed to provide sufficient capacity to retain the 1 year ARI volume of roof and garden areas. This would be within modelled soakwells and the infiltration capacity of garden areas. Separate land uses from separate catchments were attached to separate nodes, which were routing as separate catchments. Loss by evapotranspiration was assumed to be negligible due to the shortness of the rainfall durations.

Table 15 Post-development land use and types and their characteristics

LAND TYPE	INFILTRATION RATES		MANNING'S NUMBER
	INITIAL LOSS (mm)	CONTINUING LOSS (mm/h)	
POS	17.5	2	0.05
Road Pavement (40%)	1	0.1	0.014
Road Verges (60%)	5	1	0.05
Total Road (100%)			
Roofs (50%)	1	0.1	0.02
Lot Garden (25%)	17.5	2	0.05
Lot Paved (25%)	17.5	2	0.05
Total Lot (100%)			

The links with cross-sections of open rectangular channels with a length of 10 m, 5 m wide, 0.5 m deep, Manning's roughness of 0.014 and 5 per cent slope were used to link the catchment nodes. All the catchment losses and lag times of surface routing flow paths were considered in the hydrology component. Therefore links in the hydraulic component were modeled to facilitate immediate routing between nodes. The 1 year lot storage areas were modeled as 1 m depth cylindrical tanks, to represent the series of soakwells. Links connecting the catchments were modeled using their exact length as they are in the structural plan along the roads. The same channel profile was used to match the modeling links to actual flow paths through the roads and pipe drainage. The 1 year ARI roadside bio-retention storage areas and 100 year ARI retention storage areas were modeled by giving rating curves of storage surface area against depth. Bio-retention and retention storage areas were assumed to be rectangular sumps and given 1:6 side slopes and 0.5 m and 1.2 m subsequent depths. Storage capacity to retain 5 year ARI runoff was provided within the 100 year ARI retention sumps. Infiltrations from storage areas were given by a rating curve defined by flow (infiltration) against water depth of the storage area. Two different methods were used to calculate the infiltration flows in soakwells and retention sumps, considering the infiltration area of each type. Soakwells were assumed to have horizontal infiltration through their vertical surfaces as well as the bottom infiltration. Retention sumps were assumed to have equal infiltration through the bottom and side slopes. Soil permeability was assumed to be 2.5×10^{-5} m/s in designing the infiltration bio–retention areas and retention sumps. Rainfall scenarios were selected as a 1 year 1 hour event, and 5 year and 100 year critical duration events. The ARI rainfall data from BoM (2012) was extracted and applied to the hydrograph templates to create rainfall input data under each scenario.

7.5.1. Use of stormwater best management practices (BMPs)

There were number of BMPs used to make sure the land development complied with the WSUD and the guidelines of the local authorities. Rainfall runoff on the front and backyards of lots (garden areas) was modeled to either infiltrate directly at-source or, in larger rainfall events (i.e. > 1 year 1 hour ARI event), it was assumed that a portion of the runoff may discharge to the road network. The runoff from roof areas was directed to soakwells, which will infiltrate into the sandy soil and ultimately the groundwater. The soil of the area has a good infiltration rate and can facilitate the expected soakage. During modeling, the stormwater runoff from the 1 year 1 hour ARI rainfall event was retained as close to source as practicably possible; only rainfall events greater than this event were allowed to discharge from the source area. The retention storage within the model was provided through a treatment train which included soakwells, sub-surface storage cells and vegetated retention areas (located either immediately adjacent to road pavement or within downstream POS areas). The vegetation and the infiltration processes within the soil column were expected to remove a large portion of the contaminants (nutrients, gross pollutants, suspended sediments, etc.) contained within the stormwater runoff. Bio-retention areas were modeled as offline storage areas. Rainfall events greater than the 1 year 1 hour ARI event were modeled to bypass the infiltration or bio-retention areas and conveyed by overland flow or the concrete pipe network to end-of-catchment retention storage areas. Another type of 1 year 1 hour attenuating method used was the swales. They were modeled to provide both conveyance of stormwater and retention/detention storage. It was proposed to utilize swales within road reserve adjacent to POS areas. Stormwater would be directed into the swale via flush kerbing or the concrete pipe network. The swales were modeled in the same way as the bio–retention areas and were approximately 300 mm deep and 4 m wide. The use of swales provided a large surface area for the stormwater to infiltrate into the underlying sandy soil. Swales ensure that the 1 year 1 hour ARI rainfall event is retained at or near the source. For larger rainfall events, swales were used to convey or divert the runoff into the nearest end-of-catchment retention storage areas.

The end-of-catchment retention and detention sumps, also named as flood storage areas (FSA), were designed and modeled to detain the large event runoff (up to a 100 year ARI event) so that the peak discharge was comparable to the pre-development discharge rate. FSAs were not designed to be permanently wet. The size of the detention storage area could be minimized due to the retention storage provided higher up in the catchment. All discharge from the FSA was directed towards the existing Peel Sub N Drain via overland flow/discharge weir.

7.6. Results and discussion

The post-development sub-catchments USWMS for the development (including locations of sub-surface storage, bio-retention areas and FSAs) is provided within Figure 27. The 1 year 1 hour ARI event was retained within lots via soakwells and the infiltration capacity of open spaces (i.e. gardens) of the lots. Runoff from road reserves was retained within sub-surface storage cells and roadside soakage pits for minor subdivision roads and within bio-retention storage areas and roadside swales for major roads. The height of cylindrical lot soakwells was 1 m and their bottom surface area was 2.7 m^2. The height of roadside soakage pits was 2.4 m and their volume was 5.13 m^3. Sub-surface cells were used when there was not enough space to facilitate roadside soakage pits and they were 0.88 m^3 in volume. These combinations of retention storage areas retained the runoff from a 1 year 1 hour ARI event fully on site, to satisfy the stormwater management Criteria SW1. The storage volumes required within each storage area type are given in Table 16.

Table 16. 1 year 1 hour ARI event's storage design criteria

SUB CATCHMENT	LOT SOAKWELL STORAGE VOLUME (m^3)	ROADSIDE SOAKAGE				ROADSIDE SWALES
		TOTAL INFILTRATION SURFACE AREA (m^2)	TOTAL DESIGN VOLUME (m^3)	NUMBER OF ROADSIDE SOAKAGE PITS	NUMBER OF SUB SURFACE CELLS	TOTAL DESIGN VOLUME (m^3)
C 1	99.72	12.76	6.09	1	3	170.7
C 2	97.6	10.25	6.09	1	2	139
C 3.1	234.9	69.91	55.76	10	7	107
C 3.2	72.2	20.72	16.27	3	2	
C 3.3	68.3	20.97	19.9	4	0	
C 3.4	13.3	0	0	0	0	
C 4.1	214.8	15.49	11.61	2	2	183
C 4.2	70.3	20.94	17.93	4	0	64
C 4.3	52.5					62.5
C 4.4	16.7					15.7
C 4.5	35.2					
Total	975.22	171.01	131.65	25	16	741.9

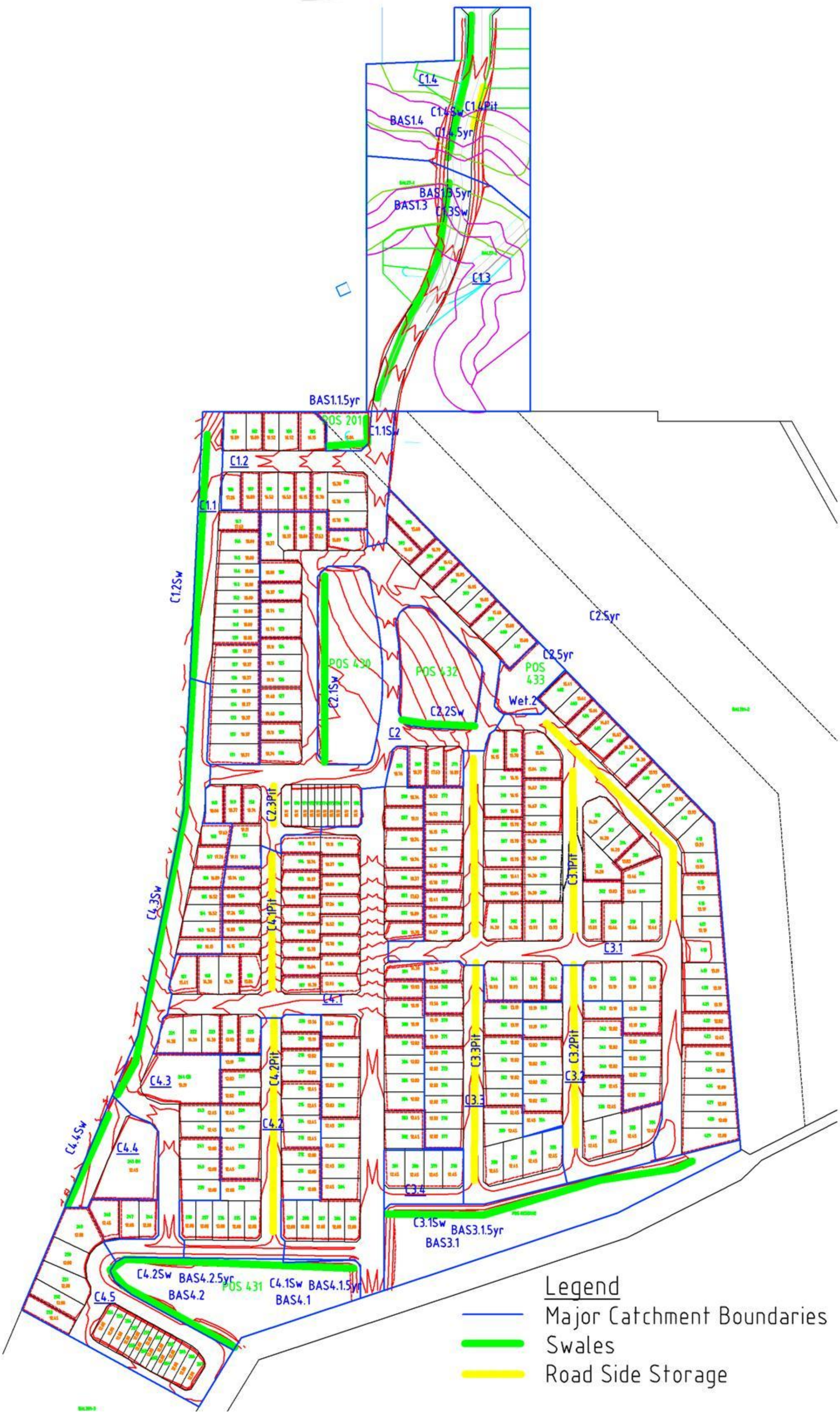


Figure 27. Post-development catchment boundaries and stormwater storage locations

The pre-development catchments documented in the LWMS were based upon the post-development earthwork and cadastral boundaries in the LWMS stage (Cardno 2010). The subdivision design and earthwork have been modified in the UWMP stage. Therefore the surface runoff sub-catchments used in this modelling were different from the sub-catchments at the LWMS stage. To avoid this discrepancy, equivalent catchment area peak flow analysis was used to make a pre-development to post-development peak flow comparison. The method considered the current catchment boundaries and took the percentage area of previous catchments within a particular current catchment boundary. Peak flows of previous catchments, according to their area percentages, were matched with current peak flows. Table 17 below shows the comparison of pre and post-development peak flows for the 5 year and 100 year ARI events. Both events show that peak flows within the post-development environment are generally consistent with the pre-development environment therefore Criteria SW2 has been satisfied. The WSUD concepts of retaining the 1 year ARI runoff on-site and using end-of-catchment retention sumps have reduced the peak flow rates downstream. As a result, the 100 year total post-development peak flow is even lower than the total pre-development peak flow. This can also be emphasised as protecting the pre-development hydrology of the catchment.

Table 17. 5 year and 100 year ARI event pre and post-development peak flow rate comparison

DISCHARGE POINT	**PRE DEVELOPMENT PEAK FLOW (m^3/s)**		**POST DEVELOPMENT PEAK FLOW (m^3/s)**	
	5 YEAR ARI EVENT	**100 YEAR ARI EVENT**	**5 YEAR ARI EVENT**	**100 YEAR ARI EVENT**
Bas 3.5	0.018		0.001	
Bas 4.5	0.005		0.004	
Bas 2.5	0.028		0.055	
Bas 5.5	0.083		0.139	
Bas 6.5	0.045		0.091	
Bas 7.5	0.028		0.015	
Bas 1.100		0.043		0.004
Bas 4.100		0.009		0.045
Bas 2.100		0.051		0.063
Bas 5.100		0.146		0.124
Bas 6.100		0.079		0.07
Bas 7.100		0.047		0.021
Total	0.206	0.374	0.305	0.324

Modelling work was done and the above results were generated with the assumption that the concrete pipe network has been designed to convey the 5 year ARI event. On this basis, Criteria SW3, which states that major roads shall be designed to convey the 5 year ARI event within the concrete pipe network, is considered to have been satisfied. The 100 year ARI runoff flow paths along the road network shown in Figure 28 confirm that the road pavement network is adequately sized to convey the 100 year ARI event within the road reserve. On this basis, Criteria SW4 is considered to have been satisfied.

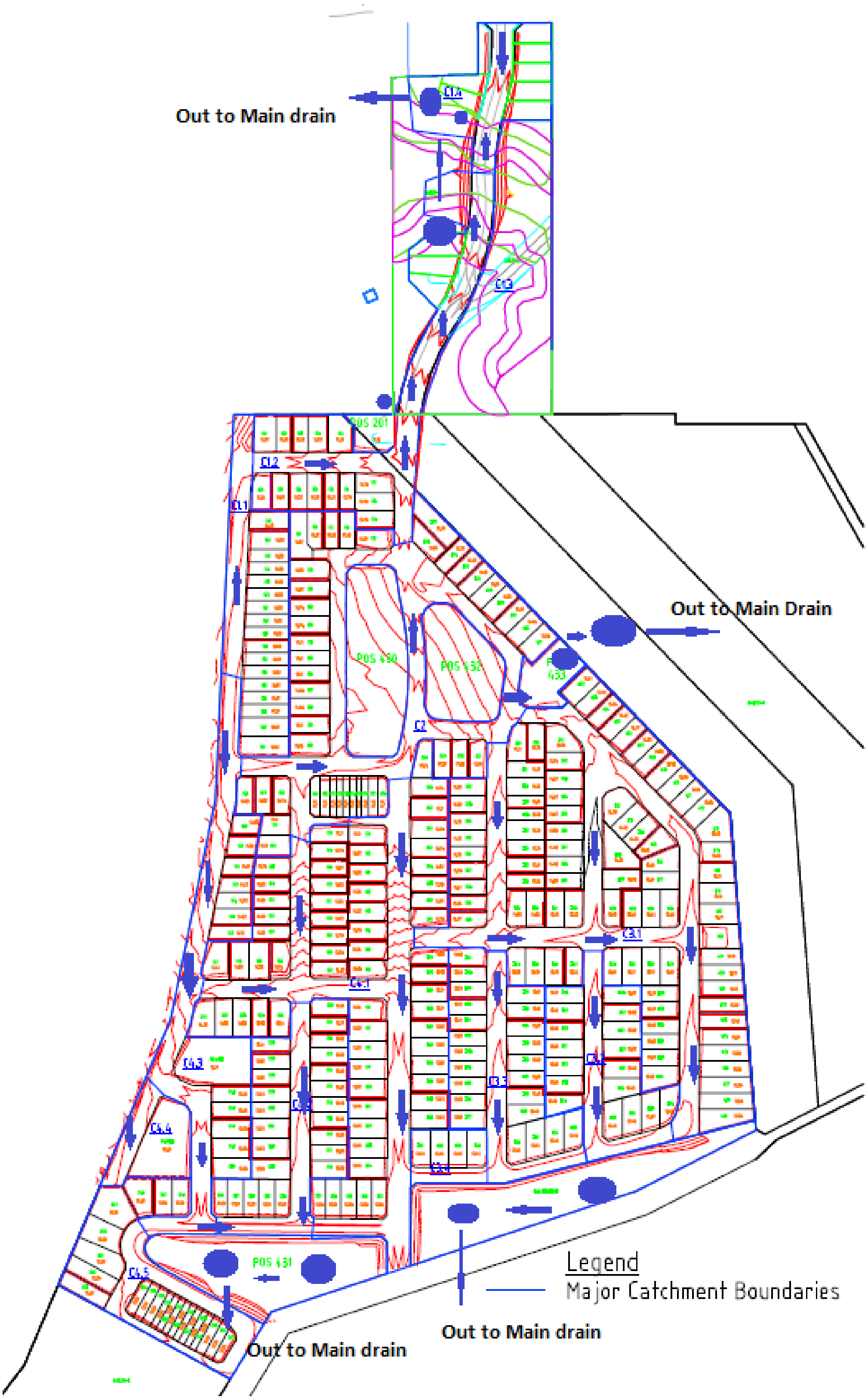


Figure 28. Post 100 year flow paths along the road network

The finished lot levels and the top water level within the adjacent infiltration sumps and the Peel Sub N Drain are detailed within Table 18. As shown in Table 20, a minimum clearance of 500 mm from the 100 year flood levels is provided and stormwater management Criteria SW5 is considered to have been satisfied. Some of the sumps were not yet designed with their exact topographies and they were noted in the table with maximum achievable top water levels to satisfy the stormwater management criteria. Table 18 provides a summary of the retention sumps and bio-retention areas' design details with their corresponding depths relative to the commence ground level (CGL). The results confirm that the modelling work has achieved stormwater management Criteria SW6 by having a minimum depth of 300 mm from TWL to CGL. The model used a maximum of 0.5 m depth for the bio-retention areas and swales and confirms that the bio-retention areas will have a maximum depth of 500 mm, therefore achieving stormwater management Criteria SW7. The connected impervious area, being the extent of road pavement which directs runoff to the stormwater network, is 3.14 ha. The surface area of roadside swales and bio-retention areas is 0.31 ha, which provides 9 per cent of the connected road pavement area. On this basis, Criteria SW8, which states that the bio-retention areas will be sized to (at least) 2 per cent of the connected impervious area, is considered to have been achieved. According to the stormwater management Criteria SW9, appropriate structural and non-structural measures shall be applied to reduce nutrient loads infiltrating to groundwater. The treatment train, which consists of lot and roadside soakage wells, bio-retention storage areas and swales, was assumed to achieve maximum nutrient removal, which ensures the Criteria SW9 is met by taking appropriate structural measures. Non-structural measures that were proposed to satisfy Criteria SW9 include:

- Stormwater system maintenance.
- Provision of educational material to residents at point of sale.
- Street sweeping to reduce particulate and sediment loads.
- 'Waterwise' garden practices.
- Initial application of nutrients restricted to manufacturer's recommendations.
- Ongoing application of nutrients based on leaf and tissue analysis.
- Ongoing monitoring to inform broader catchment management.

Table 18. 100 year ARI event top water level clearance from lot floor levels

SUMP	TWL (mAHD)	LOT FLOOR LEVEL (mAHD)	CLEARANCE (mm)
Bas 1.1	14.83	15.78	1930
Bas 1.100	14.28	15.68	1400
Bas 2.1	14.72	15.41	690
Bas 2.5	13.76	15.41	1650
Bas 2.100	11.87	15.41	3540
Bas 3.5	11.6	TWL + 500 mm*	500*
Bas 4.5	11.05	TWL + 500 mm*	500*
Bas 4.100	10.98	TWL + 500 mm*	500*
Bas 5.100	Yet to be designed	TWL + 500 mm*	500*
Bas 6.100	Yet to be designed	TWL + 500 mm*	500*
Bas 7.100	Yet to be designed	TWL + 500 mm*	500*
Peel Sub N Drain	9.07	9.57	500*

*Minimum required TWL.

The structural BMPs used to retain the 1 year 1 hour ARI on site were taken assuming a total volume of 1849 m^3 exclusive volume discharged from the system by infiltration. Considering the total catchment area of 26.9 ha, a 68.8 m^3/ha volume was demanded by the urban development. Considering the 1 year 1 hour rainfall intensity of 15.7 mm/hr, the total 1 year event rainfall was 4192 m^3. This shows about 44 per cent of the volume has been attenuated by the 1 year BMP storage areas. Considering the pervious areas and their infiltration capacities, the total infiltration through the bare land during the 1 year 1 hour ARI event was 1698 m^3. This shows again there was 645 m^3 infiltration taken off by BMP structures. Therefore the total 1 year 1 hour ARI runoff volume attenuated or infiltrated via the 1 year storage areas was 2493 m^3 and it was 59.5 per cent of the total runoff. When the infiltration capacities are higher in the subdivision, there will be less demand on the 1 year storage areas since infiltration will achieve a significant volume reduction.

Total required capacity of the 5 year and 100 year end-of-catchment retention sumps was 6088.9 m^3. Considering the 1 year total storage volume of 2493 m^3, which is 41 per cent of major event required sumps volume. Also there is a lag time in the runoff flow by having the 1 year treatment train which ultimately causes to reduce the peak flow rates at the end of catchment and the volume required in the retention sumps. Therefore it can be considered that 1 year retention storage areas can be used as a BMP that can effectively contribute not only as a stormwater quality improving technique, but also to reduce stormwater quantities and peak flow rates.

Table 19. Bio-retention area and FSA characteristics and clearance to CGL

BIO-RETENTION/FLOOD STORAGE AREA	CATCHMENT	BOTTOM SURFACE AREA (m^2)	VOLUME (m^3)	INVERT (mAHD)	TWL (mAHD)	WEIR INVERT (mAHD)	CGL ELEVATION (mAHD)	DEPTH TO CGL (m)
Bas 1.1	C1.1, C1.2	11.2	20.9	14.33	14.85	n/a	10.40	4.00
Bas 3.1	C1.3	117	92	11.05	11.53	n/a	10.35	0.70
Bas 3.5	C1.3	216	374	10.70	11.60	n/a	10.35	0.35
Bas 4.5	C1.4	161.6	101.9	10.65	11.05	10.67	10.30	0.35
Bas 2.1	C2.1, C2.2	16.3	44.5	14.25	14.72	n/a	10.00	4.25
Bas 2.5	C2.1, C2.2	82	321	12.60	13.76	13.80	10.00	2.60
Bas 5.5	C3.1, C3.2, C3.3, C3.4	1650	951	9.90*	TBC	TBC	9.60	0.30*
Bas 6.5	C4.1, C4.2, C4.3	1600	923	9.90*	TBC	TBC	9.60	0.30*
Bas 7.5	C4.4	130	104	9.90*	TBC	TBC	9.60	0.30*
Bas 1.100	C1.1, C1.2	870	281	14.0	14.28	14.3	10.40	3.60
Bas 4.100	C1.4	400	159	10.65	10.98	n/a	10.30	0.35
Bas 2.100	C2.1, C2.2	1566	496	11.57	11.86	11.87	10.00	1.57
Bas 5.100	C3.1, C3.2, C3.3, C3.4	550	1142	9.90*	TBC	TBC	9.60	0.30
Bas 6.100	C4.1, C4.2, C4.3	500	1068	9.90*	TBC	TBC	9.60	0.30
Bas 7.100	C4.4	15	168	9.90*	TBC	TBC	9.60	0.30

* Minimum required depth

7.7. Conclusions

The Wellard residential urban land development site was assessed for its proposed stormwater management strategy by using XPSWMM stormwater management model. The stormwater management guidelines and government policies to achieve WSUD by implementing BMPs have been discussed. There is a mandatory process of preparing DWMS, LWMS and UWMP during land development within Western Australia. The stormwater management guidelines can be changed from site to site according to these strategies. The site had been followed up by DWMS and LWMS. This case study was done basically to facilitate the stormwater management strategy section of preparing the UWMP. It was modelled to satisfy several stormwater management criteria stated by local authorities and the Western Australian government.

The 1 year 1 hour ARI event runoff was attenuated on site to satisfy the stormwater management Criteria SW1 by using several BMPs such as lot soakwells, roadside soakage pits, sub-surface storage areas, bio-retention areas and roadside swales. They have attenuated 2493 m^3 volume, including infiltration via them. Pre-development peak flows were calculated by using the previous results from LWMS. Post-development peak flows after using BMPs were matched with the pre-development peak flows. The peak flows were adjusted by using several end-of-catchment stormwater retention sumps and appropriate weir heights. The total post-development peak flows out of the catchment for 5 year and 100 year ARI events were matched with the pre-development peak flows to an acceptable level. Therefore stormwater management Criteria SW2 was satisfied. The rest of the stormwater management criteria were satisfied by implementing appropriate modelling assumptions and taking suitable structural and non-structural measures. Several guidelines were satisfied by the modelling work and the stormwater management strategy, supported by the modelling, has suggested that the site development complies with stormwater management guidelines.

Finally, the WSUD guideline of retaining the 1 year 1 hour event runoff on site by using several treatment methods and storage areas can be used as an effective end-of-catchment peak flow and retention sump volume reduction method, other than its main purpose as a stormwater treatment option.

CHAPTER 8

8. CONCLUSIONS

This research aims to evaluate the effect of urban land use changes on urban hydrology. A literature review was carried out prior to the study to understand the adverse impacts of land use changes and other factors (e.g. shallow groundwater effect) on urban hydrology. The literature review provided guidance for the study by ensuring its scope and the methodology was appropriate. Also, the numerical modelling of urban hydrology and stormwater management systems within the models were studied. XPSWMM was selected as the modelling tool for the study, as a result of the literature review. XPSWMM is capable of modelling urban hydrology by coupling 1D hydraulic simulation (for channels and pipe flows) with 2D surface runoff simulations. Several relevant past research works, including applications of numerical modelling in urban hydrology, have been referred. Three case studies have been assessed to demonstrate different aspects of urban hydrology and stormwater management. Finally, urban stormwater best management practices have been reviewed under the concept of water sensitive urban designs.

Prior to the use of models as tools to analyse the hydrological catchment behaviour, sensitivity analysis is needed. Sensitivity analysis was carried out to decide the most suitable modelling approach to represent the urban hydrology numerically and to find the sensitivity of catchment characteristics. Two modelling approaches were compared during the sensitivity analysis: the hydrological surface runoff routing approach and the hydraulic surface runoff routing approach. Both methods have similar conditions to route 1D drainage flows and coupling of drainage excess water into the 2D layer. However, they differed from the surface runoff routing methods. The hydrological method used either the Laurenson method or the SWMM nonlinear runoff routing method with manually input catchment characteristics such as catchment area, slope, width, etc. The hydraulic method was used to rout the surface runoff by using 2D shallow water equations. The GIS data and topography data were used to generate the DTM, and to feed the input data to the model. The results of the two approaches were compared against observational data. The results show both methods were capable of representing urban hydrology. Therefore, both methods were combined to analyse urban hydrology during the study, depending on available catchment data and catchment behaviours. Sensitivity analysis was further carried out to find the effect of shallow groundwater on the urban drainage system. The results show that there is a significant effect from the groundwater on the catchment hydrology. Therefore it has been treated as a major catchment characteristic. The surface roughness, infiltration values, depression storage and percentage of zero detention, which were based on land use categories, were analysed. Results show that the sensitivity of surface roughness and infiltration values are significant, especially in minor rainfall events (i.e. a 1 year event). These parameters show considerable effect on the 100 year ARI peak flows as well. Therefore, land use change has been identified as a major impact on urban hydrology. Calibration and verification processes were carried out to validate the catchment models. Observational data at Avenues basin outlet in Canning Vale was used to calibrate the model. The calibrated model was validated using independent observational data.

The effect of urban land use change on urban hydrology was analysed using numerical modelling for three case studies: Canning Vale Central catchment drainage assessment, Victoria Park stormwater sump capacity assessment and Wellard catchment's water sensitive urban designs assessment. The case study of Canning Vale was carried out to analyse the effect of land use changes on the existing catchment hydrology. The selected catchment has been urbanized rapidly with sub-division and new residential development processes. Natural bare lands have been converted to urban impervious surfaces. Therefore, the current stormwater management system is not capable of facilitating the runoff from storm events, which ultimately creates localised floods. The calibrated and verified models, with identified catchment characteristics values, were used to model the overall catchment. The upstream sub-catchments of Glenariff and Sanctuary Lake are contributing runoff to the Central Catchment during major rainfall events. The limitation for maximum inflow to the Central catchment from any of those upstream catchments was 1.13 L/s/ha. The Glenariff outflow was modelled as a controlled flow and kept to the limit. It shows that the proposed basin capacities for this sub-catchment are enough to limit the outflow to be under the permitted level. However, Sanctuary Lake showed a higher peak flow rate for the 100 year ARI event. This may be acceptable, as it is only for the 100 year event. The basin top water levels and peak outflows from the catchment were analysed. The results show that some of the basins are functioning as they were designed, while others are outdated. Groundwater base-flow through a sub-soil pipe with insufficient diameter has been found to be the major reason that localised floods occur upstream. The areas with higher flood risks were marked in the flood inundation maps. Overall study shows that there is a significant impact from urbanization and land use change to the urban hydrology. The increment of flow rate and volume is identified by comparing the current rates and quantities of stormwater to designed capacities of the stormwater drainage. The future scenario of proposed developments were analysed thereafter. Study identified these proposed developments will increase the flood depths and flood vulnerability. Especially further development along the main MUC will cause flood inundation at downstream POSs. Also it will increase the flood risk to the properties along the MUC. The groundwater effect has been identified as a major component of the urban hydrology in shallow groundwater catchments. Study shows that groundwater should be treated with considerable attention along with the urban surface water modelling when it affects to the infiltration rates and linked with the underground drainage.

The Victoria Park case study was carried out to assess the capacities of stormwater sumps within the Victoria Park catchment. The Town of Victoria Park has been grown rapidly over past decades. Land use has been changed from pervious to impervious in most of the areas. About 98 sumps, together with their catchments, have been modelled by using 2D surface water routing numerical modelling techniques. The sump capacities, compared against standard storm events, were analysed. The results for maximum water depths of stormwater sumps for each rainfall event were coupled with infiltration capacities measured in the basins. The results of infiltration tests carried out for about 25 selected sumps were used to estimate the actual possible top water levels of the sumps after infiltration. The results show that there were some stormwater sumps that require additional capacity to facilitate the 100 year ARI event runoff volume. Some sumps were not even capable of catering for minor storm event runoff. The results show the over-designed sumps and the under-capacity sumps. The mud and debris collected over the years could be a reason for reduced sump volumes. Also, the increase of runoff generation by recent urban land use change is the other major reason for the inadequacy of existing sump capacities. The sump emptying time was calculated and the sumps with an emptying time of more than 3 days should be carefully treated to make them meet a satisfactory condition, as set by local authority regulations. Flood maps were generated for the sumps and the catchments. These maps are helpful for analysing the flood potential of the catchment and for preventing hazardous situations. The sumps with a lack of capacity should be checked and appropriate measures should to be taken to protect surrounding residential areas from flooding. This case study gives a simple analytical method to assess the ungagged catchment by using available raw data. It shows the impact to the existing stormwater management facilities from rapid urbanization.

The Wellard residential urban land development site was assessed by using numerical modelling to analyse the effect of water sensitive urban design best management practices on catchment hydrology. The pre-development peak outflows and post-development peak outflows were matched for 5 year and 100 year rainfall events. The post-development peak runoff generation is higher than the pre-development scenario according to the model results, because of assumed post-development urban land uses. These land use changes have increased the impervious percentage of the subdivision, decreasing the infiltration values and surface roughness values. The excess volume of runoff was kept within the site, to comply with the stormwater management criteria in the government guidelines. A 1 year ARI event runoff was retained within lots using soakwells and the 1 year runoff from roads was kept in bio-retention basins, sub-soakage storage areas, soakage wells and swales. The model successfully represents the capacities of weirs and the end-of-line storage areas such as detention and retention basins used to control the 100 year ARI post-development runoff. The BMPs, including infiltration structures to infiltrate the 1 year ARI runoff and end-of-line structures, were modelled by considering the infiltration rates. The implementation of runoff retention basins with the capacity of a 1 year ARI runoff within the site were deployed as a stormwater quality protection guideline. However, results show a significant reduction of end-of-line peak flows and volumes due to the policy of retaining the 1 year storage areas during the major rainfall events. Therefore, the guideline of retaining the 1 year ARI within the catchment is not only a water quality assurance policy, but also a proper flow control mechanism. The effect is higher when the storm event is lower. Several stormwater management criteria, needed to satisfy the local government guidelines, were achieved by the modelling process. The use of BMPs as either a source control or end-of-line controls is an advantage in protecting the catchment hydrology and thereby keeping the natural balance. The case study provide the eva

The outcome of this study shows very useful results in evaluating land development on urban hydrology, including direct aspects of land developments, changes in catchment characteristics, the effectiveness of stormwater sumps and the engagement of WSUD in land development. The results will be useful for land developers, local city councils, authorities and policy and decision makers to guide sustainable land development practices to ensure minimum impacts on urban hydrology.

8.1. Recommendations

8.1.1. Recommendations from the results

- The 1D, 2D coupled model with the support of GIS, remote sensing and LiDAR data is recommended to use to analyse urban hydrology and urban stormwater management systems effectively.

- The outcome of the study clearly shows that land use changes and urbanization directly affect urban hydrology by increasing peak flows and runoff volumes. Therefore, the catchment characteristics that depend on land use changes should be treated with considering consideration of their sensitivity to the results.

- Rapid urbanization, with its accompanying land use changes increasing impervious areas, can cause localised flood inundations. Therefore, the low elevation areas in a catchment should always be treated with due care when there is a land development proposal.

- A shallow groundwater table is a significant factor in increasing the flood vulnerability in urban catchments, especially where the underground drainage is submerged. Therefore, the groundwater's effect on stormwater management should be considered in addition to the effect of land use changes in such catchments.

- The water quality based stormwater management guideline of retaining the 1 year ARI runoff within the catchment can be considered as a good quantity measure as well. Therefore, it is recommended that the guideline is followed during subdivision and land development works.

- The pre-development catchment hydrology should be protected at the post-development stage and the effect of increased impervious areas on runoff generation should be mitigated by the combination of BMPs for source control and end-of-line flow controlling.

8.1.2. **Recommendations for further studies**

- In this study, the mechanism used for groundwater mounding is a simple groundwater tool, which will not evaluate complex groundwater system. It is recommended that the groundwater effect be analysed with a groundwater specified modelling tool and then couple these results to the stormwater management model.

- Urban hydrology can change with changing climatic conditions as well. Therefore, it is highly recommended that the effects of climatic change be studied and then couple these climate change scenarios to the land use based catchment runoff model.

- An urban catchment with complex features and catchment characteristics should be modelled as a combination of 1D and 2D elements to get the best results. Also, GIS, LiDAR and remote-sensing data should be used with urban hydrological models. Such a model, together with proper groundwater mounding coupling methods integrated with climate change scenarios, will give a good prediction of the future urban hydrological behaviour of the catchments. However, it is recommended that the model is calibrated and verified by using observational or historical data.

- If related costs and benefits are available, a cost benefit analysis (economic assessment) of BMP in urban hydrology is recommended.

- More research on the positive impacts of groundwater abstraction during the winter to lower the groundwater level is highly recommended.

REFERENCES

1. Abebe, N. A., Ogden, F. L. and Pradhan N. R. (2010). Sensitivity and uncertainty analysis of the conceptual HBV rainfall–runoff model: Implications for parameter estimation. Journal of Hydrology, 389(3-4), 301-310.

2. Agarwal C., Green, G.M., Grove, J.M., Evans, T.P. and Schweik, C.M. (2002). A review and assessment of land use change models. dynamics of space time and human choice, CIPEC Collaborative Report Series No. 1, 84, USDA Forest Service Indiana.

3. Aggett, G. R. and Wilson J. P. (2009). Creating And Coupling A High-Resolution DTM With A 1-D Hydraulic Model In A GIS For Scenario-Based Assessment Of Avulsion Hazard In A Gravel-Bed River, Geomorphology 113 (1–2), 21-34.

4. Aguilar, F. J., Mills, J. P. (2010). Modelling Vertical Error In Lidar-Derived Digital Elevation Models, ISPRS Journal of Photogrammetry and Remote Sensing, 65 (1), 103-110.

5. Andre´assian, V. (2004). Waters and Forests: From Historical Controversy to Scientific Debate. Journal of Hydrology, 291, 1–27.

6. Augusto Pompêo, C. (1999). Development of a State Policy for Sustainable Urban Drainage. Urban Water 1, (2), 155-160.

7. Banaszuk, P. and Kamocki, A. (2008). Effects of climatic fluctuations and land-use changes on the hydrology of temperate fluviogenous mire. Ecological Engineering, 32(2), 133-146.

8. Barbosa, A. E., Fernandes, J. N. and David, L. M. (2012). Key Issues for Sustainable Urban Stormwater Management. Water Research (In Press).

9. Benzerra, A., Cherrared, M., Chocat, B., Cherqui, F. and Zekiouk, T. (2012). Decision Support for Sustainable Urban Drainage System Management: A Case Study of Jijel, Algeria. Journal of Environmental Management, 101, 46-53.

10. Berendrecht, W. L., Heemink, A. W., van Geer, F. C. and Gehrels, J. C. (2006). A non-linear state space approach to model groundwater fluctuations. Advances in Water Resources, 29(7), 959-973.

11. Bergkamp, G. and Orlando, B. (1999). Wetlands and Climate Change: Exploring Collaboration Between the Convention on Wetlands (Ramsar, Iran, 1971) and UN Framework Convention on Climate Change. IUCN, Gland, Switzerland.

12. Berthier, E., Andrieu, H. and Creutin, J. D. (2004). The role of soil in the generation of urban runoff: development and evaluation of a 2D model. Journal of Hydrology, 299(3-4), 252-266.

13. Best, A., Zhang, L., McMahon, T., Western, A. and Vertessy, R. (2003). A Critical Review of Paired Catchment Studies with Reference to Seasonal Flows and Climate Variability. Technical Report, CRC for Catchment Hydrology, 47.

14. Beven, K. J. (2001). Rainfall-Runoff Modelling: The Primer. United Kingdom: John Wiley & Sons Ltd.

15. BoM, (2012). Climate Data Online. Bureau of Meteorology, Government of Western Australia, http://www.bom.gov.au/climate/data/.

16. Boughton, W. (2005). Catchment water balance modelling in Australia 1960–2004, Agricultural Water Management, 71, 91–116.

17. Boyd, M. J., Rigby, E. H. and VanDrie, R. (1996). Wbnm - A Computer Software Package for Flood Hydrograph Studies. Environmental Software, 11 (1-3), 167-172.

18. Braune, M.J. and Wood, A. (1999). Best Management Practices Applied to Urban Runoff Quantity and Quality Control. Water Science and Technology, 39 (12), 117–121.

19. Brown, I. (2006). Modelling future landscape change on coastal floodplains using a rule-based GIS. Environmental Modelling &amp, Software 21 (10), 1479-1490.

20. Butler, D. and Parkinson, J. (1997). Towards Sustainable Urban Drainage. Water Science and Technology, 35 (9), 53-63.

21. Campana, N. A., and Tucci, C. E. M. (2001). Predicting Floods from Urban Development Scenarios: Case Study of the Dilúvio Basin, Porto Alegre, Brazil. Urban Water, 3 (1-2), 113-124.

22. Cardno. (2010). Lot 27 and Lot 201 Wellard East Local Water Management Strategy. Cardno Group of Companies, Perth.

23. Carlson, T. N. and Arthur, S.T. (2000). The Impact of Land Use - Land Cover Changes Due to Urbanization on Surface Microclimate and Hydrology: A Satellite Perspective. Global and Planetary Change, 25 (1–2), 49-65.

24. Chen, J., Hill, A.A. and Urbano, L. D. (2009). A GIS-Based Model for Urban Flood Inundation. Journal of Hydrology, 373 (1-2), 184-192.

25. Chow, V. T. (1959). Open-Channel Hydraulics, McGraw-Hill.

26. Chow, V. T., Maidment, D. R. and Mays, L. W. (1988). Applied Hydrology. International edition, McGraw-Hill, New York.

27. Christopher, Z. (2001). Review of Urban Stormwater Models. Environmental Modelling & Software, 16, 195–231.

28. Coombes, P. J., Argue, J.R. and Kuczera, G. (2000). Figtree Place: A Case Study in Water Sensitive Urban Development (WSUD). Urban Water, 1 (4), 335-343.

29. Cook, A. and Merwade V. (2009). Effect of topographic data, geometric configuration and modeling approach on flood inundation mapping, Journal of Hydrology, 377 (1–2), 131-142.

30.

31. Cordery, I. and Pilgrim, D. H. (1983). On the Lack of Dependence of Losses from Flood Runoff on Soil and Cover Characteristics. Hamburg Symposium, IAHS.

32. Coveney, S., Fotheringham, A. S. (2010). Dual-scale validation of a medium-resolution coastal DEM with terrestrial LiDAR DSM and GPS, Computers &amp, Geosciences, 36 (4), 489-499.

33. CSIRO Publishing, 2006, Urban Stormwater: best practice environmental management

34. DEC. (2011). Geomorphic Wetlands of the Swan Coastal Plain Dataset, Perth. Department of Environment and Conservation.

35. Dey, A. K. and Kamioka S. (2007). An Integrated Modelling Approach to Predict Flooding on Urban Basin. Water Science and Technology, 55(4), 19 -29.

36. Dey, D. A., Samaratunge, P. and Wazeer, A. (2010). Complexities in Overland Flow Patterns in an Urbanized Catchment - a 1D/2D Integrated Modelling Case Study, City of Gleneira. 50th Annual Floodplain Management Authorities Conference, Gosford, Central Coast.

37. DHI. (1996). An Integrated Modelling Package for Urban Drainage and Sewer Systems, Mouse User Manual, Version 4.0. DHI Water & Environment, Horsholm, Denmark.

38. DHI. (2007a). Mike 21 Flow Model, Hydrodynamic Module, User Guide. DHI Water & Environment.

39. DHI. (2007b). Mike She User Manual, User Guide. DHI Water & Environment.

40. Djordjevi´, S., Prodanovi´, D., Maksimovi´, C., Iveti ´, M. and Savi´, D. (2004). Simulation of Interaction between Pipe Flow and Surface Overland Flow in Networks. Water Science and Technology, 52 (5), 275–283.

41. DoW. (2004). *Perth Groundwater Atlas*, (2nd edition). Department of Water, Government of Western Australia, http://www.water.wa.gov.au/idelve/gwa/.

42. DoW. (2008a). Interim: Developing a Local Water Management Strategy. Department of Water, Government of Western Australia.

43. DoW. (2008b). Urban Water Management Plans. Department of Water, Government of Western Australia.

44. DoW. (2009). Jandakot Drainage and Water Management Plan. Department of Water, Government of Western Australia.

45. Elliott, A. H., and Trowsdale, S. A. (2007). A Review of Models for Low Impact Urban Stormwater Drainage. Environmental Modelling & Software, 22 (3): 394-405.

46. Ellis, J. B. (1995). Integrated Approaches for Achieving Sustainable Development of Urban Storm Drainage. Water Science and Technology, 32 (1), 1-6.

47. Emerge. (2012). Sunrise Wellard Stage 1-5 Urban Water Management Plan, Emerge Environmental Consultancies, Perth.

48. EPA. (1999). EPA's Preliminary Data Summary of Urban Stormwater Best Management Practices. U.S. Environmental Protection Agency, Washington, D. C.

49. EPA. (2005). Draft Guidance Statement No. 33: Environmental Guidance for Planning and Development. Environmental Protection Agency (EPA), Perth.

50. EPA. (2006). Low Impact Developments (LID) and Other Green Design Strategies. U.S. Environmental Protection Agency, Washington, D. C.

51. Fewtrell, T. J., Duncan, A., Sampson, C. C., Neal, J.C. and Bates, P. D. (2011). Benchmarking Urban Flood Models of Varying Complexity and Scale Using High Resolution Terrestrial LiDAR Data. Physics and Chemistry of the Earth, Parts A/B/C, 36 (7-8), 281-291.

52. Gersonius, B., Nasruddin, F., Ashley, R., Jeuken, A., Pathirana, A. and Zevenbergen, C. 2012. Developing the Evidence Base for Mainstreaming Adaptation of Stormwater Systems to Climate Change. Water Research, 12 (1).

53. Gioia G. and Bombardelli, F. A. (2001). Scaling and Similarity in Rough Channel Flows. The American Physical Society, 88 (01).

54. Gironás, J., Roesner, L. A., Rossman, L. A. and Davis, J. (2010). A New Applications Manual for the Stormwater Management Model (SWMM). Environmental Modelling & Software, 25 (6), 813-814.

55. Giuliano D. B., Guy S. and Paul D. B. (2009). A technique for the calibration of hydraulic models using uncertain satellite observations of flood extent. Journal of Hydrology, 367, 276–282.

56. Göbel, P., Stubbe, H., Weinert, M., Zimmermann, J., Fach, S., Dierkes, C., Kories H.,

57. Goonetilleke, A., Thomas, E., Ginn, S. and Gilbert, D. (2005). Understanding the Role of Land Use in Urban Stormwater Quality Management. Journal of Environmental Management 74 (1), 31-42.

58. GoWA. (2003). State Water Strategy. Department of Premier and Cabinet, Government of Western Australia.

59. Gozzard, J. R. (1983). The Perth Metropolitan Region 1:50,000 Environmental Geology Series – Rockingham (Part of Sheets 2033 Ii and 2033 Iii), Perth. Geological Survey of Western Australia.

60. Hill, P., Mein, R. and Siriwardena, L. (1998). How Much Rainfall Becomes Runoff? Loss Modelling for Runoff. Report 98/5, Cooperative Research Centre for Catchment Hydrology.

61. Hladik, C. and Alber M. (2012). Accuracy assessment and correction of a LIDAR-derived salt marsh digital elevation model, Remote Sensing of Environment, 121, 224-235.

62. Hsu M. H., Chen S. H. and Chang T. J. (2000). Inundation simulation for urban drainage basin with storm sewer system, Journal of Hydrology, 234 (1-2), 21-37.

63. Icke, J., van den Boomen, R. M. and Aalderink, R. H. (1999). A Cost-Sustainability Analysis of Urban Water Management. Water Science and Technology, 39 (5), 211-218.

64. IPCC. (2007). Intergovernmental Panel on Climate Change, Climate Change 2007 – Synthesis Report.

65. JDA. (1999). Canning Vale Central & Eastern Catchments Drainage & Nutrient. Management Plan, JDA Consultant Hydrologists.

66. Jefferies, C., Aitken, A., McLean, N., Macdonald, K. and McKissock, G. (1999). Assessing the Performance of Urban BMPs in Scotland. Water Science and Technology, 39 (12), 123-131.

67. Jian C., Arleen A. H. and Lensyl D. U. (2009). A GIS-based model for urban flood inundation, Journal of Hydrology, 373, 184–192.

68. Jones, P. and Macdonald, N. (2007). Making Space for Unruly Water: Sustainable Drainage Systems and the Disciplining of Surface Runoff. Geoforum, 38 (3), 534-544.

69. Kaplowitz, M. D. and Lupi, F. (2012). Stakeholder Preferences for Best Management Practices for Non-Point Source Pollution and Stormwater Control. Landscape and Urban Planning, 104 (3–4), 364-372.

70. Kazemi, F., Beecham, S. and Gibbs, J. (2009a). Street-scale Bio-retention Basins in Melbourne and Their Effect on Local Biodiversity. Ecological Engineering, 35 (10), 1454-1465.

71. Kazemi, F., Beecham, S., Gibbs, J. and Clay R. (2009b). Factors Affecting Terrestrial Invertebrate Diversity in Bio-retention Basins in an Australian Urban Environment. Landscape and Urban Planning, 92 (3–4), 304-313.

72. Kazemi, F., Beecham, S., Gibbs, J. and Clay R. (2011). Streetscape Biodiversity and the Role of Bio-retention Swales in an Australian Urban Environment. Landscape and Urban Planning, 101 (2), 139-148.

73. Lassabatere, L., Angulo-Jaramillo, R., Goutaland, D., Letellier, L., Gaudet, J.P., Winiarski, T. and Delolme, C., (2010). Effect of the settlement of sediments on water infiltration in two urban infiltration basins Geoderma, 156, 316–325.

74. Laurenson, E. M. (1964). A Catchment Storage Model for Runoff Routing. Journal of Hydrology, 2 (2), 141-163.

75. Lily, A., House-Peters, and Chang, H. (2011). Modelling the Impact of Land Use and Climate Change on Neighbourhood-Scale Evaporation and Night-time Cooling: A Surface Energy Balance Approach. Landscape and Urban Planning, 103 (2), 139-155.

76. Lin, Y. P., Hong, N. M., Wu, P., Wu, C. and Verburg, P. H. (2007). Impacts of Land Use Change Scenarios on Hydrology and Land Use Patterns in the Wu-Tu Watershed in Northern Taiwan. Landscape and Urban Planning, 80 (1–2), 111-126.

77. Liu, Y., and Sun, F. (2010). Sensitivity Analysis and Automatic Calibration of a Rainfall–Runoff Model Using Multi-Objectives. Ecological Informatics, 5 (4), 304-310.

78. Luijten, J.C. (2003). A systematic method for generating land use patterns using stochastic rules and basic landscape characteristics: results for a Colombia hillside watershed. Agriculture Ecosystem and Environment, 95, 427–441.

79. Manson, S.M. (2005). Agent-Based Modelling and Genetic Programming for Modelling Land Change in the Southern Yucatan. Agriculture Ecosystems and Environment, 111, 47–62.

80. Martin, C., Ruperd, Y., Legret, M. (2007). Urban Stormwater Drainage Management: The Development of a Multi-criteria Decision Aid Approach for Best Management Practices. European Journal of Operational Research, 181, 338- 349.

81. Messer, J., Mertsch, V., Geiger, W. F. and Coldewey, W. G. (2004). Near-Natural Stormwater Management and Its Effects on the Water Budget and Groundwater Surface in Urban Areas Taking Account of the Hydro-geological Conditions. Journal of Hydrology, 299 (3-4), 267-283.

82. Metermate. (2011). Observation Data. Water and Energy Management Pty. Ltd. http://www.metermate.com.au/MMLogin.aspx.

83. Mignot, E., Paquier, A. and Haider, S. (2006). Modelling Floods in a Dense Urban Area Using 2d Shallow Water Equations. Journal of Hydrology, 327 (1-2), 186-199.

84. Mitchell, G. (2005). Mapping Hazard from Urban Non-Point Pollution: A Screening Model to Support Sustainable Urban Drainage Planning. Journal of Environmental Management, 74 (1), 1-9.

85. Morison, P. J., and Brown, R. R. (2011). Understanding the Nature of Publics and Local Policy Commitment to Water Sensitive Urban Design. Landscape and Urban Planning, 99 (2), 83-92.

86. Mouri, G., Kanae, S. and Oki, T. (2011). Long-Term Changes in Flood Event Patterns Due to Changes in Hydrological Distribution Parameters in a Rural-Urban Catchment, Shikoku, Japan. Atmospheric Research (In Press), Corrected Proof.

87. Neal, J. C., Bates, P. D., Fewtrell, T J., Hunter, N. M., Wilson, M. D. and Horritt, M. S. (2009). Distributed Whole City Water Level Measurements from the Carlisle 2005

Urban Flood Event and Comparison with Hydraulic Model Simulations. Journal of Hydrology, 368 (1-4), 42-55.

88. Nourani, V., Singh, V. P. and Delafrouz, H. (2009). Three Geo-morphological Rainfall-Runoff Models Based on the Linear Reservoir Concept. CATENA, 76 (3), 206-214.

89. Parker, D., Meretsky, V., 2004. Measuring pattern outcomes in an agent-based model of edge-effect externalities using spatial metrics. Agricult. Ecosyst. Environ. 101, 233–250.

90. Parker, D.C., Berger, T. and Manson, S.M. (2002). Agent-based Models of Land Use and Land Cover Change. Report and Review of an International Workshop, 124, October 4–7, 2001, LUCC Focus 1 Office, Indiana.

91. Pauleit, S., Ennos, R. and Golding, Y. (2005). Modelling the Environmental Impacts of Urban Land Use and Land Cover Change - a Study in Merseyside, UK Landscape and Urban Planning, 71 (2–4), 295-310.

92. PGDER. (1999). Low-Impact Development Design Strategies, An Integrated Design Approach. Prince George's County, Maryland, Washington, U.S. Environmental Protection Agency, D.C.

93. Phillips, B.C., Yu, S., Thompson, G.R. and De Silva, N. (2005) 1D and 2D modelling of urban drainage systems using XP-SWMM and YUFLOW. 10th International conference on urban drainage, Copenhagen/Denmark, 21 -26.

94. Pilgrim, D.H. (1987). Australian Rainfall and Runoff: Volume 2, The institute of Engineers, Australia.

95. Pitt, R., Clark, S. and Field F. (1999). Groundwater Contamination Potential from Stormwater Infiltration Practices. Urban Water, 1 (3), 217-236.

96. Pontius, R.G., Malason, J., 2005. Comparison of the accuracy of land change models: cellular Automata Markov versus Geomod. Int. J. Geograph. Inform. Sci. 19, 243–265.

97. Priestnall, G., Jaafar J. (2000). Extracting Urban Features from Lidar Digital Surface Models, Computers Environment and Urban Systems, 24 (2), 65-78.

98. Pudasaini, S.P. (2011). Some exact solutions for debris and avalanche flows. Physics of Fluids 23 (4).

99. Pyke, C., Warren, M. P., Johnson, T., LaGro Jr, J., Scharfenberg, J., Groth, P., Freed, F., Schroeer, W. and Main, E. (2011). Assessment of Low Impact Development for Managing Stormwater with Changing Precipitation Due to Climate Change. Landscape and Urban Planning, 103 (2), 166-173.

100. Qi, H. and Altinakar M. S. (2011). A GIS-Based Decision Support System For Integrated Flood Management under Uncertainty with Two Dimensional Numerical Simulations. Environmental Modelling &amp, Software 26 (6), 817-821.

101. Richards, L. A. (1931). Capillary conduction of liquids through porous medium. Physics, 1, 318-333.

102. Rijsberman, M.A., and Van de Ven, F. H. M. (2000). Different Approaches to Assessment of Design and Management of Sustainable Urban Water Systems. Environmental Impact Assessment Review, 20 (3), 333-345.

103. Rossman, L. A. (2004). Stormwater Management Model User's Manual; Version 5.0. CINCINNATI, OH 45268.

104. Rounsevell, M.D.A., Annetts, J.E., Audsley, E., Mayr, T. and Reginster, I. (2003). Modelling the Spatial Distribution of Agricultural Land Use at the Regional Scale. Agriculture Ecosystems and Environment, 95, 465–479.

105. Sanders, B. F., Schubert, J. E. and Gallegos, H. A. (2008). Integral Formulation of Shallow-Water Equations with Anisotropic Porosity for Urban Flood Modelling. Journal of Hydrology, 362 (1-2), 19-38.

106. Sansalone, J., Kuang, X., Ying, G. and Ranieri, V. (2011). Filtration and Clogging of Permeable Pavement Loaded by Urban Drainage. Water Research.

107. Sarhadi, A., Soltani S. (2012). Probabilistic Flood Inundation Mapping of Ungauged Rivers: Linking GIS Techniques and Frequency Analysis. Journal of Hydrology, 458–459, 68-86.

108. Schilling, W. (1991). Rainfall Data for Urban Hydrology: What Do We Need?, Atmospheric Research, 27 (1–3), 5-21.

109. Schumann, G. J. P., Neal, J. C. (2011). The Accuracy of Sequential Aerial Photography and SAR Data for Observing Urban Flood Dynamics, a Case Study of the UK Summer 2007 Floods. Remote Sensing of Environment 115 (10), 2536-2546.

110. Scholz, M. and Sadowski A. J. (2009). Conceptual classification model for Sustainable Flood Retention Basins. Journal of Environmental Management, 90 (1), 624-633.

111. Selvalingam, S., Liong, S. Y. and Manoharan, P. C. (1987). Use of RORB and SWMM Models to an Urban Catchment in Singapore. Advances in Water Resources, 10 (2), 78-86.

112. Semadeni-Davies, A., Hernebring, C., Svensson, G. and Gustafsson, L. (2008a). The Impacts of Climate Change and Urbanisation on Drainage in Helsingborg, Sweden: Suburban Stormwater. Journal of hydrology, 350, 114-125.

113. Semadeni-Davies, A., Hernebring, C., Svensson, G. and Gustafsson, L. (2008b). The Impacts of Climate Change and Urbanisation on Drainage in Helsingborg, Sweden: Combined Sewer System. Journal of Hydrology, 350 (1–2), 100-113.

114. Simpson, M. G. (2012). Low Impact Development Modelling to Manage Urban Stormwater Runoff and Restore Predevelopment Site Hydrology. Department of Civil and Environmental Engineering, Colorado State University Fort Collins, Colorado.

115. Siriwardena, L., Finlayson, B. L. and McMahon, T. A. (2006). The Impact of Land Use Change on Catchment Hydrology in Large Catchments: The Comet River, Central Queensland, Australia. Journal of Hydrology, 326 (1–4), 199-214.

116. Smith, J. A., Phillips, B.C. and Yu, S. (2006). Modelling Overland Flows and Drainage Augmentation in Dubbo. In 46th Floodplain Management Authorities Conference, Lismore.

117. Smith, M.B. (2006). Comment on ‘Analysis and Modelling of Flooding in Urban Drainage Systems’. Journal of Hydrology, 317 (3-4), 355-363.

118. SoSJ. (2003). Local Planning Policy Lpp9; Multiple Use Trails within the Shire of Serpentine – Jarrahdale. Sire of Serpentine Jarrahdale (SoSJ), Western Australia.

119. Stewart, T.J., Janssen, R. and Van Herwijnen, M. (2004). A Genetic Algorithm Approach to Multi Objective Land Use Planning. Computer & Operation Research, 31, 2293–2313.

120. Suarez, P., Anderson, W., Mahal, V. and Lakshmanan, T. R. (2005). Impacts of Flooding and Climate Change on Urban Transportation: A System-wide Performance Assessment of the Boston Metro Area. Transportation Research Part D: Transport and Environment 10 (3), 231-244.

121. Syme, W.J. (1991). Dynamically Linked Two-Dimensional / One-Dimensional Hydrodynamic Modelling Program for Rivers, Estuaries & Coastal Waters. William Syme, M.Eng.Sc (Research) Thesis, Department of Civil Engineering, The University of Queensland.

122. Syme, W.J., Pinnell, M.G., Wicks, J. M. (2004). Modelling Flood Inundation of Urban Areas in the UK Using 2D / 1D Hydraulic Models. The Institution of Engineers, Australia, 8th National Conference on Hydraulics in Water Engineering.

123. Tarekegn, T. H., Haile, A. T. (2010). Assessment of an ASTER-Generated DEM for 2D Hydrodynamic Flood Modelling, International Journal of Applied Earth Observation and Geoinformation, 12 (6), 457-465.

124. Theo, G. S., Martin, T., Norman, E. (2004). Analysis and Modelling of Flooding in Urban Drainage Systems. Journal of Hydrology, 299, 300-311.

125. Thorkild, H. J. (1988). Analysis of Rainfall Series in the Design of Urban Drainage Control Systems. Water Research, 22 (4), 491-496.

126. ToK. (2005). Draft Eastern Residential Intensification Concept. Town of Kwinana (ToK), Western Australia.

127. TUFLOW. (2010). TUFLOW Manual: GIS Based 2D/1D Hydrodynamic Modelling. Build 2010-10-AA, BMT & WBM.

128. Unidata. (2007). Starflow Ultrasonic Doppler Instrument, Manual for Model 6526.

129. United Nations Centre for Human Settlement, 1996, United Nations Centre for Human Catchmenttlement (HABITAT), 1996, An Urbanizing World. Global Report on Human Catchmenttlements. Oxford University Press, Oxford.

130. Urbonas, B. and Stahre, P. (1993). Stormwater - Best Management Practices and Detention. Prentice Hall, Englewood Cliffs, NJ.

131. Vijay, P. S. and David, A. W. (2002). Mathematical Modelling of Watershed Hydrology. American Society of Civil Engineers, 150th anniversary paper.

132. Wagener, T., Wheater, H. S. and Gupta, H. V. (2004). Rainfall-Runoff Modelling in Gauged and Ungauged Catchments. London, Imperial College Press.

133. Wagner, C. (2009). Canning Vale Central Catchment Stormwater Drainage Review. City of Gosnells, Connell Wagner.

134. Wallingford. 1997. Using Hydroworks™ . Wallingford Software Ltd.

135. Wang, X., Yu, S., Huang, G.H., 2004. Land allocation based on integrated GIS-optimization modelling at a watershed level. Landscape Urban Plan. 66, 61–74.

136. WAPC. (2006a). State Planning Policy 2.9 Water Resources. Western Australian Planning Commission (WAPC), Perth, Western Australia.

137. WAPC. (2006b). Statement of Planning Policy No. 3: Urban Growth and Settlement. Western Australian Planning Commission (WAPC), Perth, Western Australia.

138. WAPC. (2007). Liveable Neighbourhoods; a Western Australian Government Sustainable Cities Initiative, January 2009 Update 2. Western Australia Planning Commission (WAPC), Perth, Western Australia.

139. WAPC. (2008). Better Urban Water Management, Western Australian Planning Commission (WAPC), Perth, Western Australia.

140. WAPC. (2009). Metropolitan Region Scheme (Mrs), Western Australian Planning Commission (WAPC), Perth.

141. Wheater, H. and Evans, E. (2009). Land Use, Water Management and Future Flood Risk. Land Use Policy 26, Supplement 1, S251-S264.

142. Willems, P., Arnbjerg-Nielsen, K., Olsson, J. and Nguyen, V. T. V. (2012). Climate Change Impact Assessment on Urban Rainfall Extremes and Urban Drainage: Methods and Shortcomings. Atmospheric Research, 103, 106-118.

143. XPSoftware. (2009). XPSWMM Stormwater & Wastewater Management Model. Getting Started Manual.

144. Yen, B. C. (1986). Hydraulics of Sewers. Advances in Hydroscience, 14, 1-122.

145. Yu, D. (2010). Parallelization of a Two-Dimensional Flood Inundation Model Based on Domain Decomposition. Environmental Modelling & Software, 25 (8), 935-945.

146. Zhang, L., Dawes, W.R. and Walker, G.R. (1999). Predicting the Effect of Vegetation Changes on Catchment Average Water Balance, Technical Report 99/12, Cooperative Research Centre for Catchment Hydrology, 35.

147. Zoppou, C. (2001). Review of Urban Stormwater Models. Environmental Modelling & Software, 16 (3), 195-231.

APPENDIX A

FIGURES TO THE CASE STUDY OF CANNING VALE CENTRAL CATCHMENT DRAINAGE ASSESSMENT

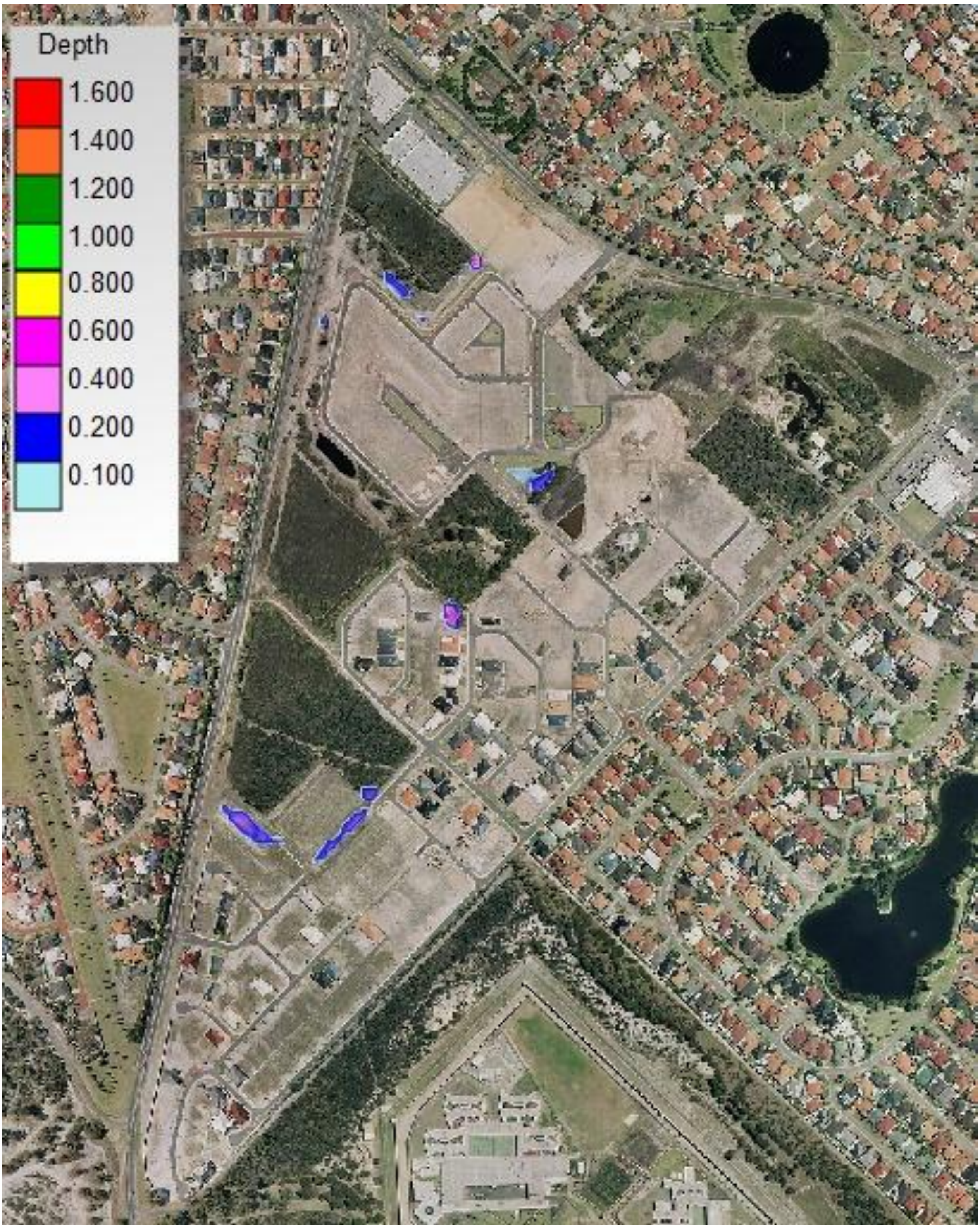


Figure 29. Glenariff 1 year ARI flood inundation map

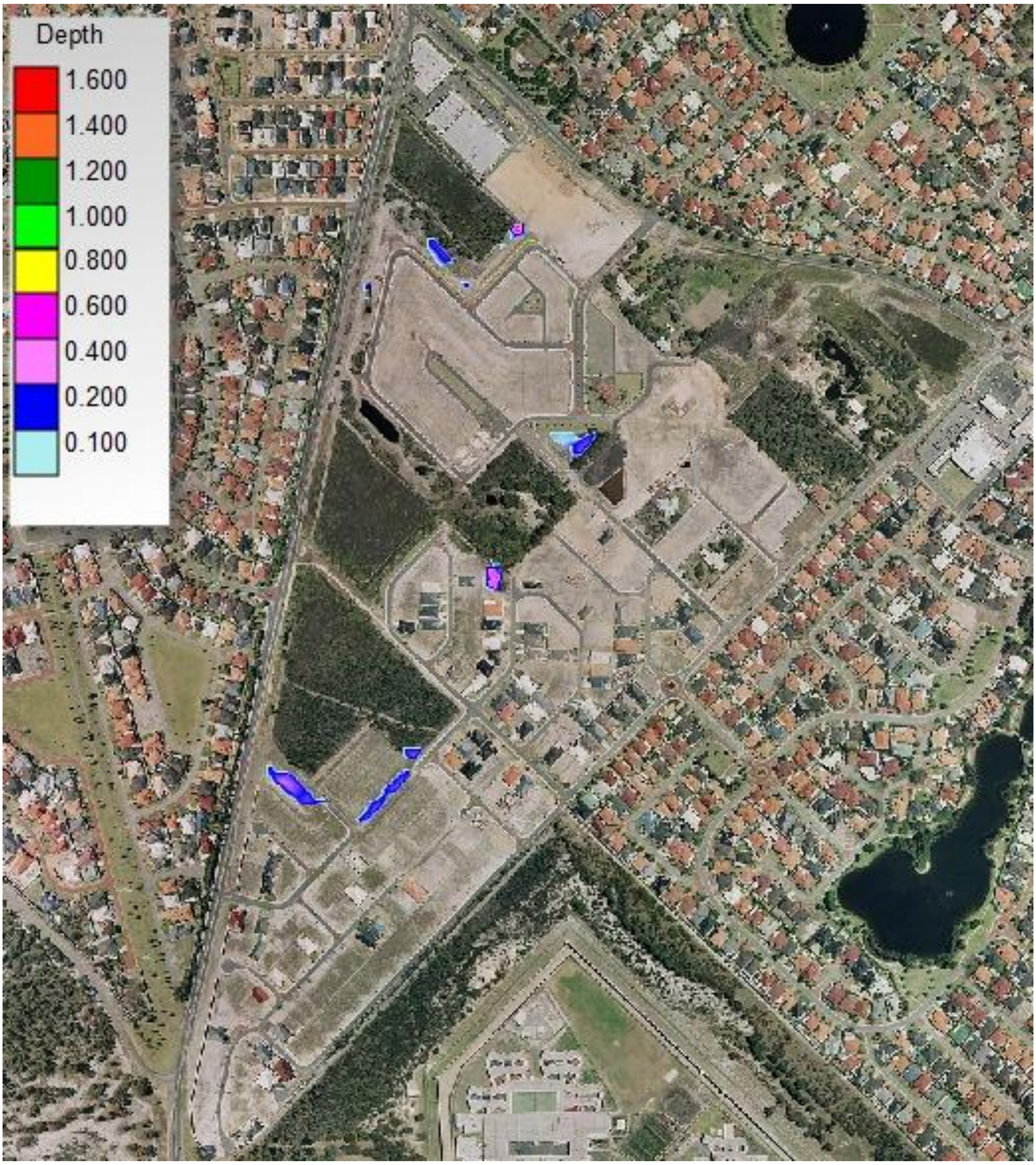


figure 30. Glenariff 5 year ARI flood inundation map

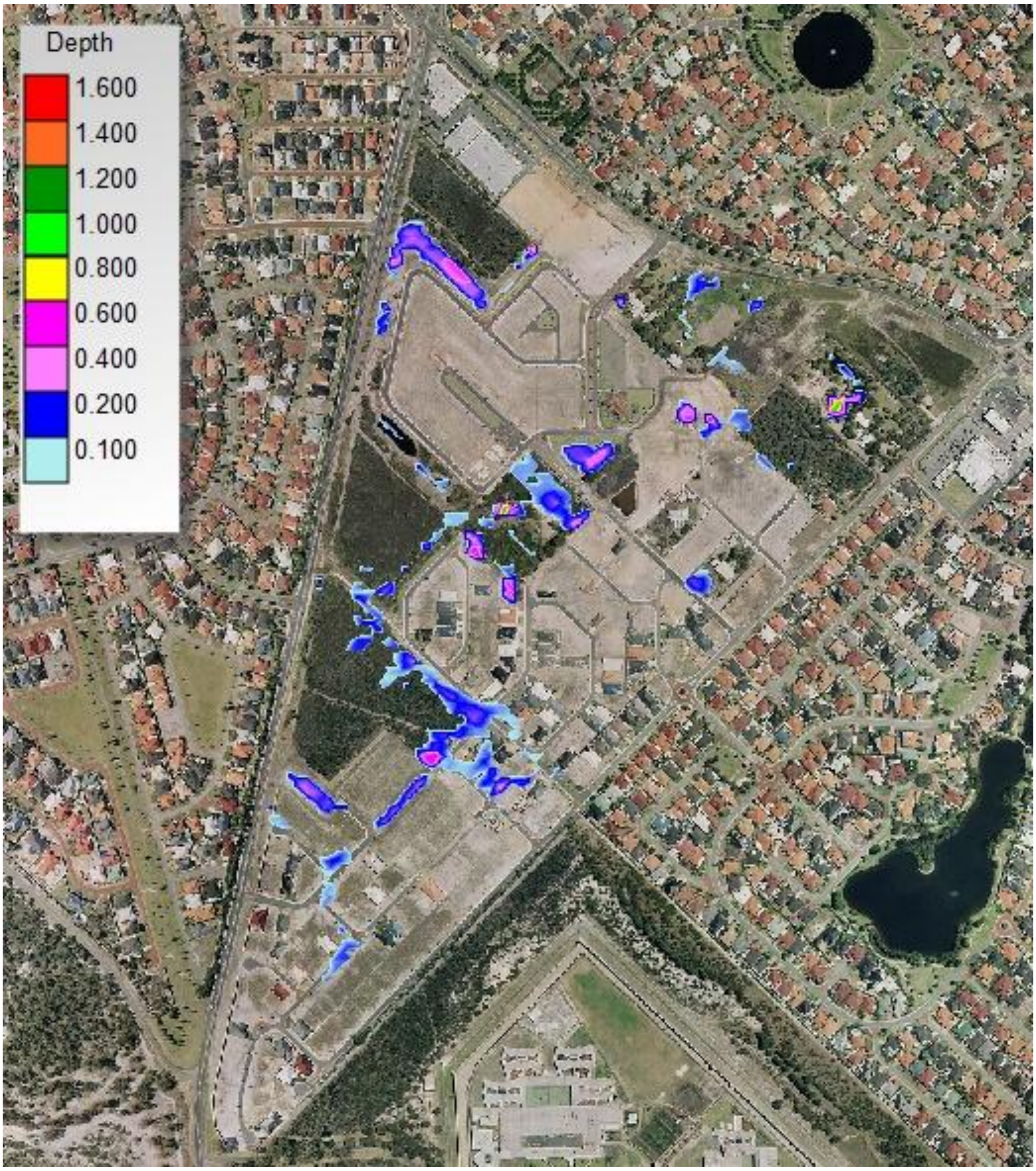


Figure 31. Glenariff 10 year ARI flood inundation map

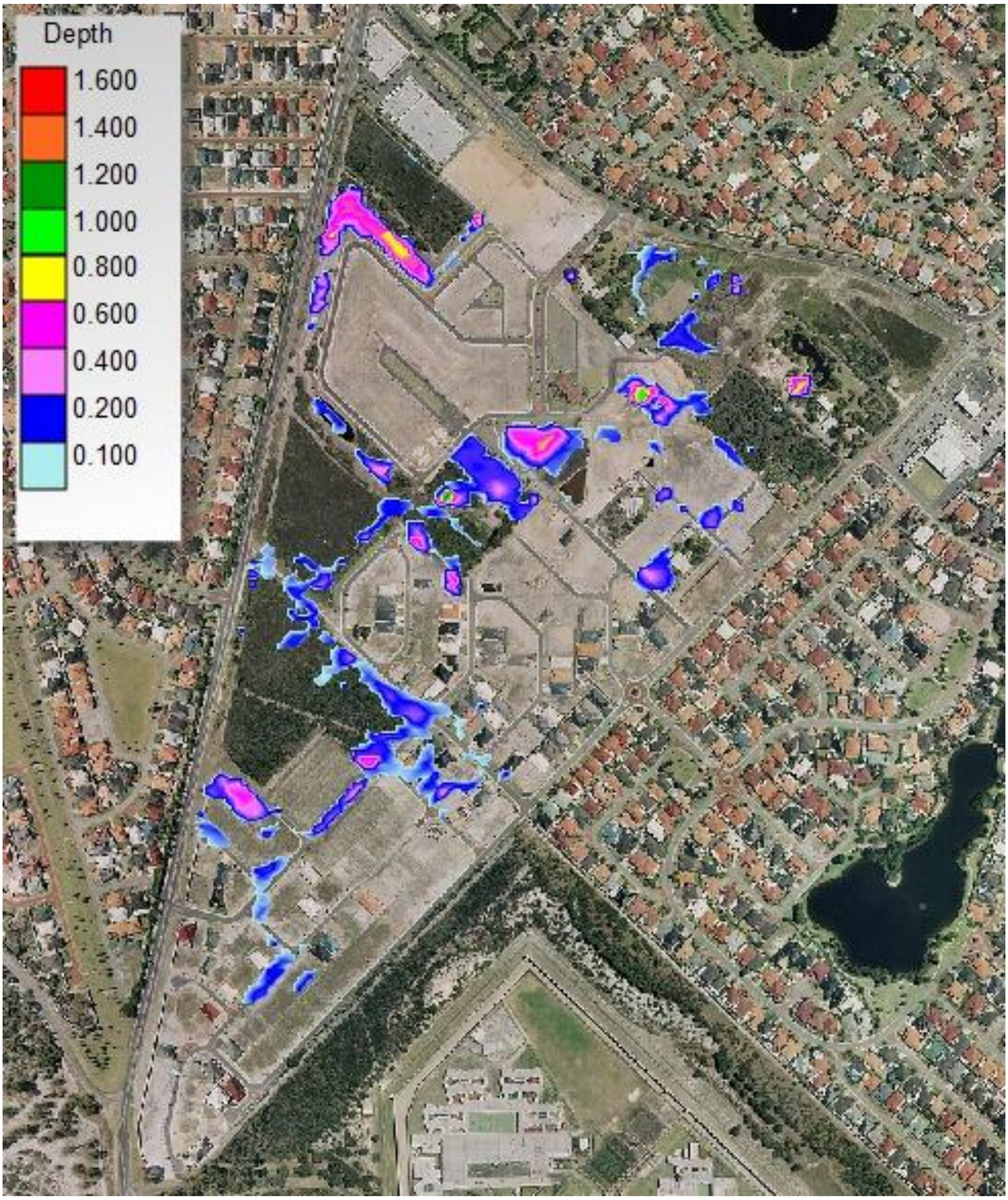


Figure 32. Glenariff 100 year ARI flood inundation map

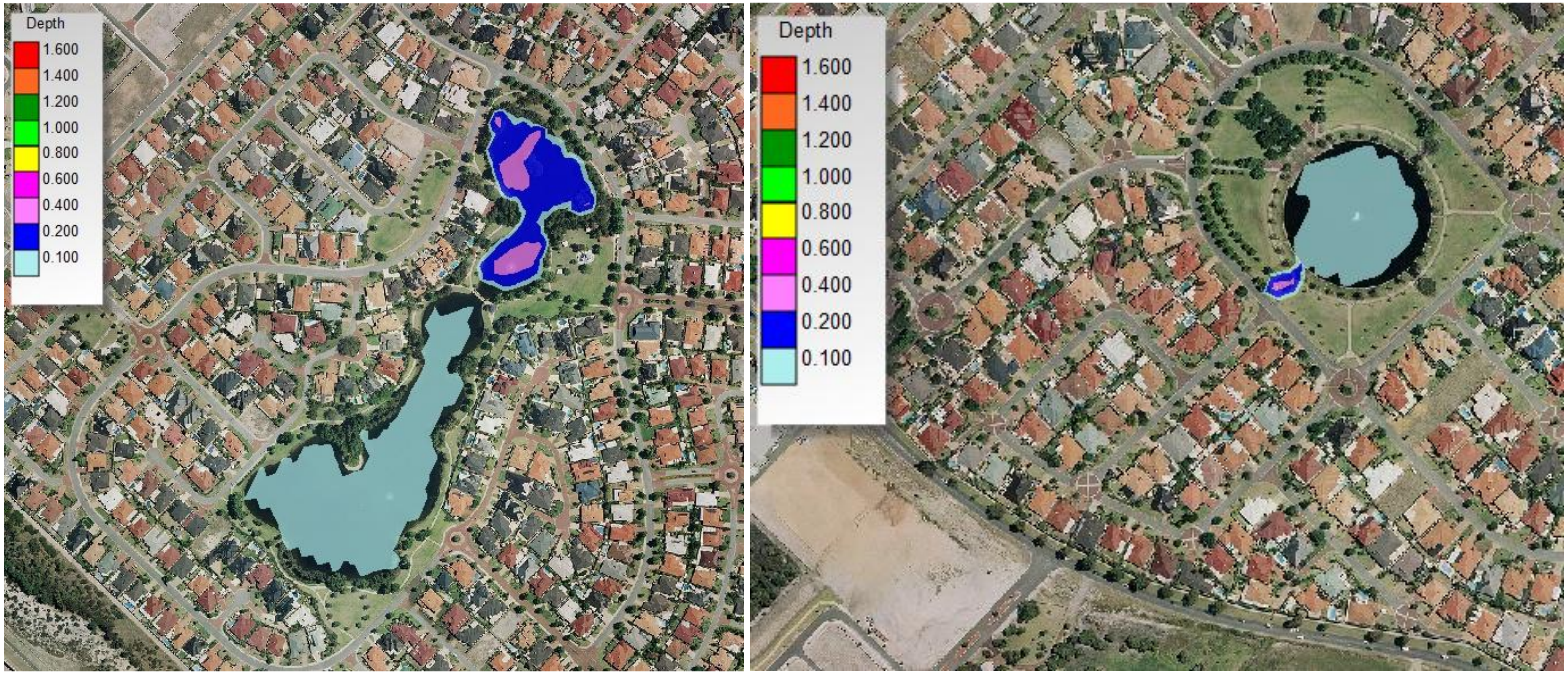


Figure 33. Sanctuary Lake and Avenues 1 year ARI flood inundation map

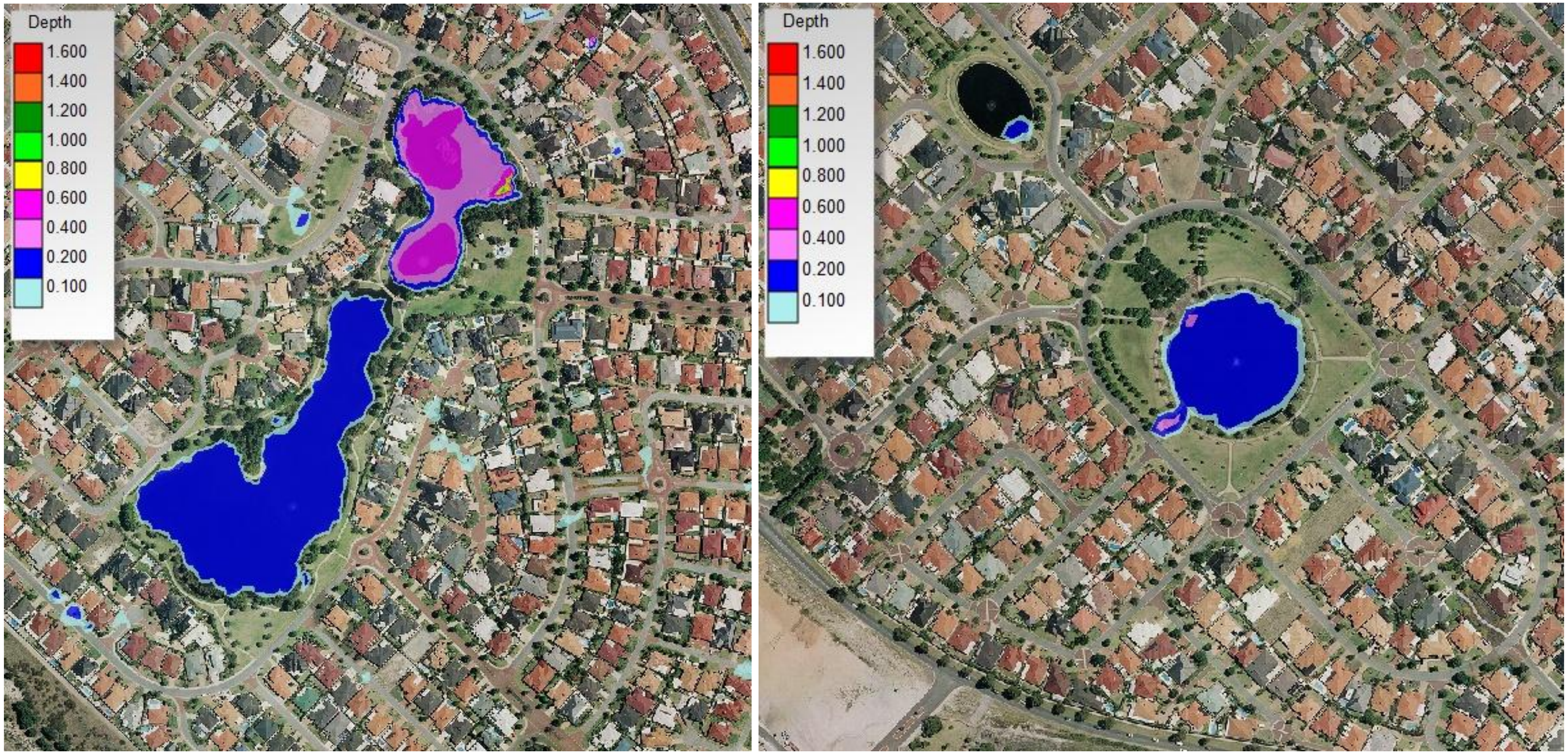


Figure 34. Sanctuary Lake and Avenues 5 year ARI flood inundation map

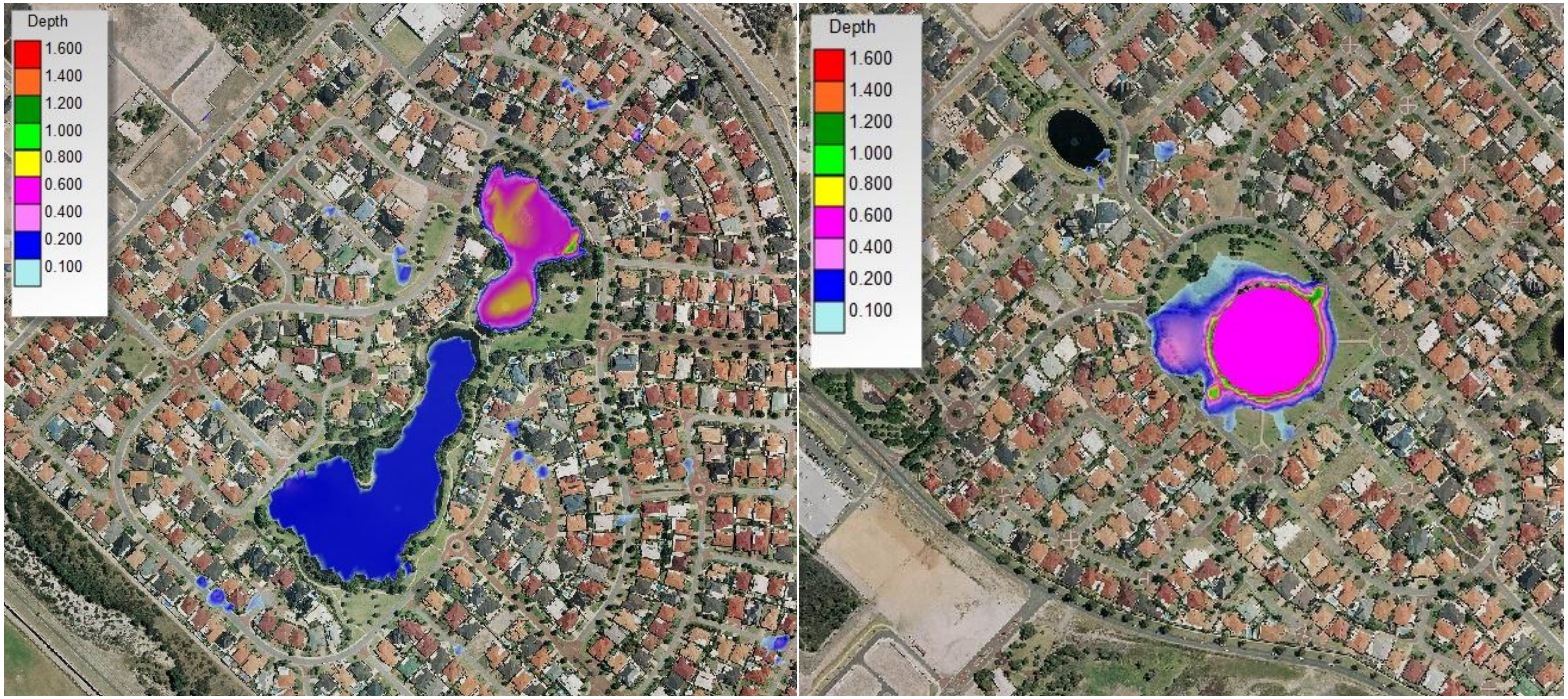


Figure 35. Sanctuary Lake and Avenues 10 year ARI flood inundation map

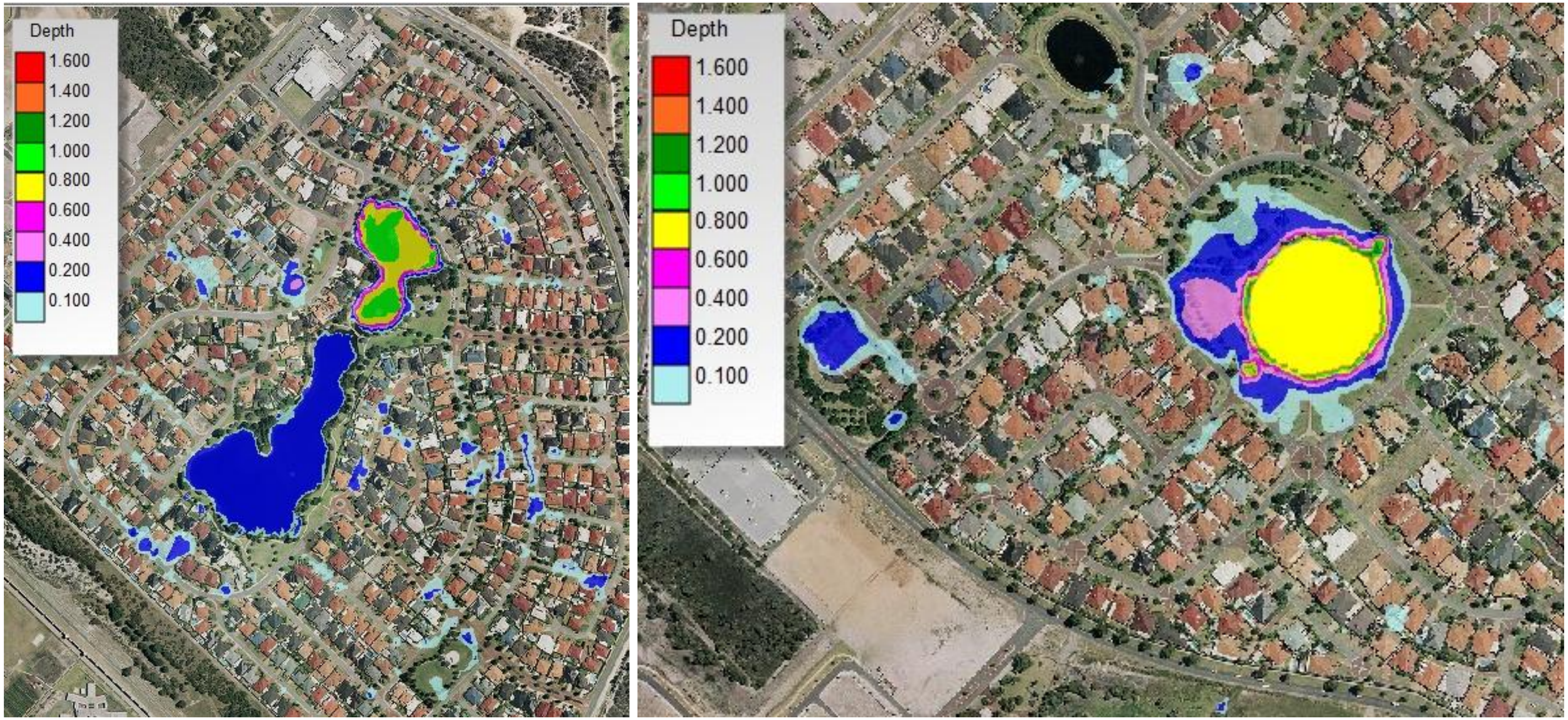


Figure 36. Sanctuary Lake and Avenues 100 year ARI flood inundation map

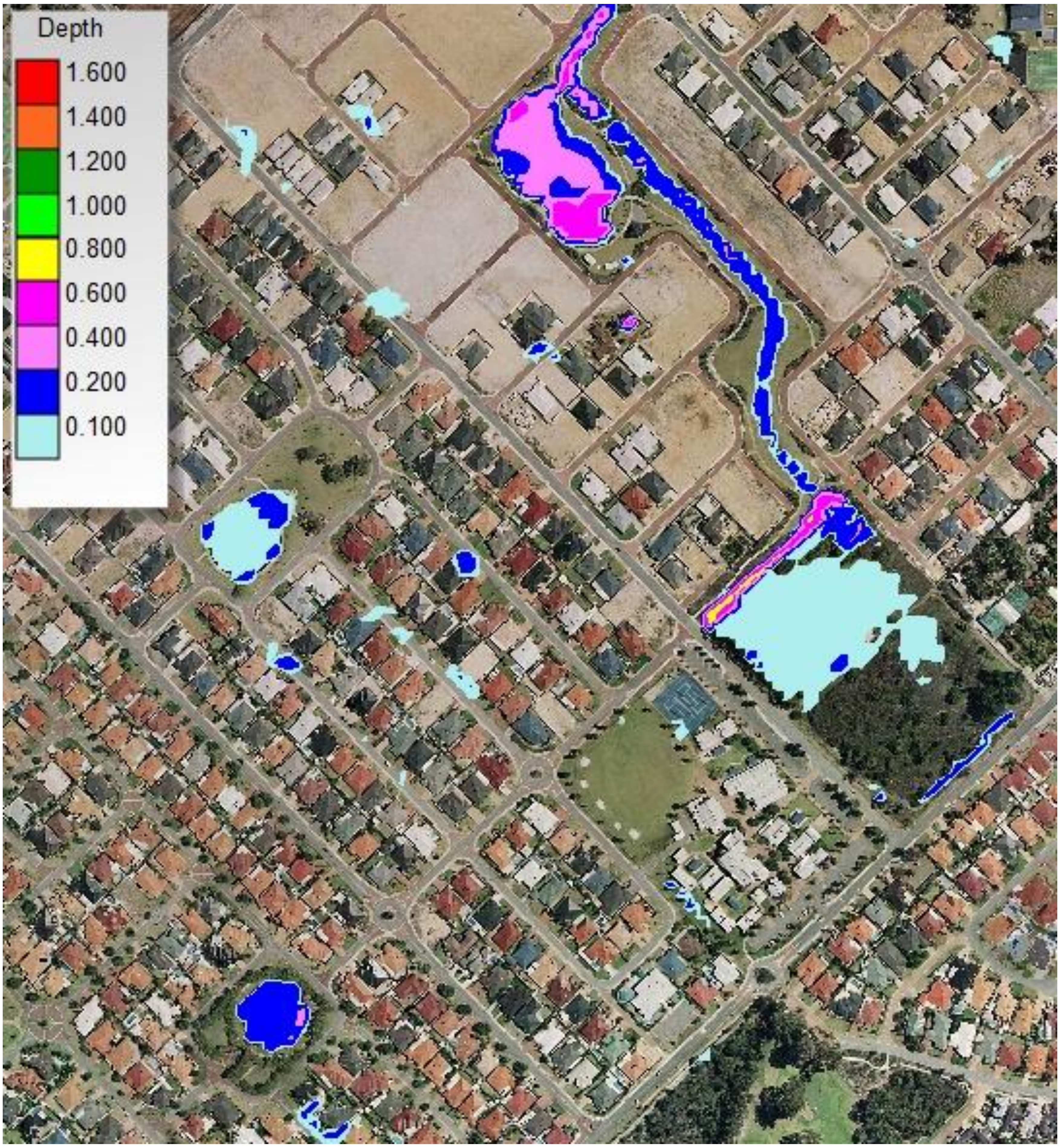


Figure 37. Main Drain 1 year ARI flood inundation map #1

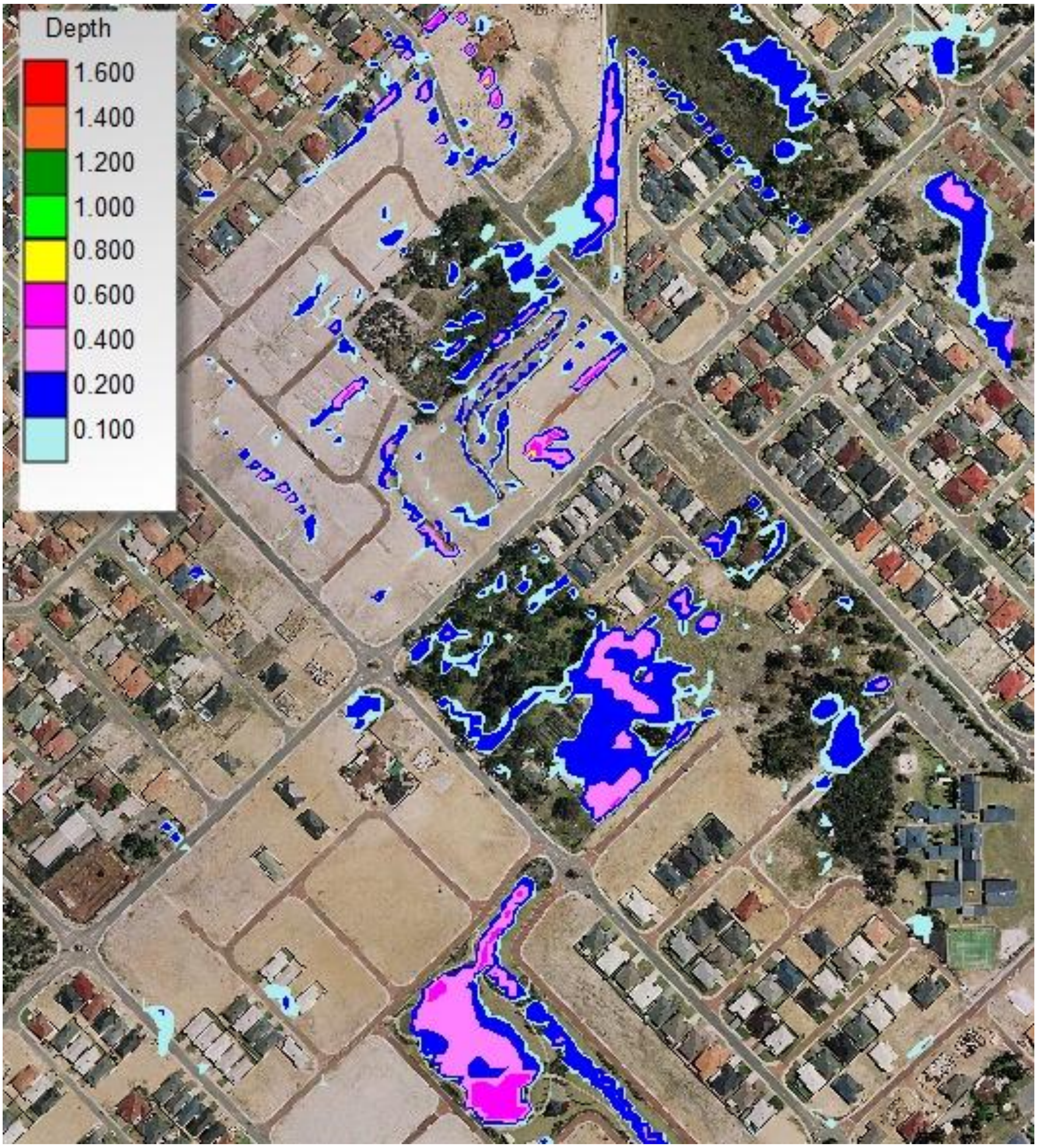


Figure 38. Main Drain 1 year ARI flood inundation map #2

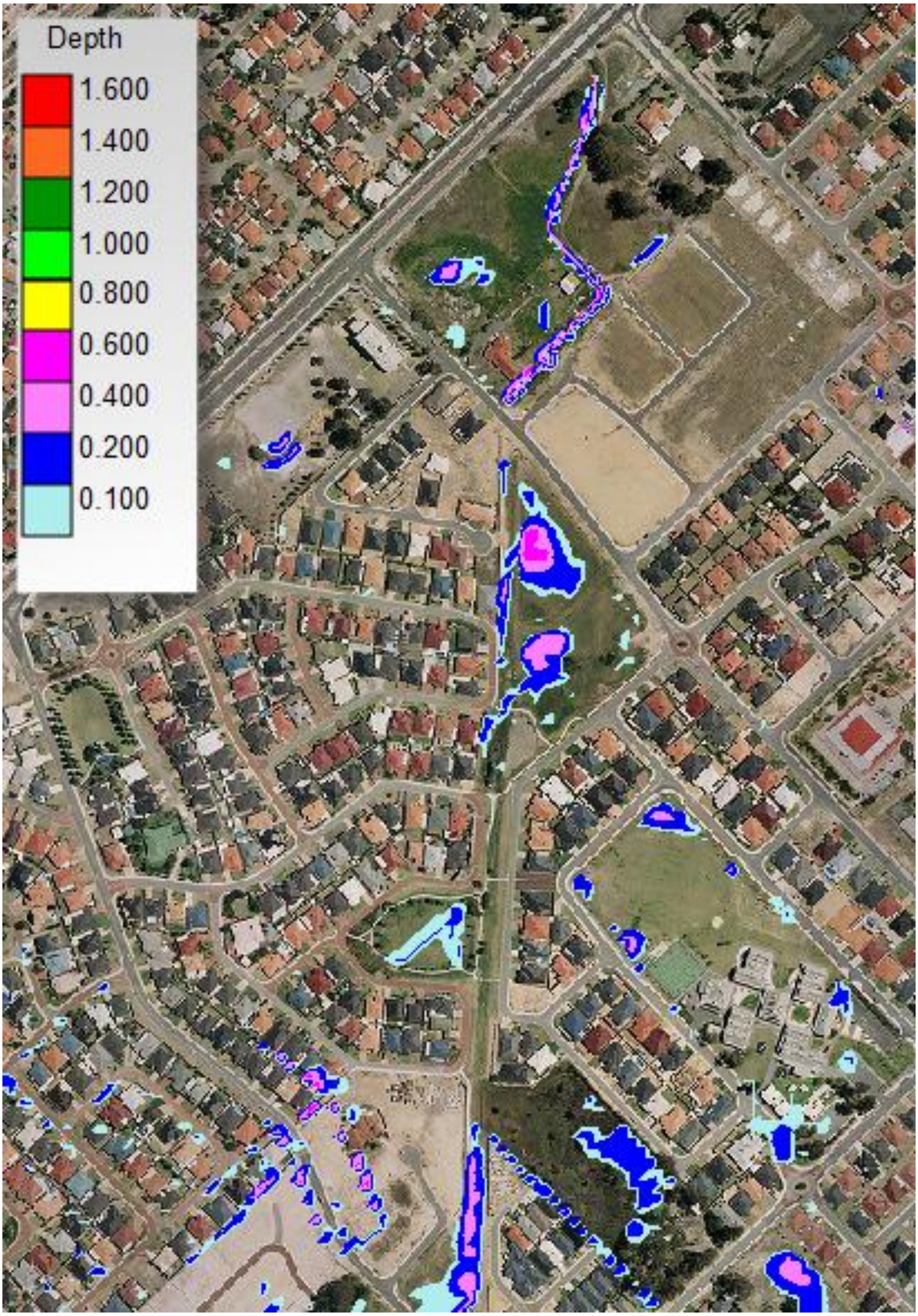


Figure 39. Main Drain 1 year ARI flood inundation map #3

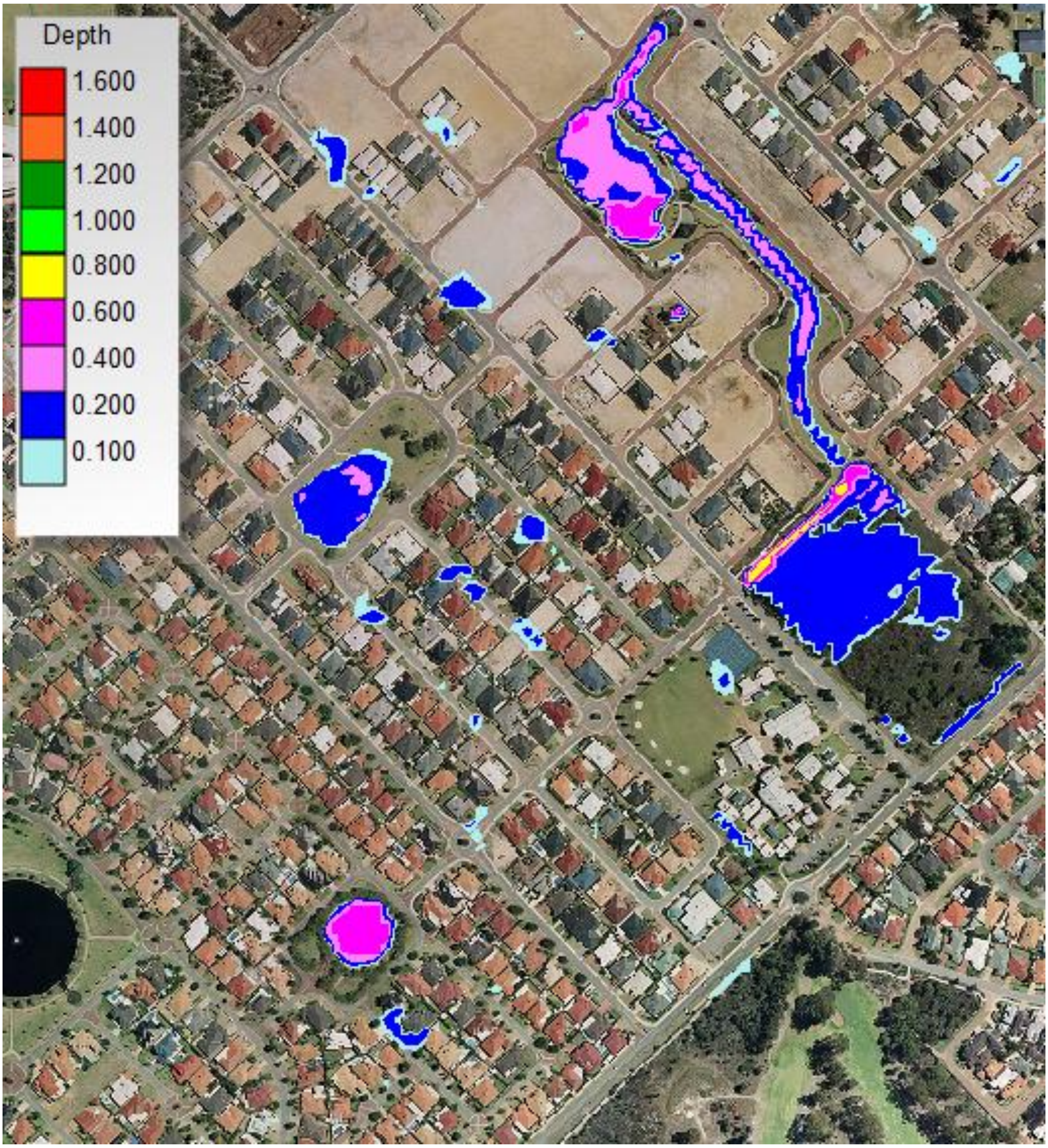


Figure 40. Main Drain 5 year ARI flood inundation map #1

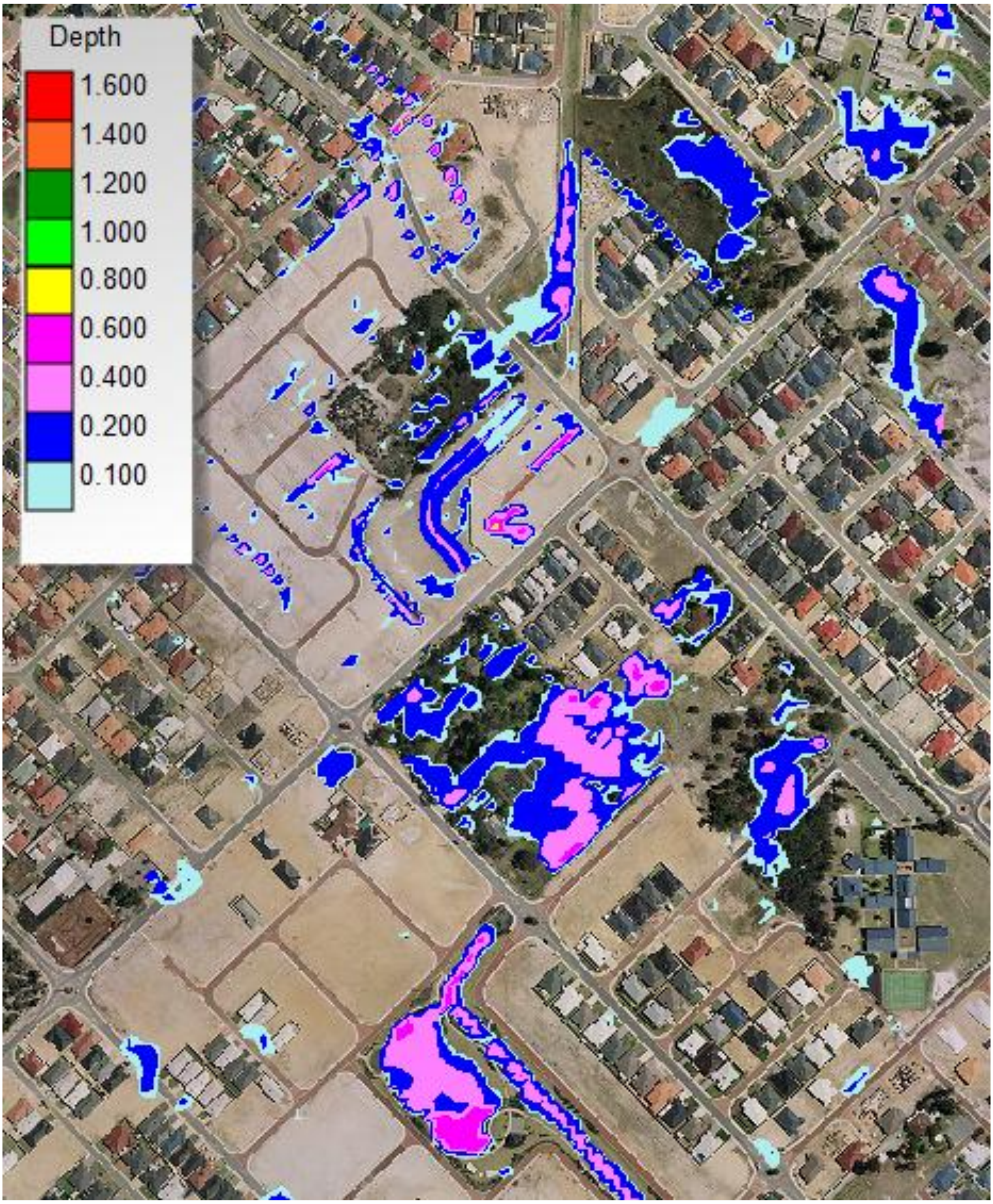


Figure 41. Main Drain 5 year ARI flood inundation map #2

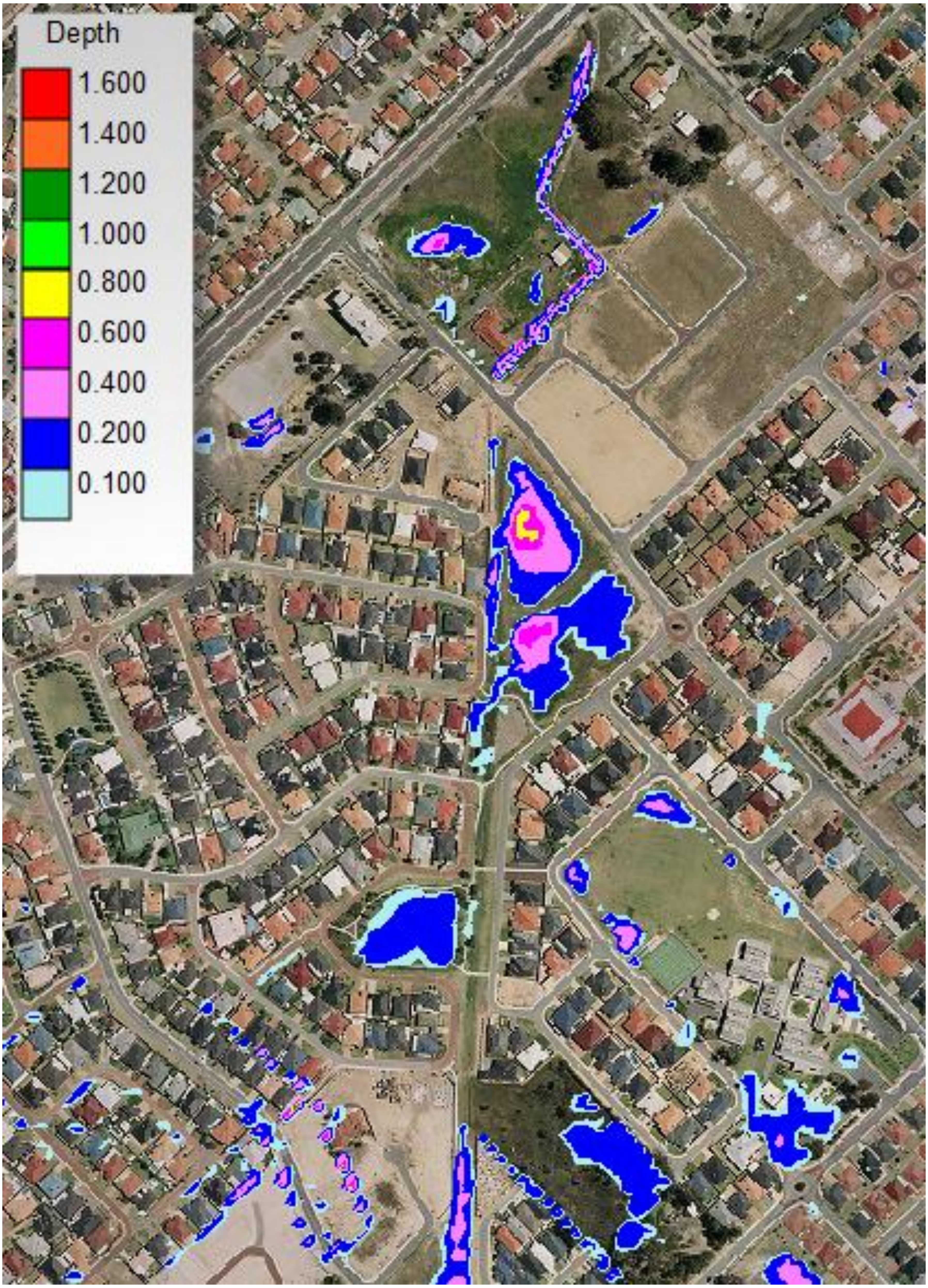


Figure 42. Main Drain 5 year ARI flood inundation map #3

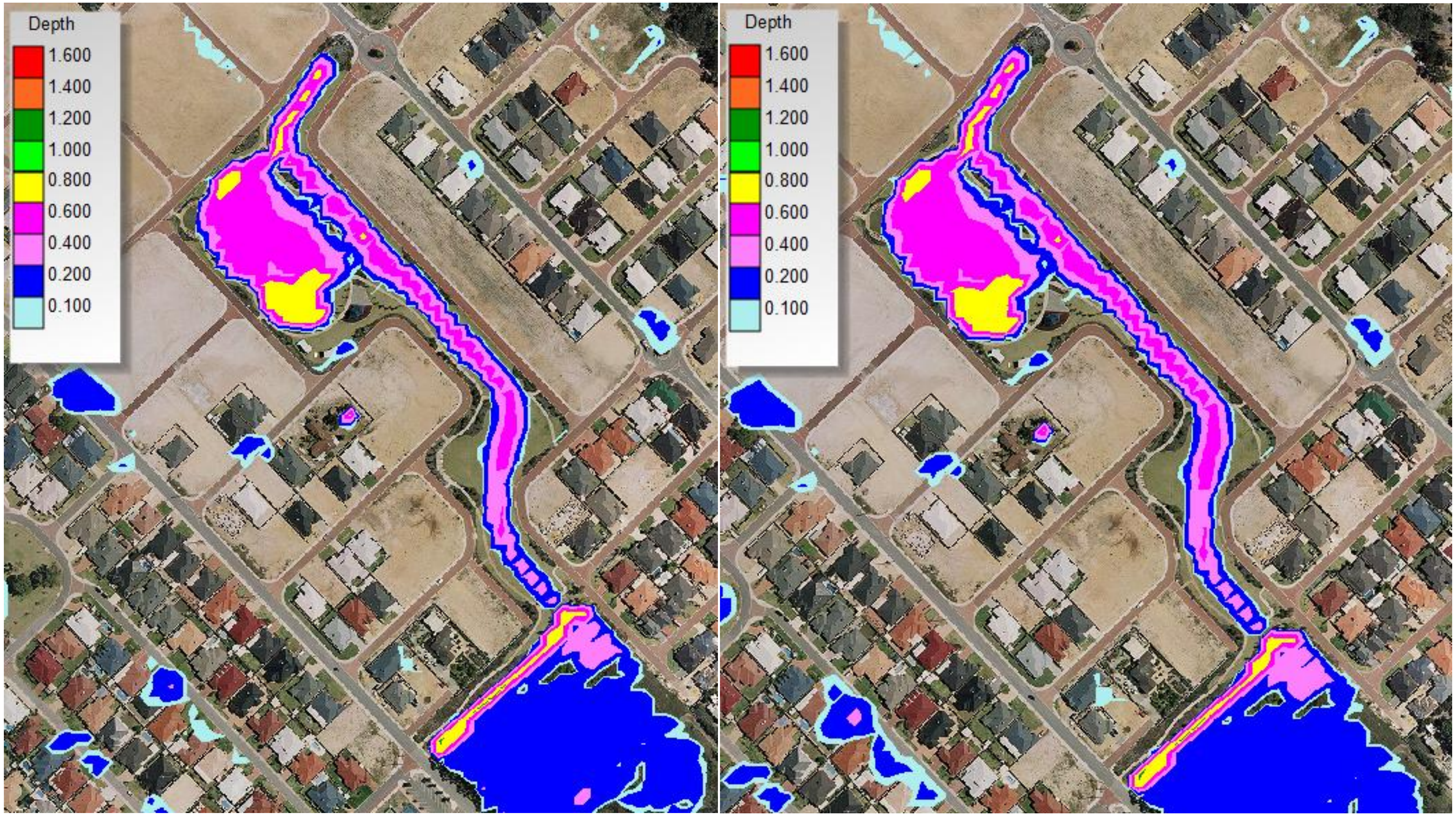


Figure 43. Auckland swale flood inundation maps for 10 year and 100 year ARI events.

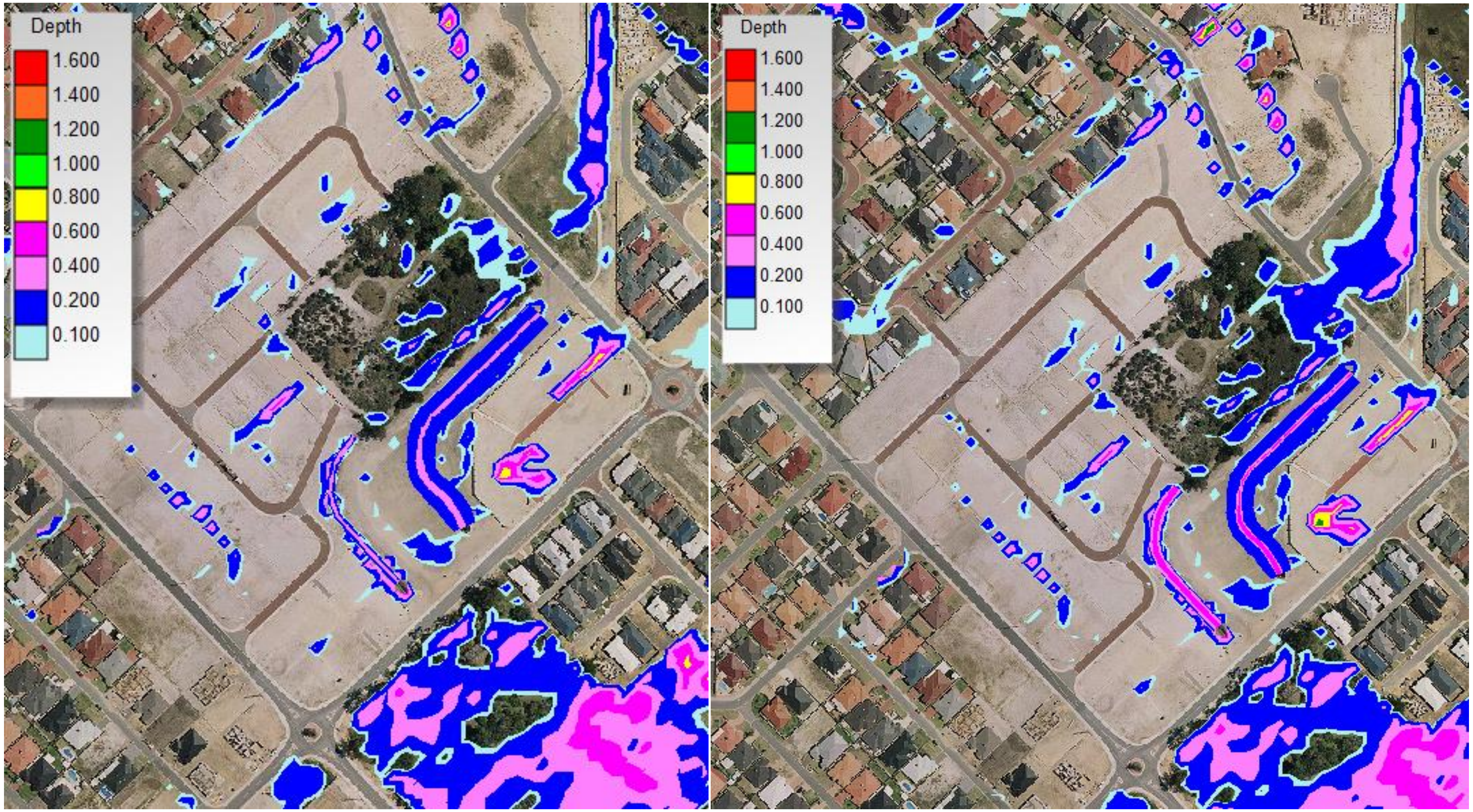


Figure 44. Doncaster open drains flood inundation maps for 10 year and 100 year ARI events.

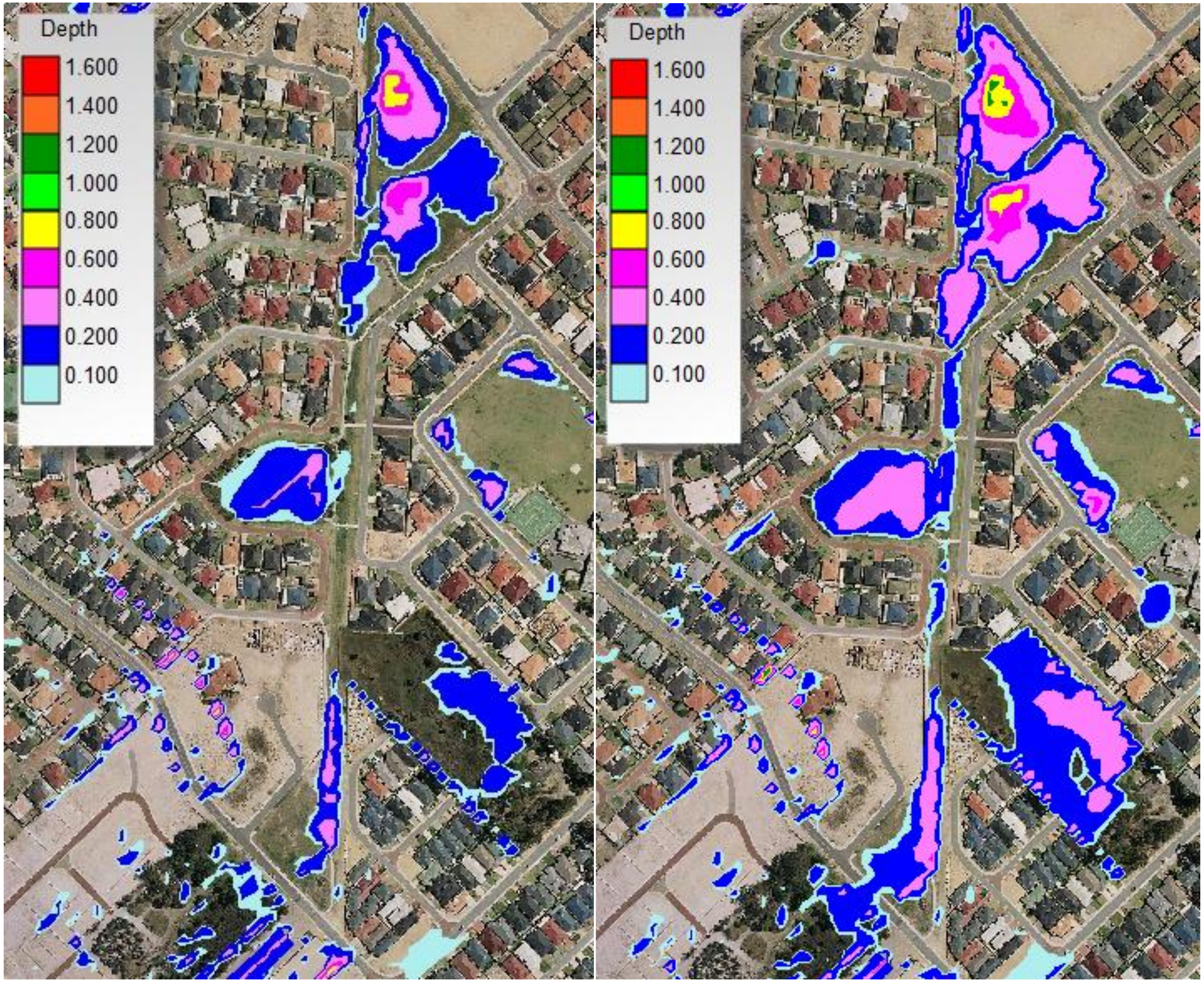


Figure 45. flood maps for the area along the MUC for 10 year and 100 year ARI events.

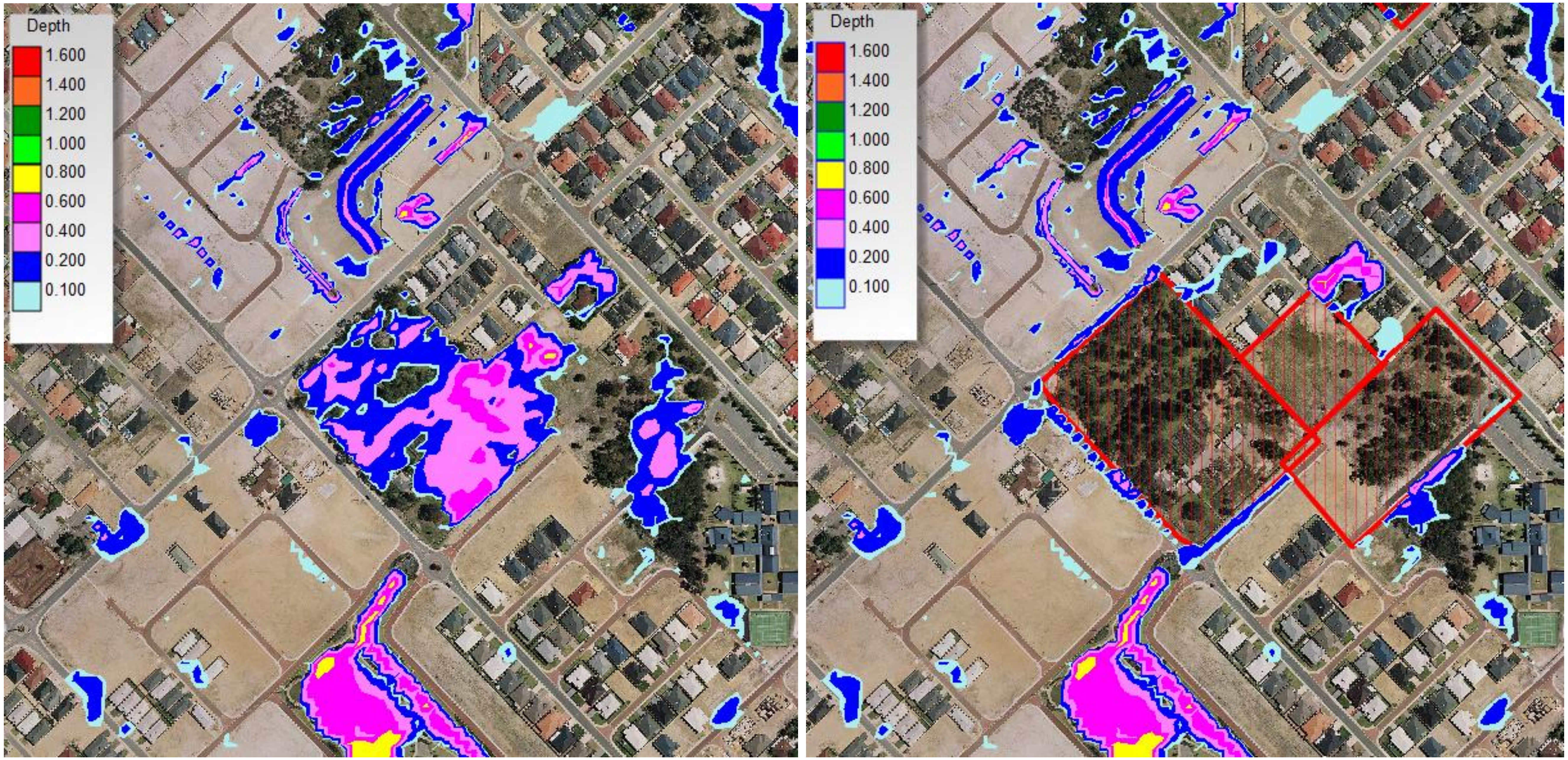


Figure 46. Warrendale Nursery subdivision site and Church subdivision flood maps for 10 year ARI event under present and future scenarios

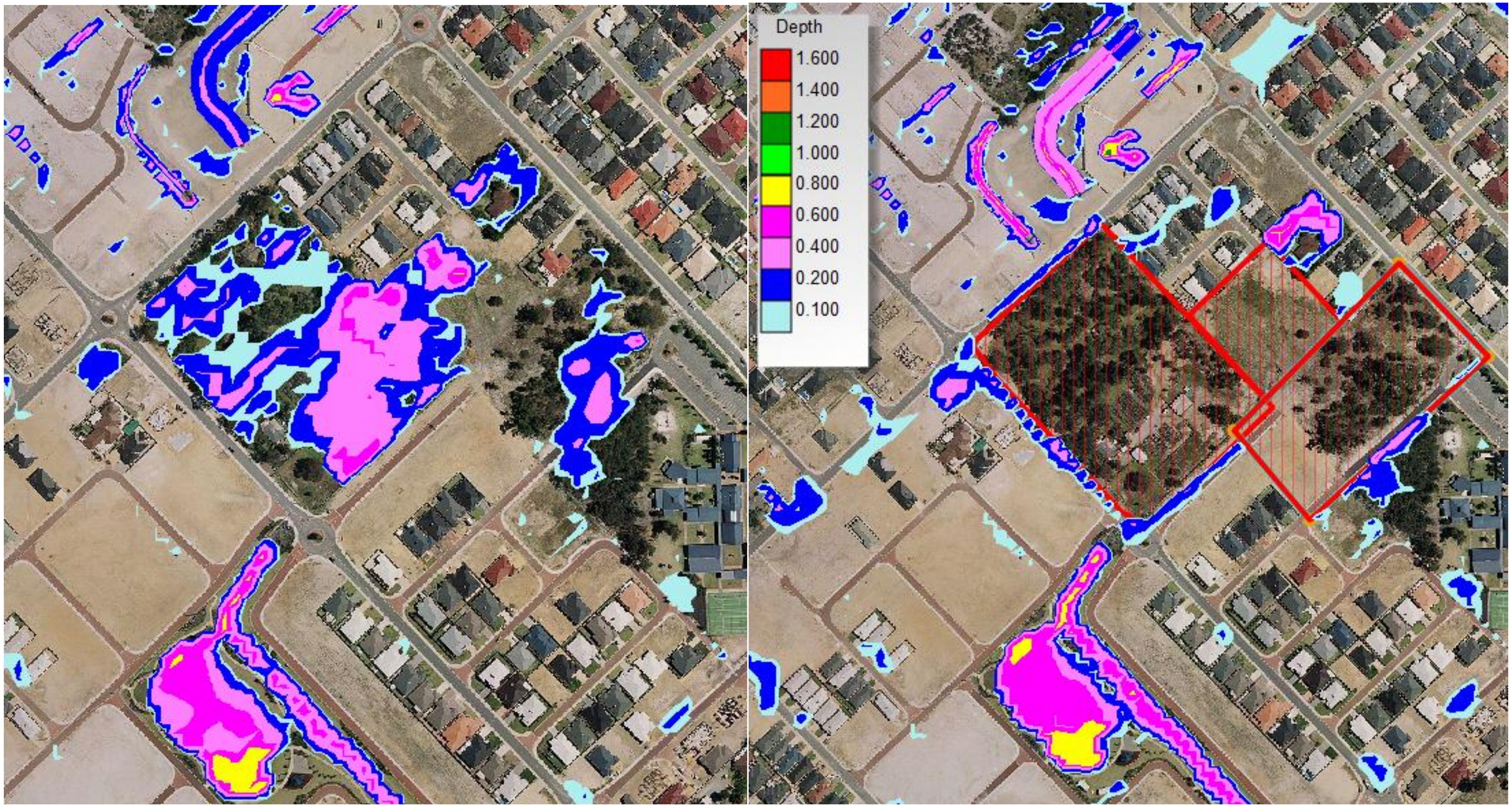


Figure 47. Warrendale Nursery subdivision site and Church subdivision flood maps for 100 year ARI event under present and future scenarios

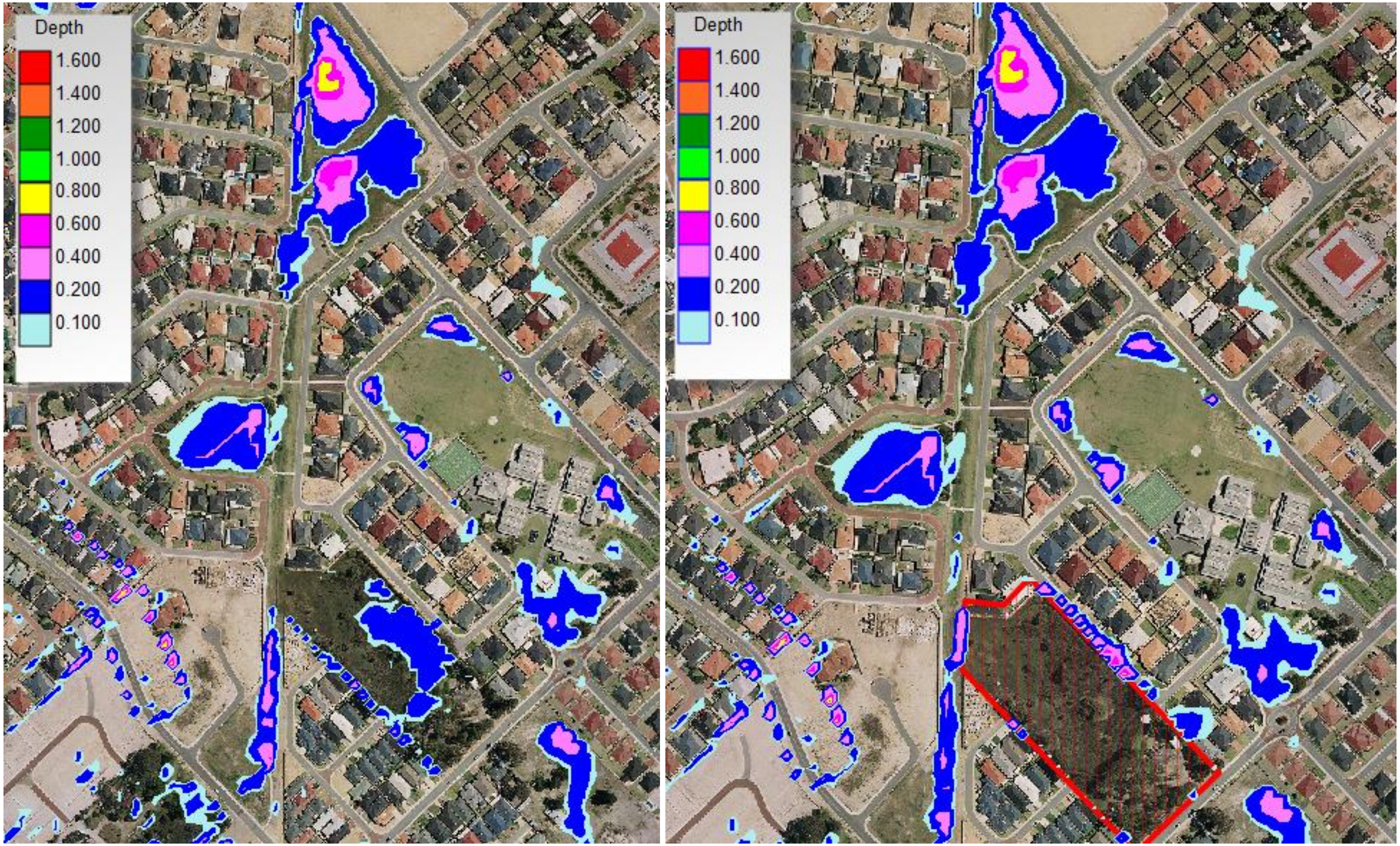


Figure 48. Fraser Road North subdivision site flood maps for 10 year ARI event under present and future scenarios

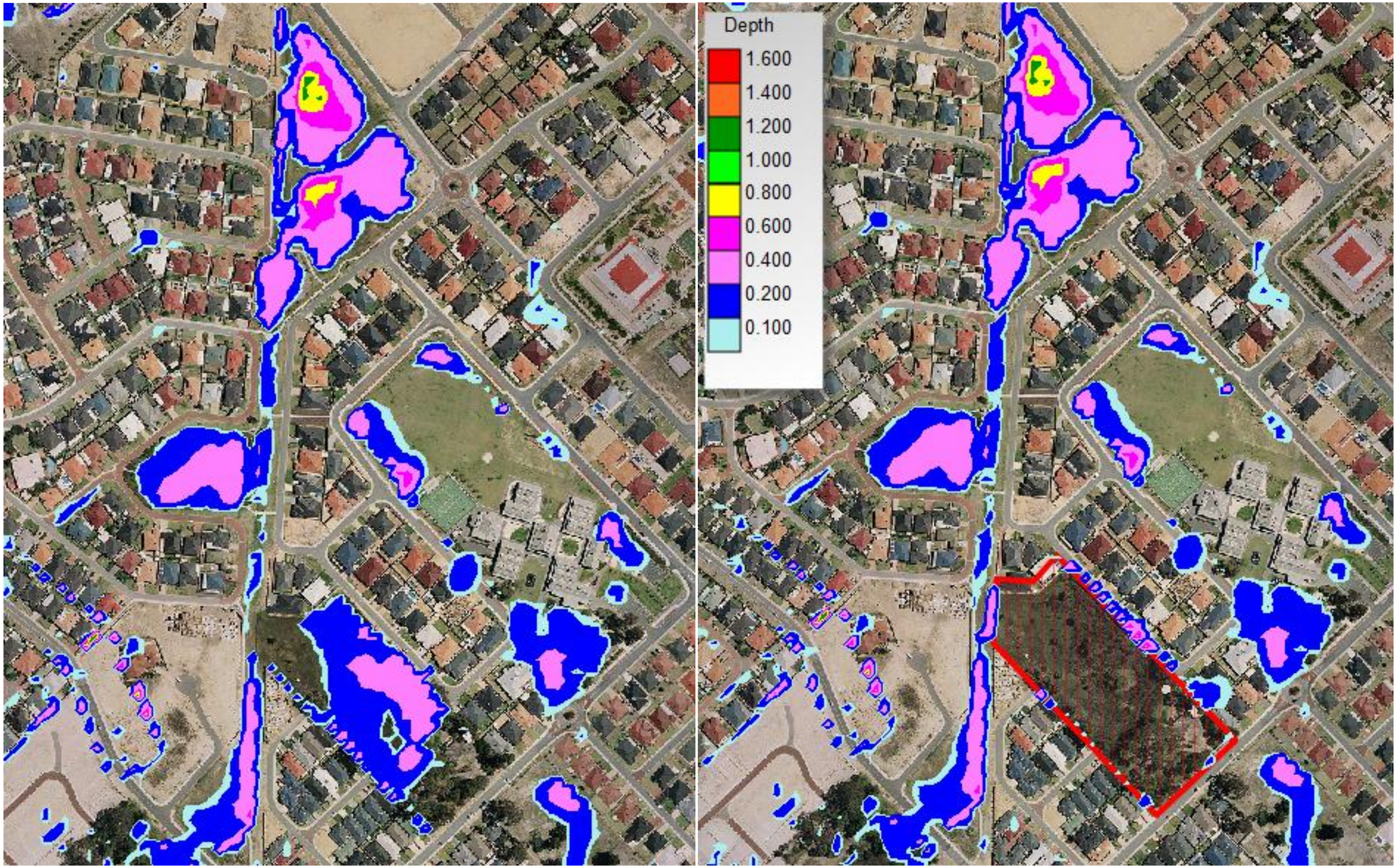


Figure 49. Fraser Road North subdivision site flood maps for 100 year ARI event under present and future scenarios

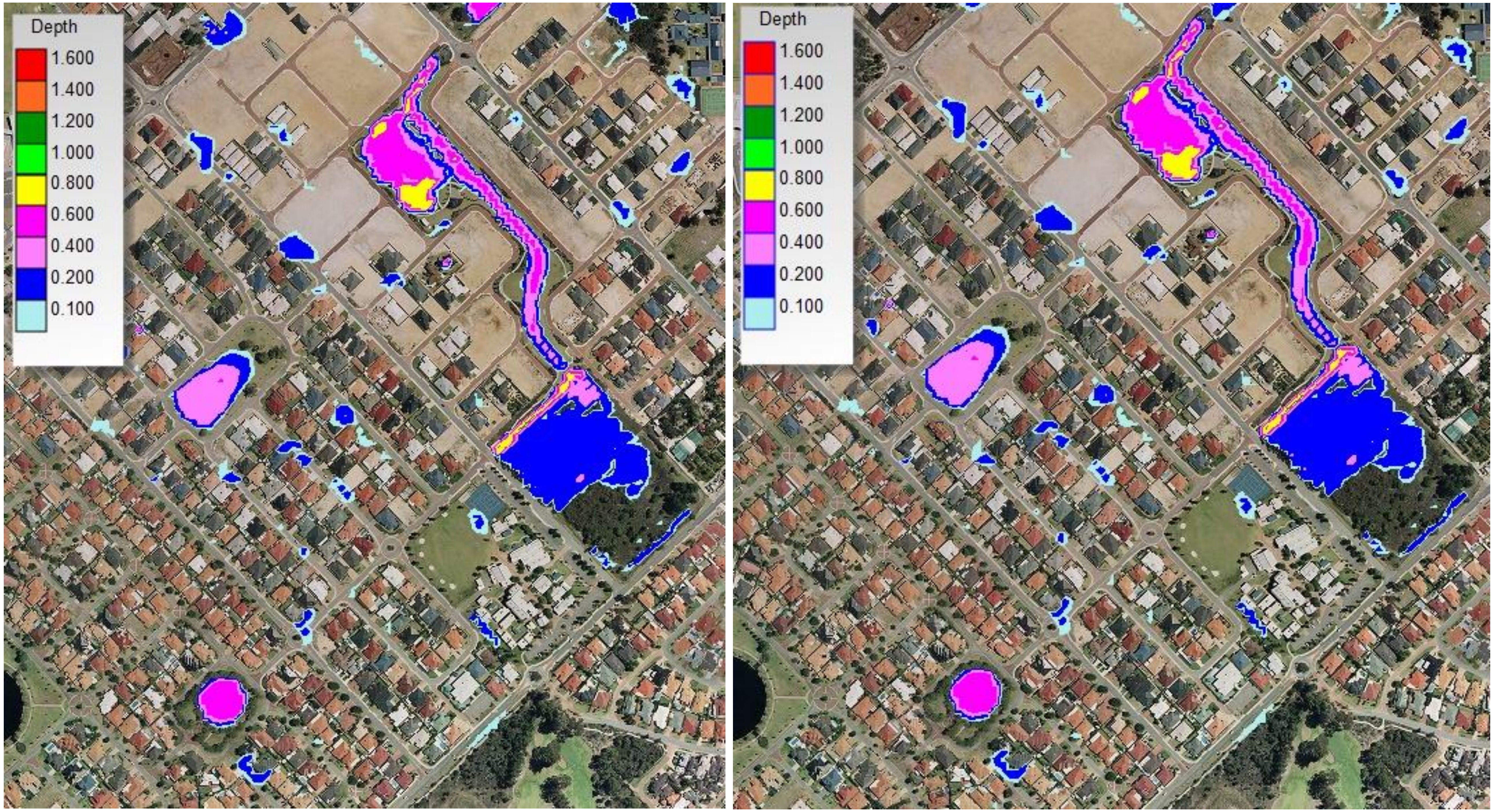


Figure 50. Main Drain 10 year ARI pre and post development flood inundation map #1

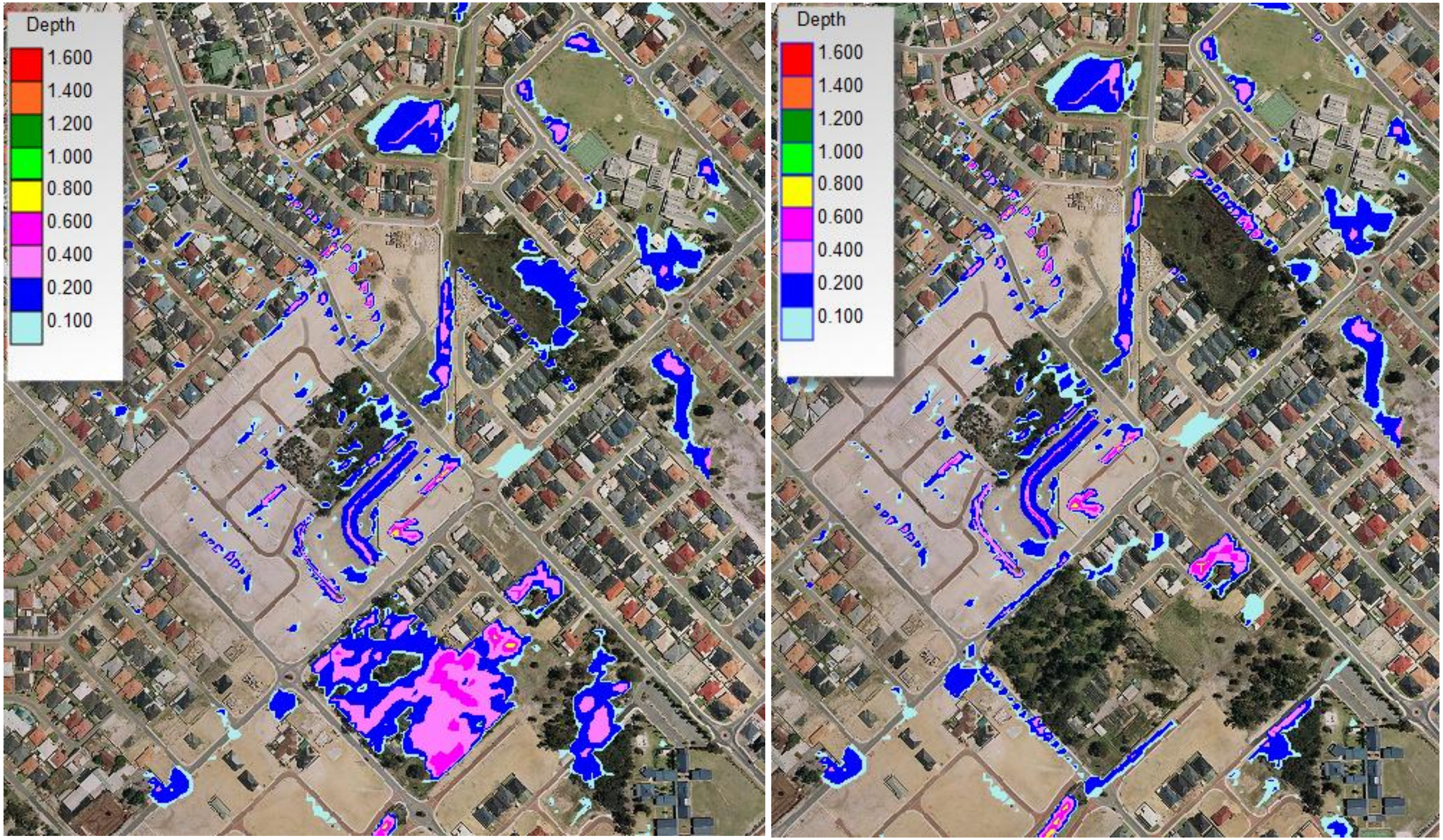
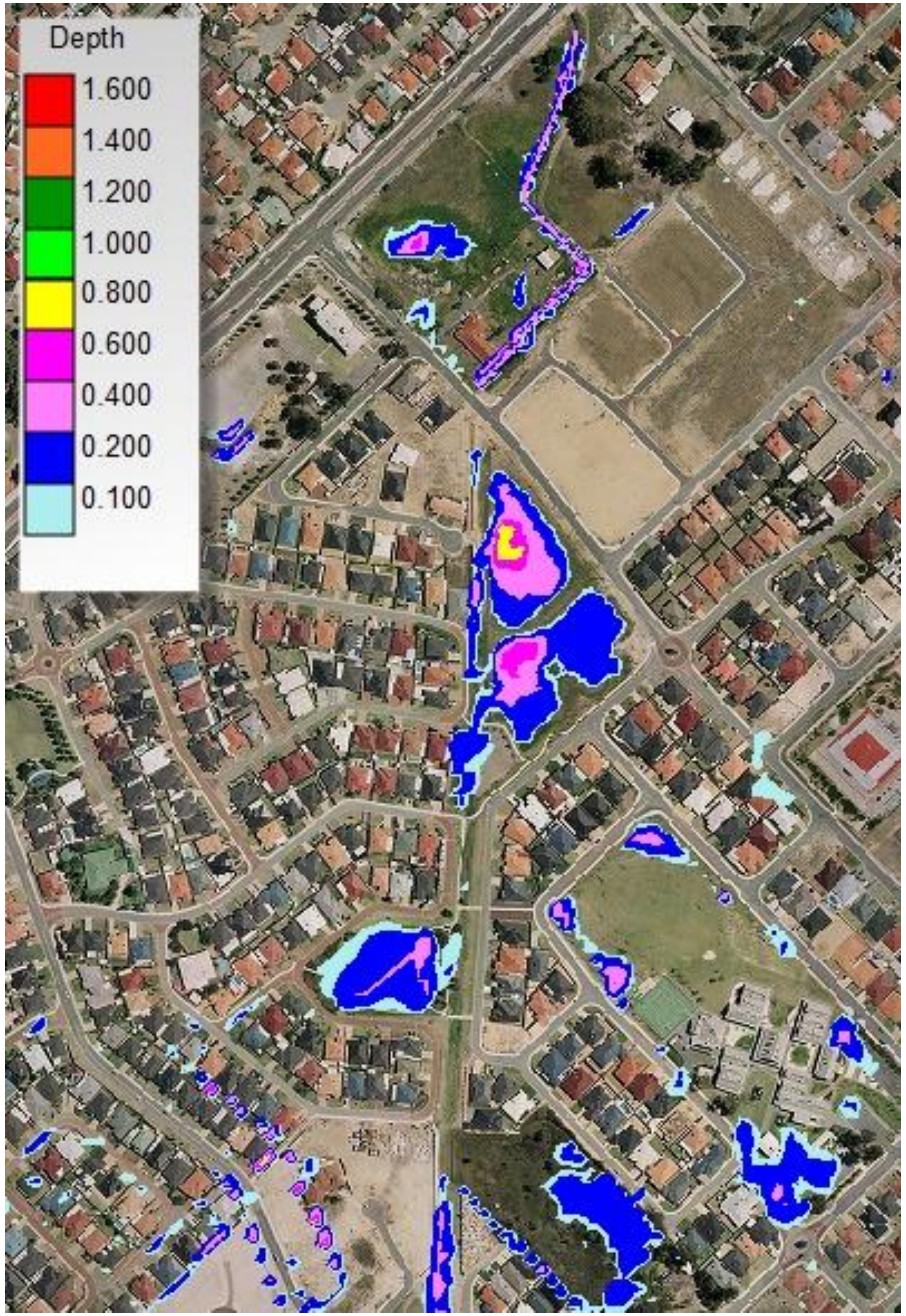


Figure 51. Main Drain 10 year ARI pre and post development flood inundation map #2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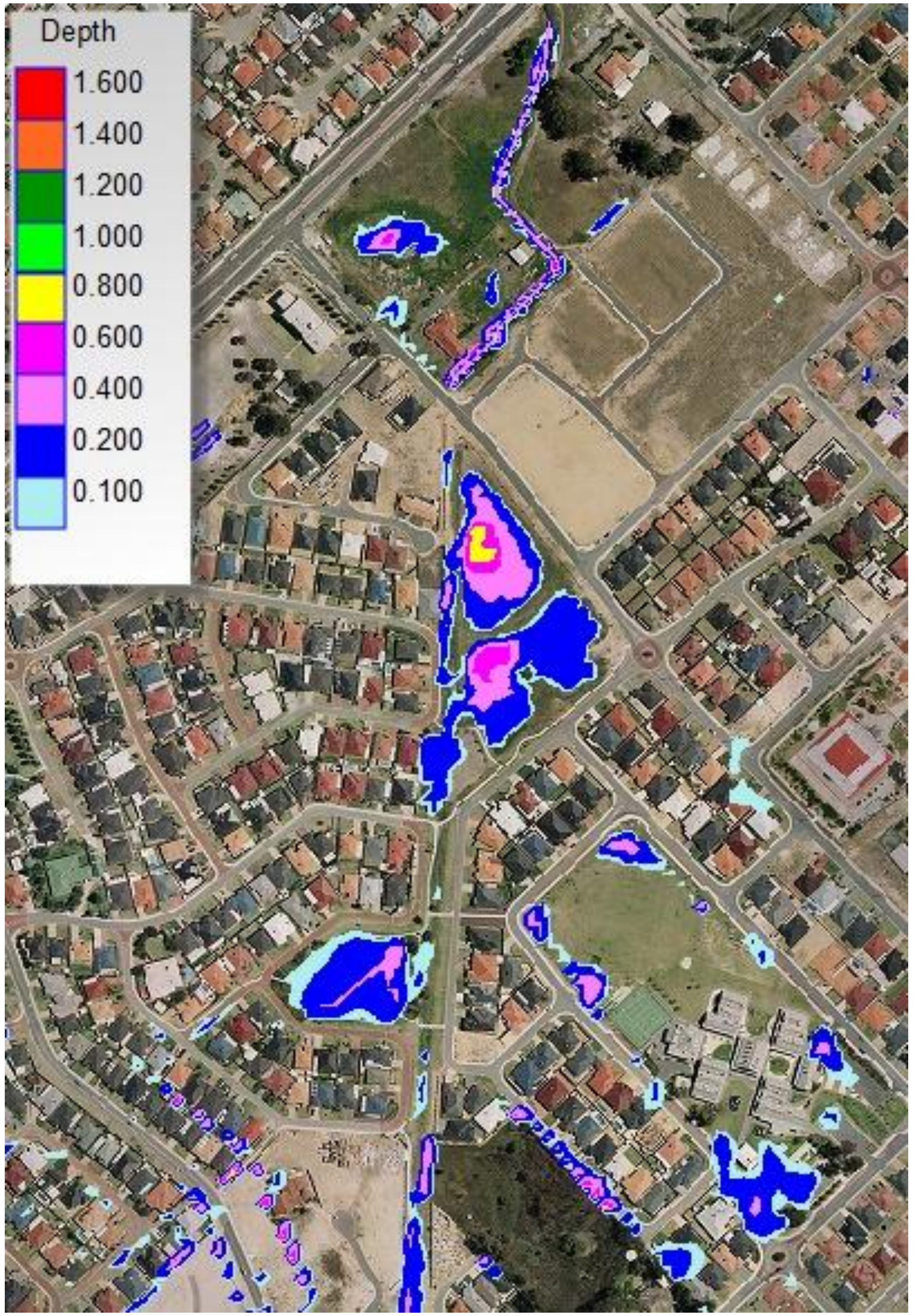
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Figure 52. Main Drain 10 year ARI pre and post development flood inundation map #3

Figure 53. Main Drain 100 year ARI pre and post development flood inundation map #1

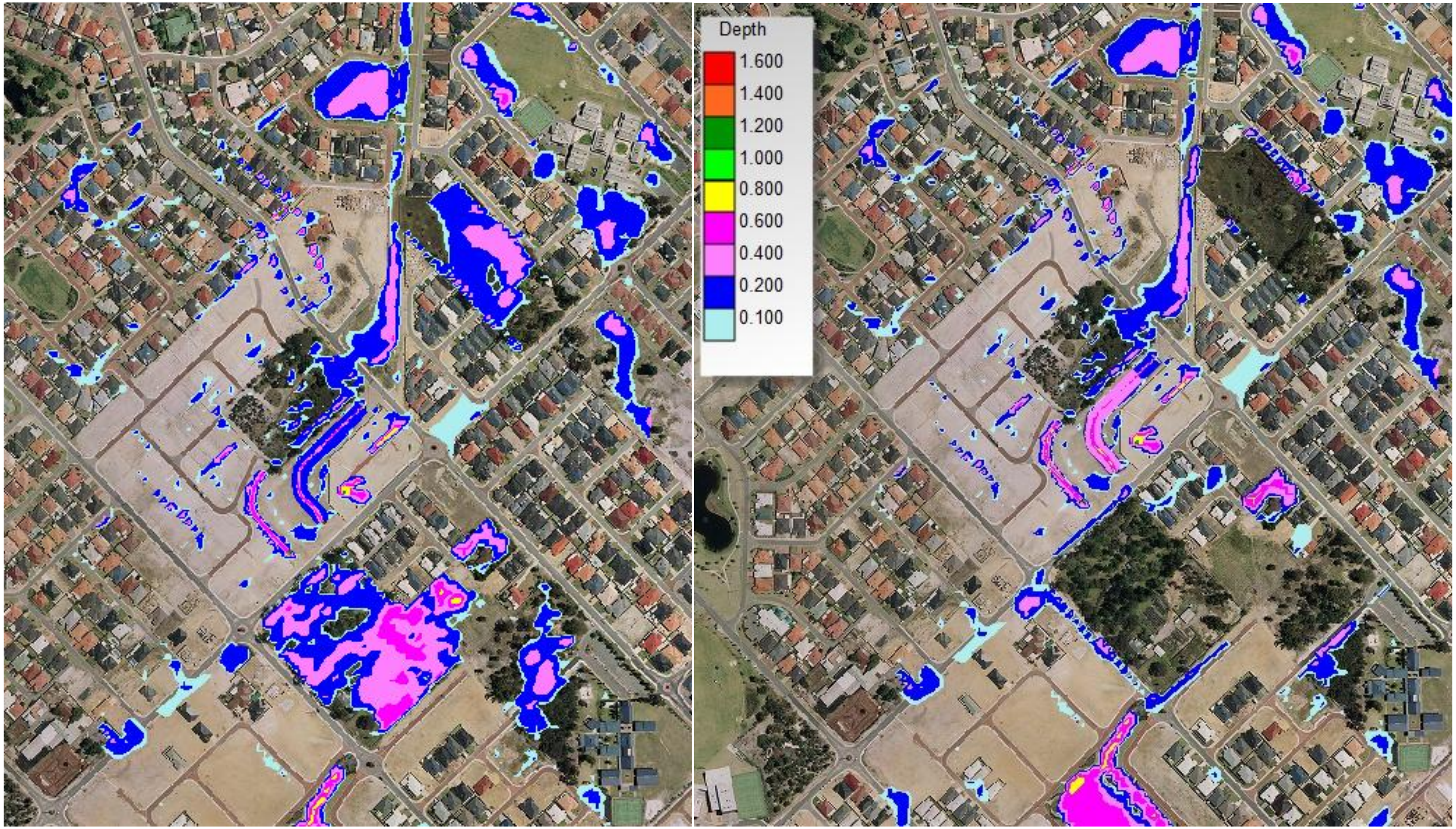

Figure 54. Main Drain 100 year ARI pre and post development flood inundation map #2

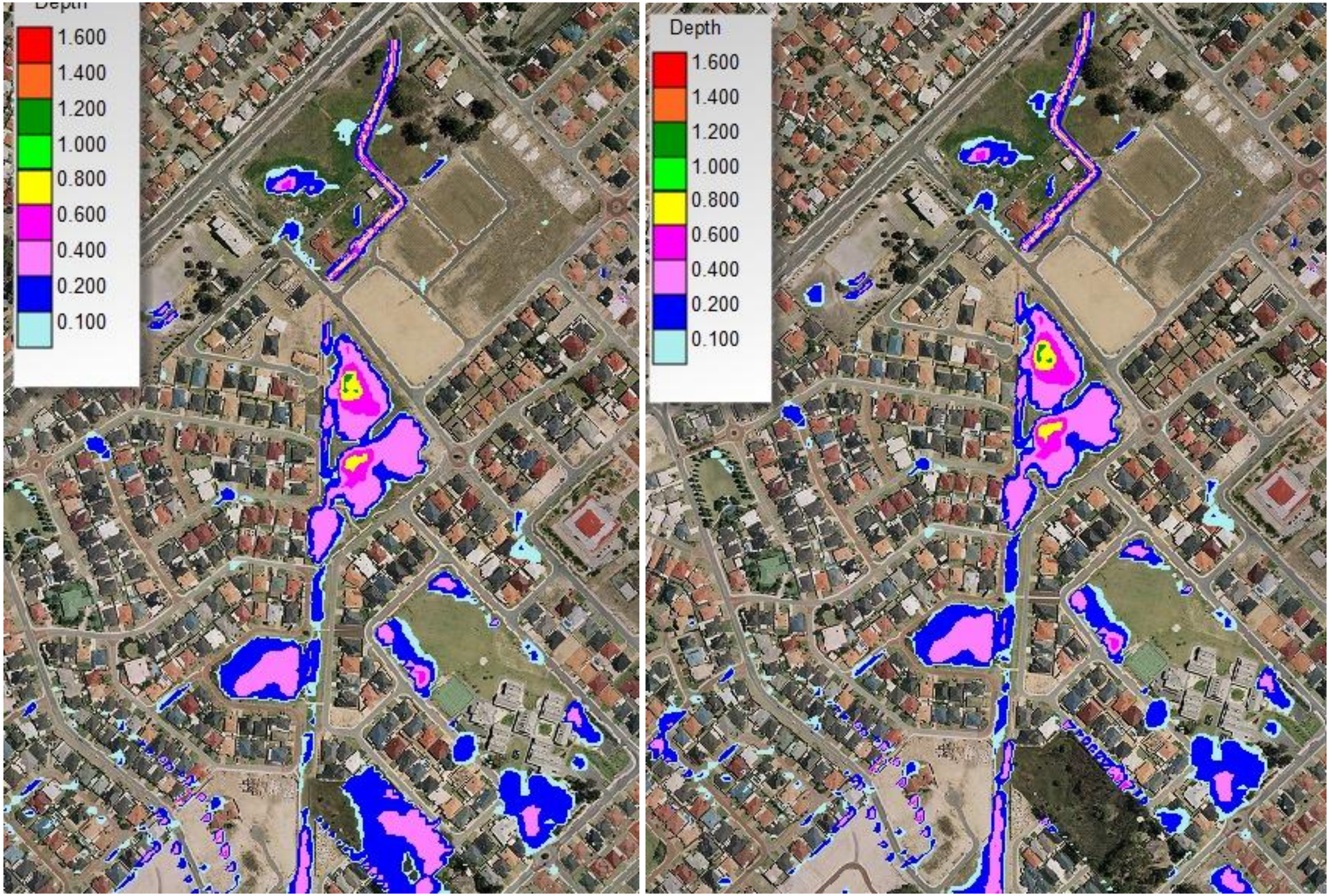

Figure 55. Main Drain 100 year ARI pre and post development flood inundation map #3