

# EVALUATION OF THE USE OF FLOOD ATTENUATION CONTROLS FOR THE MANAGEMENT OF URBAN STORMWATER IMPACTS IN CAPE TOWN, SOUTH AFRICA

Timothy Stephen Hotchkiss

*Thesis presented in partial fulfilment of the requirements for  
the degree of Master of Science in Engineering in Civil Engineering at  
Stellenbosch University*



Supervisor: Prof GR Basson *PrEng PhD*

Stellenbosch University  
Department of Civil Engineering  
Private Bag X1, MATIELAND, 7602, South Africa

March 2015

## Declaration

By submitting this thesis, I declare that the entirety of the work contained therein is my own, original work, that I am the sole author thereof (save to the extent explicitly otherwise stated), that reproduction and publication thereof by Stellenbosch University will not infringe any third party rights and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

Timothy S. Hotchkiss

March 2015

Copyright © 2015 Stellenbosch University

All rights reserved

## Acknowledgements

The author thankfully acknowledges the following people and institutions for their input and assistance in the compilation of this thesis:

**Prof Basson** for his feedback and guidance regarding this thesis.

**AECOM** for providing me with a bursary to fund these studies and for allowing me to use some stormwater models, previously compiled by the author and others while employed by AECOM, as a basis for some of the hydrological and hydraulic assessments undertaken for this study.

**Isak Malherbe** and **Aldu le Grange** for mentorship and guidance in the field of floods and stormwater during my time working at AECOM.

**Mike Wiese** for sharing his knowledge of the Upper Kuils River stormwater system and for assisting with the translation of my abstract.

**Sipho Tefu** for his work done on the Upgrading of the Bayside Canal project which provided a basis for the Bayside Canal Catchment modelling undertaken in this study.

**The City of Cape Town** for allowing me to use some stormwater models commissioned by the City of Cape Town as a basis for some of the hydrological and hydraulic assessments undertaken for this study.

**Ben de Wet** and **Rod Arnold** of the City of Cape Town for their support of this research and for providing the necessary information and data where needed.

**My family** and **friends** for their unconditional love and support, and for showing interest in my passion for engineering and flood control.

**Jade**, my wife and best friend, for her unconditional love and support during these studies, and her invaluable contribution in assisting with the editing of this thesis.

**Jesus** for His unfailing love, strength and provision.

## Abstract

In the context of rapidly expanding cities, it is imperative that urban planning in South Africa has sufficient guidance regarding stormwater and river corridor management, in order to provide solutions that address issues of flood risk and the environmental health of river systems. Attenuation of stormwater runoff, the focus of this study, is one of the most important structural mechanisms used for the mitigation of many of the negative impacts caused by uncontrolled urban runoff. Typically, it involves the use of attenuation ponds or wetlands, which temporarily store runoff during a storm and release flow downstream at a reduced rate so as to mimic natural flow patterns.

The focus of urban stormwater management and flood control has historically been on the protection of human life and property. However, in recent decades, through growing environmental awareness and the advancement of the concept of sustainable development, urban stormwater management has become a growing field of research worldwide, with a broader focus which considers not only flood control, but also water quality, aquatic biodiversity and the amenity value of urban drainage systems. Flood attenuation controls are becoming more widely used within South African urban areas, primarily due to policies or legislation brought into effect by local authorities. However, there is often little understanding regarding the positive and perhaps negative effects that these attenuation controls are having on receiving watercourses downstream.

Three case studies were assessed by means of stormwater modelling simulations to evaluate various flood attenuation practices which are currently in use in South Africa. Two of the study areas, the Mosselbank River Catchment and the Bayside Canal Catchment, were selected in areas of Cape Town where future development has been proposed by spatial planners. The third study area, the Upper Kuils River Catchment, was evaluated in terms of the performance of existing attenuation facilities in an area which is already almost completely developed. The study found that attenuation facilities constructed with a single culvert-type outlet structure, designed to reduce flows during large storm events, do not mitigate the impact of post-development runoff occurring during lower recurrence interval storm events. Attenuation facilities with multi-stage outlet structures were found to be much more effective at mimicking pre-development flow during a range of storm events. It was also found that because attenuation does not reduce post-development runoff volumes to pre-development levels, but merely reduces peak flow rates, the cumulative runoff from multiple attenuation controls across a large (>30 km<sup>2</sup>) urban catchment resulted in higher runoff peaks in downstream watercourses.

The study concluded that more widespread use of stormwater Best Management Practices (BMPs) and Sustainable Drainage System (SuDS) controls allows a greater portion of runoff to infiltrate, resulting in less runoff volume and therefore reduced peak flows downstream, especially during low recurrence interval storm events. In addition, the study recommended the use of detailed catchment-wide stormwater modelling to understand specific catchment dynamics holistically, thus increasing the potential for designing effective attenuation controls in urban stormwater systems.

## Opsomming

In die konteks van die vinnige tempo van stedelike uitbreiding, is dit noodsaaklik dat stedelike beplanning in Suid-Afrika plaasvind met in aggenome van voldoende riglyne vir die bestuur van stormwater en rivierkorridors, ten einde oplossings te vind vir die kwessies van vloedrisiko en die omgewingsgesondheid van rivierstelsels. Vloedvertraging, wat die fokus van hierdie studie is, is een van die belangrikste strukturele meganismes wat gebruik word vir die verligting van talle negatiewe impakte wat veroorsaak word deur onbeheerde stormwaterafloop in stedelike gebiede. Tipies behels dit die gebruik van vloedvertragingsdamme of vleilande, wat afloop vertraag tydens 'n storm en dus vloei stroom-af teen 'n verlaagde tempo uitlaat met die doel om natuurlike vloeioptritte na te boots.

Die fokus van stedelike stormwaterbestuur en vloedbeheer was in die verlede hoofsaaklik op die beskerming van lewe en eiendom gefokus, maar het egter die afgelope dekades verskuif na water gehalte, die biodiversiteit van waterekosisteme en die geriefswaarde van stedelike dreineringsstelsels. Hierdie verskuiwing van fokus is weens die groeiende omgewingsbewustheid en die bevordering van die konsep van volhoubare ontwikkeling wat wêreldwyd 'n groter navorsingsgebied geraak het. Vloedvertraging beheermeganismes word al hoe meer algemeen gebruik in Suid-Afrikaanse stedelike gebiede, hoofsaaklik as gevolg van die beleide of wetgewing wat deur plaaslike owerhede in werking gestel is. Daar is egter dikwels min begrip vir die positiewe en moontlike negatiewe gevolge wat hierdie vertragingsmeganismes op stroom-af sisteme het.

Drie gevallestudies is geëvalueer deur middel van numeriese modelstudies wat verskeie benaderings van vloed beheer, wat tans in Suid-Afrika gebruik is, in ag neem. Twee van die studie areas, naamlik die Mosselbank en die Bayside-kanaal opvanggebiede in die Kaapse metropool, is gekies in areas waar toekomstige ontwikkeling in die vooruitsig gestel is deur stadsbeplanners. Die derde studie area, die opvangsgebied van die bolope van die Kuilsrivier, is in terme van die prestasie van bestaande stormwater infrastruktuur in 'n gebied wat reeds byna heeltemal ontwikkel is, geëvalueer. Die studie het bevind dat vloedvertragingsfasiliteite met 'n enkele duiker uitlaatstruktuur, wat ontwerp is met die doel om die vloeioptritte tydens groot storms te demp, nie die impak van die na-ontwikkeling afloop, wat gedurende storms met laer herhalingsinterval voorkom, verminder nie. In terme van vloedvertragingsfasiliteite met 'n veelvuldige uitlaatstruktuur, is dit bevind dat voorontwikkelingsafloop tydens 'n reeks van groot en kleiner storms veel meer effektief nageboots word. Daar is egter ook bevind dat die demping van die vloedoptritte nie die na-ontwikkeling afloopvolumes verminder tot voorontwikkelingsvlakke nie, maar slegs tot die vermindering van maksimum snelhede lei. Die gevolg is dat die totale afloop van 'n kombinasie van 'n aantal vertragingsdamme oor 'n groot (> 30 km<sup>2</sup>) stedelike opvanggebied 'n hoër spitsvloei in die stroom-af riviere tot gevolg het.

Die studie het bevind dat die wydverspreide gebruik van bestebestuurspraktyke (BMPs) en volhoubare stedelike dreineringsstelsels (SuDS) tot die infiltrasie van 'n groter gedeelte van die afloop lei, wat laer afloopvolume en dus verminderde spitsvloei stroomaf tot gevolg het, veral gedurende storms met 'n lae herhalingsinterval. Daarbenewens word die aanwending van gedetailleerde modellering van stormwatersisteme binne die groter opvangsgebied aanbeveel ten einde 'n meer holistiese begrip van spesifieke aspekte van die opvanggebied dinamika, om sodoende die potensiaal vir die ontwerp van effektiewe vloedvertragingskontroles in stedelike stormwaterstelsels te verbeter.

**MASTERS THESIS 2014****EVALUATION OF THE USE OF FLOOD ATTENUATION CONTROLS FOR THE MANAGEMENT OF  
URBAN STORMWATER IMPACTS IN CAPE TOWN, SOUTH AFRICA****TABLE OF CONTENTS**

	<u>Page</u>
Declaration	i
Acknowledgements	ii
Abstract	ii
Opsomming	iv
Table of Contents	v
List of Appendices	vii
List of Figures	viii
List of Tables	xii
List of Abbreviations	xiv
<b>1. Introduction</b>	<b>1</b>
1.1 Background	1
1.2 Research objectives	2
1.3 Methodology	2
1.4 Structure of thesis	3
<b>2. Literature review</b>	<b>4</b>
2.1 Introduction	4
2.2 Impacts of post-development stormwater runoff	4
2.2.1 Changes to stream flow	4
2.2.2 Changes to stream geometry	6
2.2.3 Environmental impact	6
2.2.4 Resulting impacts on communities	6
2.3 Historical context to stormwater management	7
2.4 Addressing runoff impacts through attenuation	8
2.5 Sustainable drainage systems	10
2.6 Stormwater regulatory requirements in South Africa	12
2.6.1 National legislation	12
2.6.2 Guidelines for Human Settlement Planning and Design (The Red Book)	13
2.6.3 City of Cape Town	14
2.6.4 City of Johannesburg	17
2.6.5 eThekweni Municipality	18
2.7 International stormwater attenuation regulations and guidelines	19
2.7.1 US guidelines and legislation	19
2.7.2 General floods and stormwater management in Europe	22
2.7.3 UK guidelines and policies	23
2.7.4 Australian guidelines and policies	24
2.8 Technical considerations for implementation of attenuation practices	26
2.8.1 Pond sizing parameters	26
2.8.2 Pond outlet design	27
2.8.3 On-site versus regional attenuation	31
2.8.4 Runoff timing and catchment dynamics	33
2.8.5 Cumulative effect of catchment-wide attenuation	34
2.8.6 Downstream assessments	36

2.9	A paradigm shift for attenuation controls in South Africa .....	37
2.9.1	Current flood attenuation practices in South Africa .....	37
2.9.2	Design of attenuation facilities for control of multiple storm events.....	38
2.9.3	Implementation of catchment-wide modelling .....	39
<b>3.</b>	<b>Case studies.....</b>	<b>40</b>
3.1	Introduction .....	40
3.2	Selection of study areas .....	40
3.2.1	Mosselbank River Catchment – Kraaifontein/Klipheuwel .....	41
3.2.2	Bayside Canal Catchment – Blaauwberg .....	44
3.2.3	Upper Kuils River Catchment – Durbanville .....	45
3.3	Selection of hydraulic modelling software .....	46
3.4	Modelling parameters .....	48
3.4.1	Calibration data limitations.....	48
3.4.2	Sub-catchment delineation .....	49
3.4.3	Infiltration and interception.....	50
3.4.4	Sub-catchment slopes .....	51
3.4.5	Sub-catchment width .....	52
3.4.6	Percent imperviousness.....	52
3.4.7	Rainfall .....	53
3.4.8	Watercourse geometry.....	66
3.4.9	Attenuation routing.....	66
3.5	Mosselbank River Model verification .....	67
3.5.1	Available logging data.....	67
3.5.2	Peak flow verification using deterministic flood hydrology methods.....	71
3.5.3	Previous flood studies conducted for the Mosselbank River.....	74
3.5.4	Mosselbank River Model parameter sensitivity analysis .....	77
3.6	Bayside Canal Model verification .....	83
3.6.1	Peak flow verification using deterministic flood hydrology methods.....	83
3.6.2	Previous flood studies conducted for the Bayside Canal .....	85
3.6.3	Bayside Canal Model parameter sensitivity analysis.....	86
3.7	Upper Kuils River Model verification.....	89
3.8	Attenuation scenarios .....	89
3.9	Mosselbank River Model results.....	93
3.9.1	Attenuation results at sub-catchment level.....	93
3.9.2	Mosselbank River Model points of interest.....	94
3.9.3	Mosselbank River design storm modelling results .....	96
3.9.4	Mosselbank River “continuous” modelling results .....	99
3.10	Bayside Canal Model results .....	103
3.10.1	Attenuation results at sub-catchment level.....	103
3.10.2	Bayside Canal Model points of interest .....	104
3.10.3	Bayside Canal design storm modelling results .....	106
3.10.4	Bayside Canal “continuous” modelling results.....	108
3.11	Upper Kuils River Model results .....	112
3.11.1	Attenuation results at sub-catchment level.....	112
3.11.2	Upper Kuils River Model points of interest.....	112
3.11.3	Upper Kuils River design storm modelling results .....	114
3.11.4	Upper Kuils River “continuous” modelling results .....	115
3.12	Summary of the modelling scenario results.....	120
3.12.1	Scenario 1 .....	120
3.12.2	Scenario 2.....	120
3.12.3	Scenario 3.....	121
3.12.4	Scenario 4.....	121
3.12.5	Scenario 5.....	122
<b>4.</b>	<b>Discussion of findings .....</b>	<b>123</b>
4.1	Findings from the Mosselbank River case study.....	123
4.1.1	Ineffective attenuation with single-stage outlet structures .....	123
4.1.2	Cumulative effect of increase in runoff volume.....	123
4.1.3	Changes to catchment dynamics caused by attenuation .....	123

4.1.4	Natural attenuation in the river floodplain .....	124
4.1.5	Poor results for small storm events .....	124
4.2	Findings from the Bayside Canal case study .....	124
4.2.1	Value of constructed “natural” systems .....	124
4.2.2	Limited effect of increased runoff volumes .....	124
4.3	Findings from the Upper Kuils River case study .....	125
4.3.1	Limited effectiveness of current attenuation facilities .....	125
4.3.2	Addressing flooding and erosion with additional BMPs and SuDS controls.....	125
<b>5.</b>	<b>Conclusions .....</b>	<b>126</b>
<b>6.</b>	<b>Recommendations for further research .....</b>	<b>129</b>
6.1	Case studies in other regions of South Africa .....	129
6.2	Modelling parameters and calibration data.....	129
6.3	Research of other SuDS techniques .....	129
6.4	Urban drainage guidelines for South Africa.....	130
6.5	Economic implications of various flood attenuation controls .....	130
	<b>References .....</b>	<b>131</b>

**Appendices (included separately):**

Appendix A: Flood hydrology calculations and results

Appendix B: Model layouts

Appendix C: Mosselbank River Model results

Appendix D: Bayside Canal Model results

Appendix E: Upper Kuils River Model results

## List of Figures

Figure 2.1: Typical characteristics of stormwater runoff in natural and urban catchments (adapted from Melbourne Water Draft Stormwater Strategy (Melbourne Water, 2012)) .....	5
Figure 2.2: Typical hydrographs for a catchment pre-development, post-development, and post-development with attenuation.....	8
Figure 2.3: Typical detention (dry) pond in the Brackenfell area, Cape Town .....	9
Figure 2.4: Typical retention (wet) pond in the Durbanville area, Cape Town.....	10
Figure 2.5: Sustainable drainage systems (adapted from Armitage et al., 2013b) .....	11
Figure 2.6: Single culvert-type outlet of a detention pond in the Blaauwberg area .....	28
Figure 2.7: Detention pond outlet structure in Kraaifontein area.....	28
Figure 2.8: Retention pond outlet in Durbanville area .....	29
Figure 2.9: Typical attenuation pond (adapted from the Knox County Stormwater Management Manual).....	30
Figure 2.10: Multi-level outlet structure in Sunningdale area, Blaauwberg.....	31
Figure 2.11: Multi-leveled outlet structure in Atlantis .....	31
Figure 2.12: On-site versus regional attenuation .....	32
Figure 2.13: Effect of attenuation on runoff timing .....	35
Figure 2.14: Cumulative effect of multiple tributaries with attenuation .....	36
Figure 3.1: Future growth corridors for Cape Town as proposed by the CCT (City of Cape Town, 2012a) .....	42
Figure 3.2: Locality plan of the three case study catchments (Mosselbank River, Bayside Canal and Upper Kuils River).....	43
Figure 3.3: 24-hour 2-year recurrence interval rainfall depths adopted for the Mosselbank River Model.....	54
Figure 3.4: 24-hour 10-year recurrence interval rainfall depths adopted for the Mosselbank River Model.....	55
Figure 3.5: 24-hour 50-year recurrence interval rainfall depths adopted for the Mosselbank River Model.....	56
Figure 3.6: 24-hour 2-year recurrence interval rainfall depths adopted for the Upper Kuils River Model.....	57
Figure 3.7: 24-hour 10-year recurrence interval rainfall depths adopted for the Upper Kuils River Model.....	58

Figure 3.8: 24-hour 50-year recurrence interval rainfall depths adopted for the Upper Kuils River Model..... 59

Figure 3.9: 24-hour 2-year recurrence interval rainfall depths adopted for the Bayside Canal Model60

Figure 3.10: 24-hour 10-year recurrence interval rainfall depths adopted for the Bayside Canal Model ..... 61

Figure 3.11: 24-hour 50-year recurrence interval rainfall depths adopted for the Bayside Canal Model ..... 62

Figure 3.12: City of Cape Town and SA Weather Service Rainfall Stations..... 64

Figure 3.13: 24-hour rainfall depths at the Kraaifontein Roads Depot ..... 65

Figure 3.14: 24-hour rainfall depths at the Tygerberg Reservoir ..... 65

Figure 3.15: 24-hour rainfall depths at the Potsdam Wastewater Treatment Works..... 65

Figure 3.16: Typical multi-stage outlet structure used for modelling purposes ..... 67

Figure 3.17: Location of Department of Water Affairs Flow Gauge G2H013 ..... 68

Figure 3.18: Flow Gauge G2H013 rating curve ..... 69

Figure 3.19: Statistical plot for Flow Gauge G2H013 using Log Pearson Type 3 distribution..... 70

Figure 3.20: Catchments used for verification purposes using deterministic flood hydrology methods ..... 72

Figure 3.21: Log-log plot comparison of flood hydrology results for the Upper Mosselbank River (Catch-M3) ..... 76

Figure 3.22: Log-log plot comparison of flood hydrology results for the Mosselbank River (Catch-M5) ..... 76

Figure 3.23: Catchments used for verification purposes using deterministic flood hydrology methods ..... 84

Figure 3.24: Mosselbank River Model ..... 95

Figure 3.25: Comparison of peak flows at Location M-1 ..... 96

Figure 3.26: Comparison of peak flows at Location M-2 ..... 97

Figure 3.27: Comparison of peak flows at Location M-3 ..... 97

Figure 3.28: Comparison of peak flows at Location M-4 ..... 97

Figure 3.29: Comparison of peak flows at Location M-5 ..... 98

Figure 3.30: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 28 and 31 August 2013..... 100

Figure 3.31: Flow hydrograph at M-2 following heavy rainfall of 28 August 2013 ..... 100

Figure 3.32: Flow hydrograph at M-5 following heavy rainfall of 28 August 2013 ..... 100

Figure 3.33: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 12 and 16 August 2013 ..... 101

Figure 3.34: Flow hydrograph at M-2 following rainfall between 12 and 16 August 2013 ..... 101

Figure 3.35: Flow hydrograph at M-5 following rainfall between 13 and 16 August 2013 ..... 101

Figure 3.36: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 1 and 4 June 2013..... 102

Figure 3.37: Flow hydrograph at M-2 following rainfall between 1 and 4 June 2013 ..... 102

Figure 3.38: Flow hydrograph at M-5 following rainfall between 1 and 4 June 2013 ..... 102

Figure 3.39: Bayside Canal Model..... 105

Figure 3.40: Comparison of peak flows at Location B-1 ..... 106

Figure 3.41: Comparison of peak flows at Location B-2 ..... 107

Figure 3.42: Comparison of peak flows at Location B-3 ..... 107

Figure 3.43: Comparison of peak flows at the Bayside Canal outfall ..... 107

Figure 3.44: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 28 and 31 August 2013 ..... 109

Figure 3.45: Flow hydrograph at B-2 following heavy rainfall of 28 August 2013..... 109

Figure 3.46: Flow hydrograph at B-3 following heavy rainfall of 28 August 2013..... 109

Figure 3.47: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 13 and 16 August 2013 ..... 110

Figure 3.48: Flow hydrograph at B-2 following rainfall between 13 and 16 August 2013 ..... 110

Figure 3.49: Flow hydrograph at B-3 following rainfall between 13 and 16 August 2013 ..... 110

Figure 3.50: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 1 and 4 June 2013 ..... 111

Figure 3.51: Flow hydrograph at B-2 following rainfall between 1 and 4 June 2013 ..... 111

Figure 3.52: Flow hydrograph at B-3 following rainfall between 1 and 4 June 2013 ..... 111

Figure 3.53: Upper Kuils River bulk stormwater system..... 113

Figure 3.54: Comparison of peak flows in the Upper Kuils River at De Villiers Drive ..... 114

Figure 3.55: Comparison of peak flows in the Upper Kuils River at the N1..... 115

Figure 3.56: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 28 and 31 August 2013..... 117

Figure 3.57: Flow hydrograph at De Villiers Drive following heavy rainfall of 28 August 2013 ..... 117

Figure 3.58: Flow hydrograph at B-3 following heavy rainfall of 28 August 2013..... 117

Figure 3.59: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 12 and 16 August 2013..... 118

Figure 3.60: Flow hydrograph at De Villiers Drive following rainfall between 13 and 16 August 2013 ..... 118

Figure 3.61: Flow hydrograph at the N1 following rainfall between 13 and 16 August 2013..... 118

Figure 3.62: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 1 and 4 June 2013..... 119

Figure 3.63: Flow hydrograph at De Villiers Drive following rainfall between 1 and 4 June 2013..... 119

Figure 3.64: Flow hydrograph at the N1 following rainfall between 1 and 4 June 2013..... 119

## List of Tables

Table 2.1: Summary of the Unified Stormwater Sizing Criteria (adapted from the GSMM) .....	22
Table 3.1: Hydrological modelling software packages.....	47
Table 3.2: Infiltration parameters.....	51
Table 3.3: Runoff parameters .....	51
Table 3.4: Stormwater catchment land use characteristics .....	52
Table 3.5: Annual maximum time series for Flow Gauge G2H013 .....	69
Table 3.6: Comparison of results from statistical analysis and PCSWMM model .....	71
Table 3.7: Deterministic method input parameters .....	73
Table 3.8: Results of flood hydrology using deterministic methods .....	74
Table 3.9: Results of the Africon 2001 and Bergstan 2009 studies .....	75
Table 3.10: Test for sensitivity of Manning roughness coefficient.....	78
Table 3.11: Test for sensitivity of catchment width.....	79
Table 3.12: Test for sensitivity of infiltration rate – pre-development scenario.....	80
Table 3.13: Test for sensitivity of infiltration rate – post-development scenario .....	81
Table 3.14: Test for sensitivity of % imperviousness – post-development scenario.....	82
Table 3.15: Deterministic method input parameters .....	83
Table 3.16: Results of flood hydrology using deterministic methods .....	85
Table 3.17: Test for sensitivity of Manning roughness – post-development scenario.....	86
Table 3.18: Test for sensitivity of catchment width – post-development scenario .....	87
Table 3.19: Test for sensitivity of infiltration rate – pre-development scenario.....	87
Table 3.20: Test for sensitivity of infiltration rate – post-development scenario .....	88
Table 3.21: Test for sensitivity of percent imperviousness – post-development scenario .....	88
Table 3.22: Impact of BMPs on stormwater runoff parameters .....	91
Table 3.23: Comparison of stormwater runoff parameters for various scenarios.....	91
Table 3.24: Comparison of sub-catchment post-development and pre-development peak runoff ....	93
Table 3.25: Mosselbank River Model points of interest .....	94
Table 3.26: Summary of Mosselbank River design storm modelling results .....	96

Table 3.27: Comparison of sub-catchment post-development and pre-development peak runoff ..	104
Table 3.28: Bayside Canal Model points of interest .....	104
Table 3.29: Summary of Bayside Canal design storm modelling results .....	106
Table 3.30: Upper Kuils River Model points of interest .....	112
Table 3.31: Summary of Upper Kuils River design storm modelling results .....	114

## List of Abbreviations

BMPs	Best Management Practices
CCT	City of Cape Town
CHI	Computational Hydraulics International
CJ	City of Johannesburg
CSIR	Council for Scientific and Industrial Research
DEFRA	Department for Environment, Food and Rural Affairs
DHI	Danish Hydraulic Institute
EC	European Commission
EISA	Energy Independence and Security Act
EPA	Environmental Protection Agency
FEMA	Federal Emergency Management Agency
GSMM	Georgia Stormwater Management Manual
GUI	Graphical User Interface
LID	Low-Impact Development
NEMA	National Environmental Management Act
NFIP	National Flood Insurance Program
NWA	National Water Act
SANRAL	South African Roads Agency Limited
SWMM5	Stormwater Management Model Version 5
SWMPs	surface water management plans
SuDS	Sustainable Drainage Systems
USACE	United States Army Corps of Engineers
WSUD	Water Sensitive Urban Design

## 1. Introduction

### 1.1 Background

The growth of South African cities is a trend which continues to put pressure on government to provide more infrastructure and expand the cities' urban edges to accommodate a growing urban population. The shortage of affordable formal accommodation in urban areas remains a problem despite the South African Government's ongoing roll-out of low-cost housing. Private developers have also been proactive in purchasing potential sites for residential development to accommodate the increasing middle-class population of South Africa. In addition, there is pressure to develop the commercial and industrial sectors to provide job opportunities for an increasing urban work force.

From a stormwater catchment management perspective, the expansion of South African cities is resulting in new development in catchments which are predominantly undeveloped at present. As a consequence of urban development, the increase in impervious areas results in a number of changes in the storm runoff response. Undeveloped land intercepts a large portion of rainfall which infiltrates into the ground replenishing groundwater reserves and reducing the overland flow that reaches the receiving watercourses. In contrast, built-up land provides a relatively hard, smooth surface for rainfall to flow off with minimal interception or infiltration, resulting in more frequent flooding and reduced base flows in the receiving watercourses (Konrad, 2003).

The impact that urban development has on local river systems has become a great concern from both an environmental and human safety point of view, especially when these impacts can potentially propagate downstream. Today, most national governing authorities worldwide have implemented regulations which provide a legal basis for how stormwater should be managed and controlled in order to mitigate potential negative impacts. Generally this is achieved through the implementation of a variety of structural and non-structural measures, informed by policies or guidelines, and enforced by legislation.

At this stage, it is imperative that urban planning in South Africa has sufficient guidance regarding stormwater and river corridor management, in order to provide solutions that address issues of flood risk and the environmental health of river systems. Attenuation of stormwater runoff, the focus of this particular study, is one of the most important structural mechanisms used for the mitigation of many of the negative impacts caused by uncontrolled urban runoff. It typically involves the use of attenuation ponds or wetlands, which temporarily store runoff during a storm and release flow downstream at a reduced rate so as to mimic natural flow patterns.

## 1.2 Research objectives

The focus of urban stormwater management and flood control has historically been on the protection of human life and property. However, in recent decades, through growing environmental awareness and the advancement of the concept of sustainable development, urban stormwater management has become a growing field of research worldwide, with a broader focus which considers not only flood control, but also water quality, aquatic biodiversity and the amenity value of urban drainage systems. However, South African literature still contains very little guidance when it comes to implementing an urban stormwater system that aims to mitigate the impacts of development and mimic natural flow regimes.

Flood attenuation controls are becoming more widely used within South African urban areas, primarily due to policies or legislation brought into effect by local authorities, however, there is often little understanding regarding the positive and perhaps negative effects that these attenuation controls are having on receiving watercourses downstream. The purpose of this study is therefore to critically evaluate current flood attenuation practices in South Africa, and to investigate various technical concepts which need to be taken into account when implementing attenuation as part of an urban stormwater network that aims to mimic natural flood hydrology in downstream watercourses.

## 1.3 Methodology

This research consists firstly of a literature review which summarises much of the international and local literature regarding flood control in urban areas, especially related to the use of flood attenuation. A portion of the literature review focuses on the relevant legislation, policies and design guidelines currently in use in South Africa. For comparative purposes some of the more comprehensive urban drainage manuals, guidelines, policies and legislation from other countries, which are known to be at the forefront of research in the field of stormwater management, are also discussed.

The second component of this research consists of three case studies which were assessed by means of stormwater modelling simulations to evaluate various flood attenuation practices which are currently in use in South Africa. Two of the study areas, the Mosselbank River Catchment and the Bayside Canal Catchment, were selected in areas of Cape Town where future development has been proposed by spatial planners. The third study area, the Upper Kuils River Catchment, was evaluated in terms of the performance of existing attenuation facilities in an area which is already almost completely developed. The models were compiled using the PCSWMM stormwater modelling software developed by Computational Hydraulics International (CHI), which uses the United States (US) Environmental Protection Agency's (EPA) Stormwater Management Model Version 5 (SWMM5) modelling engine.

## 1.4 Structure of thesis

This thesis consists of six chapters and is structured as follows:

- **Chapter 1** presents the introductory context of the research presented in this thesis in terms of background, research objectives, methodology and the structure of the thesis.
- **Chapter 2** is a literature review which summarises some of the most pertinent literature available regarding the topic of urban stormwater management, particularly as it relates to mitigating urban stormwater impacts through the implementation of runoff attenuation controls.
- **Chapter 3** presents three case studies which investigate and evaluate various attenuation scenarios by means of stormwater modelling. This chapter outlines the methodology behind the selection of the study areas and the modelling software package, the modelling parameters used, and the results of the various modelling simulations.
- **Chapter 4** discusses the findings from the case studies and how stormwater attenuation controls can be used most effectively in addressing urban stormwater impacts.
- **Chapter 5** presents the conclusions based on the key findings of this research.
- **Chapter 6** provides recommendations for further research into the implementation of effective attenuation controls in South African urban areas.

References consulted for the purpose of this research are provided thereafter.

## 2. Literature review

### 2.1 Introduction

This chapter provides a review of some of the available literature regarding the topic of urban stormwater management, particularly as it relates to mitigating urban stormwater impacts by means of implementing runoff attenuation controls.

### 2.2 Impacts of post-development stormwater runoff

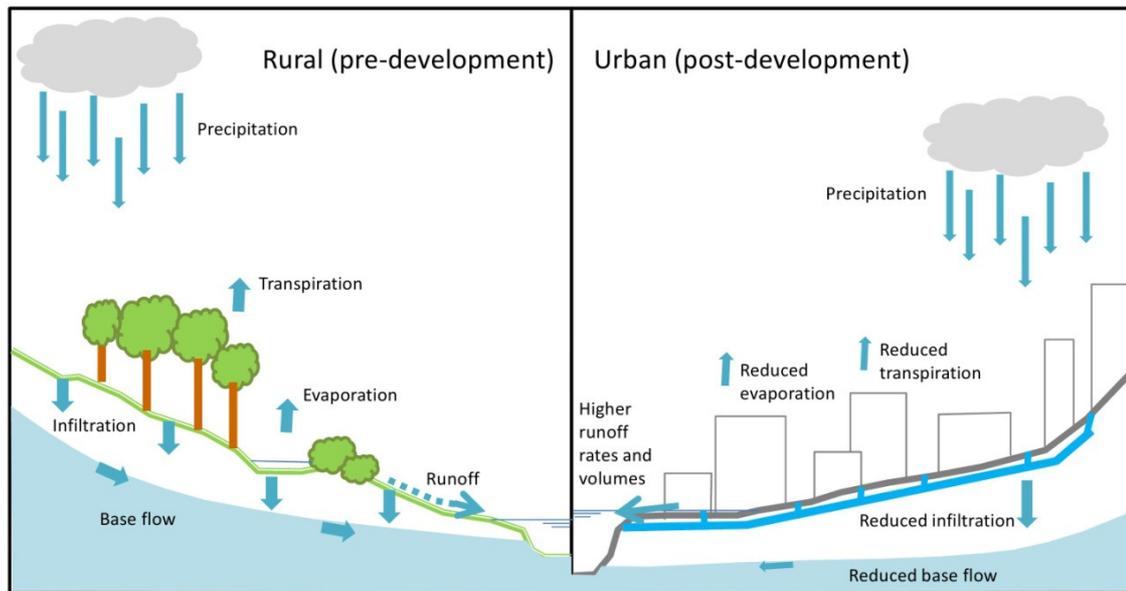
Development is a process of growth and change with the objective of improvement (CSIR, 2000). The construction of buildings, roads and other infrastructure creates the platform for economic growth in an urban centre. However, urban development can have a significant effect on the natural hydrological dynamics of the catchment in which it takes place, causing a number of unwanted consequences (Konrad, 2003; Wang and Jin, 2001).

The impact of urban development on runoff can be considered from both a social and an environmental perspective, both of which have associated economic impacts. From a social perspective, urban runoff often results in higher flood risk which endangers human life and property, while from an environmental point of view, the natural condition of watercourses, in terms of physical, chemical and biological characteristics, is often significantly altered. Some of these impacts are discussed in more detail in the following sections.

#### 2.2.1 Changes to stream flow

Under natural conditions (pre-development) rainfall would collect in vegetation, in the soil and in natural depressions resulting in a large portion of rainfall not reaching defined watercourses as overland runoff. In contrast, urban development typically reduces vegetation coverage and replaces it with larger areas of impervious surfaces which not only reduce stormwater infiltration into the soil, but also tend to facilitate faster runoff responses (Konrad, 2003). In addition, instead of the slower overland flow and natural ponding which would occur in natural pre-development conditions, runoff in urban areas is typically concentrated and conveyed as quickly as possible via gutters, pipes, roads and canals in order to avoid the perceived nuisance caused by stormwater (Urbonas and Glidden, 1983).

Du et al. (2012) conducted a study on the Qinhuai River catchment in Jiangsu Province in China which correlated recorded stream flow data with the increase in impervious coverage in the catchment and found that peak runoff rates and volumes increase linearly as impervious coverage increases, with higher percentage increases for small floods and lower percentage increases for larger floods. **Figure 2.1** illustrates schematically the comparison between natural and urban catchment runoff.



**Figure 2.1: Typical characteristics of stormwater runoff in natural and urban catchments (adapted from Melbourne Water Draft Stormwater Strategy (Melbourne Water, 2012))**

Some of the resulting changes to stream flow are listed as follows (adapted from the Georgia Stormwater Management Manual (Haubner et al., 2001)):

- Increased runoff volumes – Increased land coverage of impervious and compacted surfaces can lead to a dramatic increase in the total volume of runoff from a catchment since most of the volume of rainfall which would normally get intercepted, slowed and infiltrated or evaporated, is now being conveyed directly into the stormwater system and then to downstream watercourses.
- Increased peak runoff discharges and velocities – The rapid runoff response of a developed catchment with a traditional drainage system often results in significantly higher peak flows and velocities in downstream drainage systems. Conventional drainage infrastructure has been designed to be hydraulically efficient in removing as much flow as possible as quickly as possible, however, this tends to lead to problems such as flooding, erosion and environmental degradation downstream.
- Timing – The development of a catchment, and the introduction of an artificial stormwater drainage system, typically results in dramatic changes to the flow hydrograph of the receiving watercourse in terms of higher peak flows, faster response times and shorter flow durations.

- Increased frequency of bank-full and near bank-full events – Higher peak flows and runoff volumes during lower order storm events increase the frequency of significant stream flow within receiving watercourses. This can significantly affect the stability of natural watercourse channels (discussed further in **Section 2.2.2**).
- Increased flooding – Higher peak flows and runoff volumes result in more frequent flooding in areas adjacent to watercourses.
- Lower dry weather flows (base-flow) – Development reduces the potential for rainfall to infiltrate and recharge groundwater aquifers which results in lower base-flows in watercourses during periods of dry weather.

### **2.2.2 Changes to stream geometry**

The physical shape, or morphology, of a watercourse is a product of long-term stream flow characteristics. Natural sediment transportation processes along a watercourse typically exist in a state of equilibrium until an external influence, which could be natural or induced by human activities, results in a shift in the equilibrium (Matsuda, 2004). When this occurs, watercourse morphology often undergoes changes to establish a new equilibrium.

With the increase in runoff volumes and peak flows, as discussed in **Section 2.2.1**, watercourse channels are typically subjected to significant bank erosion and down-cutting as the flow attempts to widen or deepen the channel to increase conveyance capacity (Environment Agency, 2008). As a result, valuable soil and riparian vegetation is often eroded, causing a build-up of sediment and debris in areas downstream.

### **2.2.3 Environmental impact**

Numerous studies have shown the negative impact that runoff from urban areas has on water quality in downstream aquatic systems. Increased toxins, nutrients, and other contaminants, as well as higher water temperatures and changes in stream flow rates, can impact on the health of the river system (United States Environmental Protection Agency, 1999).

In turn, the health of a river system often has direct impacts on nearby communities, as discussed in **Section 2.2.4**.

### **2.2.4 Resulting impacts on communities**

The following points summarise the impacts that urban stormwater runoff can have on communities (adapted from the Georgia Stormwater Management Manual (Haubner et al., 2001)).

- Endangerment of human life from floodwaters – Uncontrolled runoff from urban areas has the potential to cause flash flooding in developed areas downstream, ultimately causing significant risk to human life.
- Property and structural damage due to flooding – Higher runoff volumes and peak flows from urban areas means that downstream properties which were previously outside areas of high flood hazard could now be flooded more frequently and severely. In addition, more frequent flooding increases potential for erosion of watercourse channels and floodplain areas, damaging buildings, roads, bridges, and buried services, or removing valuable land used for recreational activities.
- Impairment of drinking water supplies (surface and groundwater) – Poor stormwater quality can potentially contaminate surface and groundwater potable water supplies, making these water sources unusable, or more expensive to treat.
- Loss of recreational opportunities on streams, lakes, rivers, and beaches – Uncontrolled and unmaintained drainage systems within urban areas have the potential to become unsightly health hazards, associated with bad odours, litter and inundation by alien vegetation.
- Declining property values of waterfront homes and businesses – The poor condition of urban water bodies has a direct negative influence on property owners or users' desirability of the adjacent properties.

### 2.3 Historical context to stormwater management

In response to the impacts of post-development stormwater runoff, described in **Section 2.2**, urban populations around the world have implemented a variety of stormwater management principles throughout the history of civilisation. Typical overlapping stages of stormwater management which have been observed in urban areas historically include the following (Tarlock, 2012):

- Reactive adaption approach
- Implementation of formalised systems
- Introduction of upstream attenuation measures
- Floodplain management strategies
- Integrated stormwater management

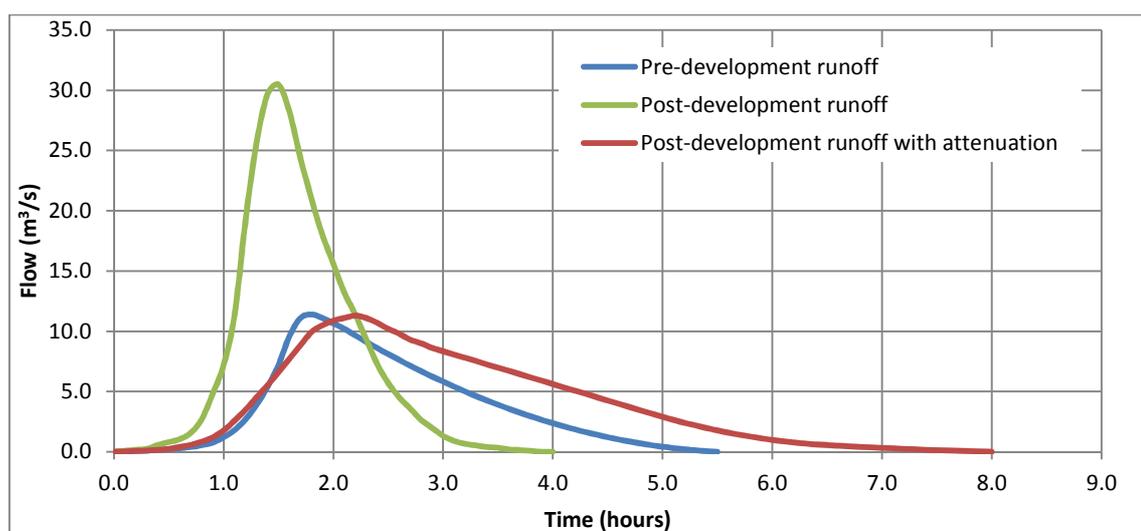
Most towns and cities around the world have a combination of formal stormwater systems, attenuation facilities and floodplain management strategies which have been in operation for many years with the primary purpose of minimising flood risk. Historically, stormwater runoff

has been regarded as an inconvenience which needed to be conveyed as quickly as possible away from urban areas via hardened ground surfaces, pipes and canals (Urbanas and Glidden, 1983). It is only really since the 1980s that stormwater management started to address the environmental impacts of urban runoff (Armitage et al., 2013b). This was initially through mitigating sources of pollution in stormwater and addressing the ecological aspects of watercourses.

More recently however, urban stormwater management has begun to encourage the preservation of natural drainage systems. This means that the changes to runoff caused by urban development in a catchment need to be mitigated with the use of a range of controls, located both regionally and near the runoff source, which aim to mimic the natural pre-development flow characteristics of the catchment (Monk and Chalmers, 2006). This approach, although relatively simple in concept, has proven extremely difficult to achieve in practice.

## 2.4 Addressing runoff impacts through attenuation

Runoff attenuation is one means of addressing runoff impacts and is now regarded as standard practice in most urban stormwater networks around the world (Fennessey et al., 2001). Attenuation ponds or wetlands temporarily store runoff during a storm and release flow downstream at a reduced rate. The pond outlet is ideally designed to release flow at a peak rate which does not exceed that which would have occurred prior to urban development in the catchment. Notably, attenuation is a means to control runoff flow rates and does not reduce the total volume of runoff for a storm event. **Figure 2.2** shows typical runoff hydrographs for a catchment prior to development, post-development, as well as post-development with the implementation of runoff attenuation.



**Figure 2.2: Typical hydrographs for a catchment pre-development, post-development, and post-development with attenuation**

From **Figure 2.2** it is seen that the post-development runoff hydrograph reaches a peak which is significantly higher than both the pre-development runoff hydrograph and the attenuated post-development runoff hydrograph. In addition, the post-development runoff hydrograph achieves its peak more quickly (i.e. the hydraulic lag time is reduced), and the total runoff volume is accounted for in a quicker time. By attenuating the post-development runoff so that the peak is reduced to the same level of the pre-development hydrograph, one can see that the falling limb of the hydrograph extends for a longer time due to the greater runoff volume passing through the system.

Stormwater attenuation facilities are normally implemented in the form of a constructed and landscaped pond consisting of an inlet with some means of energy dissipation, and an outlet structure which controls the rate of flow being discharged downstream. The pond can consist of a temporary ponding area which is only inundated during storms, or alternatively ponds can have a permanent pool with additional storage volume above the normal water level. Dry ponds, often referred to as detention ponds, typically serve the single function of flood control. Wet ponds, often referred to as retention ponds, have additionally been shown to assist in improving stormwater quality as they are more able to capture the contaminants carried by runoff from smaller storm events (Woods-Ballard et al., 2007). Typical detention and retention ponds are shown in **Figures 2.3** and **2.4** below:



**Figure 2.3: Typical detention (dry) pond in the Brackenfell area, Cape Town**

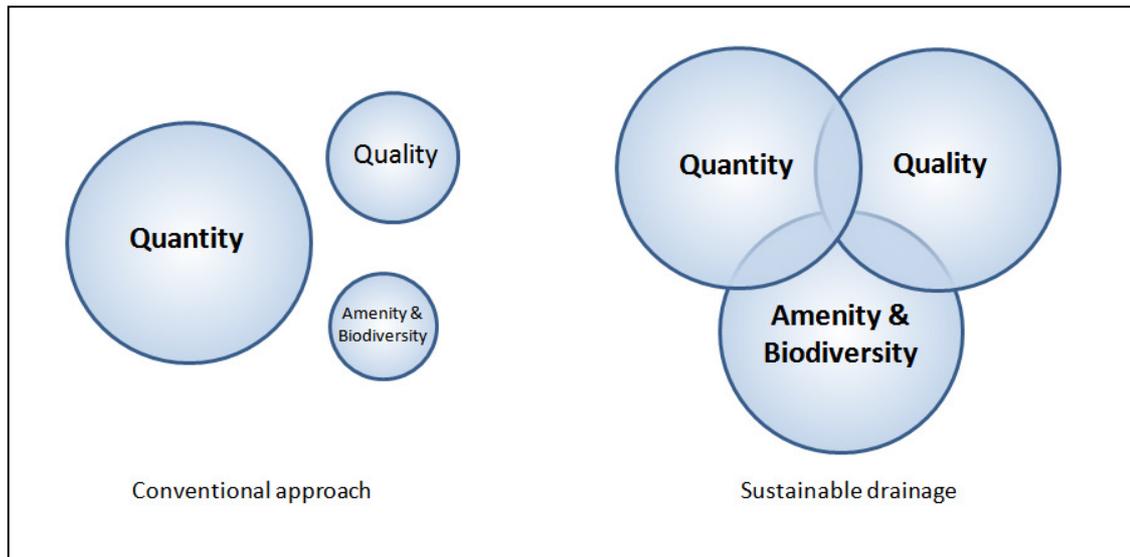


**Figure 2.4: Typical retention (wet) pond in the Durbanville area, Cape Town**

## 2.5 Sustainable drainage systems

Various forms of attenuation constitute a key component of what is often referred to as “Sustainable Drainage Systems” (SuDS). The term SuDS was originally used in the United Kingdom (UK) to describe constructed stormwater systems which attempt to mimic natural conditions as closely as possible as an alternative to conventional stormwater pipes and channels (DEFRA, 2010). Other countries use different terminology for urban design practices which include a similar approach to stormwater management. The terms “best management practices” (BMPs) and “low-impact development” (LID) are used in the United States of America (USA) to broadly describe a more environmentally sensitive design and development approach. In Australia, the term “water sensitive urban design” (WSUD) describes an integrated approach to managing all aspects of the water cycle in an urban setting with minimal impact to the environment.

SuDS aims to address quantity and quality of runoff, as well as the amenity and biodiversity value of surface water in the urban environment (Woods-Ballard et al., 2007). **Figure 2.5** illustrates the difference between the conventional approach to stormwater management and a more sustainable integrated design.



**Figure 2.5: Sustainable drainage systems (adapted from Armitage et al., 2013b)**

The impervious surfaces which traditionally make up most urban areas result in very unnatural surface runoff as discussed in **Section 2.2**. The water quality is often poor and the hydrological lag time is severely shortened causing an increased risk of flooding. SuDS techniques look to use natural processes of attenuation, sedimentation, filtration, absorption and biological degradation to reduce runoff rates and volumes, and treat the water, reverting surface runoff to a state which is as natural as possible. This is done by minimising runoff from hard surfaces, keeping roads clean, and employing innovative design techniques such as the use of filter strips, swales, permeable surfaces, and constructed wetlands and ponds. SuDS do not need to be used in isolation but should be designed to operate holistically at a catchment scale and in conjunction with existing conventional drainage systems (Ellis et al., 2002).

With the increasing implementation of a wide range of stormwater controls with a focus on treatment and infiltration one must not lose sight of the fact that the stormwater system is also required to operate safely during flood events (Faulkner, 1999). Through effective control of runoff from its point of origin using SuDS techniques, the need for large flood control structures such as attenuation ponds is reduced, but will not be completely eliminated.

The *SuDS Manual* (Woods-Ballard et al., 2007), most widely used in the UK, provides a comprehensive guideline to the philosophy of SuDS, the holistic design procedure which addresses all aspects of stormwater management, as well as recommended designs for a wide variety of components which make up SuDS, from small on-site measures to much larger regional ones.

In South Africa, however, SuDS is not a technique that has been used to any great extent and it has only recently been integrated into the stormwater guidelines and policies of some of South Africa's municipalities (Armitage et al., 2013b). Going forward, however, it is anticipated that the benefits of implementing SuDS will be realised to a greater extent by both stormwater management authorities and private property developers. The recently published document entitled *The South African Guidelines for Sustainable Drainage Systems* (Armitage et al., 2013a) provides a broad overview of many SuDS options and associated conceptual design criteria applicable in South Africa.

The SuDS approach presented in both the *SuDS Manual* and *The South African Guidelines for Sustainable Drainage Systems* recommends the use of attenuation facilities as part of a sequence of controls which address quantity, quality, amenity and biodiversity within a system.

## **2.6 Stormwater regulatory requirements in South Africa**

Generally speaking, South African legislation grants the relevant governing body the right to change the natural environment in a manner deemed to be in the interest of the public as a whole (CSIR, 2000). This includes the implementation of transportation, water, sanitation, electrical and drainage infrastructure, as well as building and landscaping developments in accordance with land use planning in the area. However, with the governing bodies' statutory powers comes the responsibility of due diligence when it comes to the implementation and maintenance of infrastructure in order to avoid third party damage wherever possible. It is within this context that national and local regulatory documentation is discussed in **Sections 2.6.1 to 2.6.5.**

### **2.6.1 National legislation**

South African national legislation broadly governing water related matters can be found in the National Water Act (NWA) (Republic of South Africa, 1998a), while environmental matters are governed by the National Environmental Management Act (NEMA) (Republic of South Africa, 1998b). These acts provide the basis for the more specific regulations instituted by local authorities.

The NWA recognises the need for regional or catchment level management of water resources so that all points within the water cycle can be managed in an integrated manner. In terms of flood management in particular, the NWA instructs local authorities to restrict new development within floodplains and to ensure that those people who may be affected by flood hazard are made aware of the risks. This would in most cases require the determination of the estimated maximum flood level which would be reached during a flood event that occurs on average once every 100 years.

Another major focus of the NWA is the protection of aquatic and associated ecosystems, and the prevention of pollution and degradation of water resources. The NWA legislates that *“the person who owns, controls, occupies or uses the land in question is responsible for taking measures to prevent pollution of water resources. If these measures are not taken, the catchment management agency concerned may itself do whatever is necessary to prevent the pollution or to remedy its effects, and to recover all reasonable costs from the persons responsible for the pollution.”*

By providing stormwater controls in urban areas, such as attenuation facilities which address flooding and water quality concerns, both public and private entities can assist in fulfilling South Africa’s national legislative requirements.

### **2.6.2 Guidelines for Human Settlement Planning and Design (The Red Book)**

The Council for Scientific and Industrial Research (CSIR) was responsible for the compilation of the *Guidelines for Human Settlement Planning and Design* (CSIR, 2000), also referred to as *The Red Book*, under the patronage of the then South African National Department of Housing. Since its first publication in 2000, this document has provided the most comprehensive and relevant source of information regarding public stormwater infrastructure planning and design in the South African context. As such, many South African local authorities rely solely on this document as a regulatory basis for the implementation of stormwater infrastructure.

The chapter on stormwater in *The Red Book* begins by providing the basic objectives of stormwater management in South Africa from a technical and legislative background, and builds an argument for integrated planning and sustainable development. It is acknowledged that, historically, stormwater has been regarded as a nuisance which must be disposed of as quickly and efficiently as possible. Typically, this was done by means of the so-called dual drainage system, consisting of an underground pipe network to convey the more frequent lower-order storm runoff, and a series of roads and open spaces to safely convey runoff during the more significant flood events. *The Red Book*, however, goes on to describe a more holistic approach to stormwater management, based on protection of health, improvement of quality of life, water conservation, environmental preservation, economic development, and sustainable financial contributions by the relevant beneficiaries. It also describes a number of BMPs for controlling stormwater and limiting environmental degradation. These include various methods of retarding or infiltrating runoff, such as attenuation, promoting overland flow, reducing impervious areas, and maintaining vegetation cover.

*The Red Book* recommends that the regional or local authority should take responsibility for catchment-wide master planning studies which take into account the above-mentioned considerations. In addition, the holistic approach should result in land use planning realistically corresponding with the stormwater requirements laid out in the master planning. The need for detailed catchment information, consisting of topography, geology, hydrology, and fauna and flora, is highlighted as this information is required for the optimisation of land use and drainage planning.

A more detailed discussion of detention and retention facilities notes that many municipalities are introducing certain requirements with regards to stormwater attenuation in urban areas. In this regard, *The Red Book* reiterates the need for catchment-wide planning and the need to assess the combined interaction of runoff from various sub-catchments in order to avoid unexpected changes in runoff patterns downstream.

*The Red Book* also provides some useful design considerations with regards to stormwater attenuation facilities, including brief descriptions of a number of outlet types such as culverts, and various weir configurations. It is acknowledged that space limitations along drainage routes often seem to limit the option for attenuation measures, however, innovative land use planning and design can often reduce land requirements while still providing a sustainable stormwater control solution.

### **2.6.3 City of Cape Town**

The Cape Peninsula's natural drainage system consists of fast-flowing mountain streams around the Table Mountain range, some wider channels in the hills to the north, and wide slow-flowing rivers and wetlands on the Cape Flats. Historically, many more interconnected wetlands would have dominated the Cape Flats, some of which may have continuously shifted with the wind-blown sand dunes (Brown and Magoba, 2009). These natural drainage characteristics have inevitably led to many flooding and environmental challenges as Cape Town has developed over the years.

The first Dutch settlers in Cape Town canalised some of the smaller streams for drainage and water supply. As the settlement grew, alternative means of water supply were developed, but the canals (or *grachts* as they were called in Dutch) continued fulfilling a drainage purpose for some years. Through most of the Dutch and British occupation in the 18<sup>th</sup> and 19<sup>th</sup> centuries, little attention was given to maintaining the drainage system until eventually a more formal underground drainage system was designed and built at the end of the 19<sup>th</sup> century. The stormwater system in the older areas of central Cape Town still consists almost entirely of an underground conveyance network.

During the 20<sup>th</sup> century, as Cape Town expanded rapidly, long portions of many of the major rivers in Cape Town, including the Liesbeeck, Black, and Elsieskraal Rivers, were canalised so that development could take place in the historical floodplains.

Today, the City of Cape Town (CCT) holistically manages both the artificial built stormwater infrastructure as well as the natural drainage system within their area of jurisdiction, including watercourses and their floodplains, wetlands and estuaries (Haskins, 2012). In terms of flood risk, any changes to the existing stormwater system or land use within the catchments, which may affect flood levels, need to be approved by the CCT and may require professional studies to be undertaken in accordance with the CCT's Stormwater By-law (City of Cape Town, 2005).

For a number of years, the CCT has been proactive, from an institutional and management point of view, in trying to address urban drainage related issues. The final draft of the CCT's *Catchment, Stormwater and River Management Strategy 2002 to 2007* (City of Cape Town, 2002) recognised the need for a paradigm shift in the way the CCT initiated drainage services around Cape Town. The document notes that, traditionally, Cape Town's urban rivers have been used as a convenient drain to convey waste and stormwater runoff. Most of the urban rivers and wetlands were modified to allow for quicker, more hydraulically efficient draining, while the catchment areas were developed to accommodate a rapidly growing population (Haskins, 2012). Consequently, Cape Town's urban rivers typically have dramatically altered runoff response hydrographs and are characterised by very poor water quality. The poor water quality has in turn created adverse health hazards and has significantly reduced the amenity value of Cape Town's watercourses.

The new strategy for how the CCT should initiate drainage services focused on an integrated stormwater management approach, incorporating water quantity and quality objectives, as well as local socio-economic considerations in support of broader city objectives. It was no longer acceptable to merely address hydraulic considerations, such as flood risk. The CCT recognised the need for ecologically healthy watercourses which are safe from a human health perspective, and attractive to the local community from a recreational or general amenity perspective. These objectives would need to be addressed by means of structural measures such as SuDS and WSUD, as well as non-structural measures such as better land use planning, community educational programmes and capacity building initiatives.

During the period since 2002, the CCT began investigating ways in which the new strategy's objectives could be effectively implemented. The latest documentation, which provides a practical basis for the implementation of these objectives, consists of two comprehensive policy documents, namely, the *Floodplain and River Corridor Management Policy* (City of Cape Town, 2009a) and the *Management of Urban Stormwater Impacts Policy* (City of Cape Town, 2009b).

The *Floodplain and River Corridor Management Policy* “outlines the procedure for managing development adjacent to watercourses and wetlands taking cognisance of the flood regime, aquatic and riparian ecology as well as socio-economic factors”. It provides requirements as to where certain new developments are permitted in relation to ecological buffers and floodplains, while also recognising the need for integration of watercourses into the existing urban landscape by improving their aesthetic appeal.

In order to support this policy, the CCT has, over the past decade, commissioned a number of floodline and flood hazard mapping studies which cover a significant portion of Cape Town’s urban rivers and allow for better decision making when it comes to the approval of new developments.

The *Management of Urban Stormwater Impacts Policy* outlines the practical measures which must be undertaken in order to mitigate the impacts typically resulting from uncontrolled stormwater runoff. The document notes that “*In order to reduce impacts of urban stormwater systems on receiving waters, all stormwater management systems shall be planned and designed in accordance with best practice criteria and guidelines laid down by Council, to support Water Sensitive Urban Design principles and the following specific sustainable urban drainage system objectives:*

- *Improve quality of stormwater runoff;*
- *Control quantity and rate of stormwater runoff;*
- *Encourage natural groundwater recharge.”*

Besides non-structural controls, such as stormwater focused educational initiatives and land use planning, the policy focuses on the requirements which need to be considered by developers in the design of on-site structural stormwater infrastructure. These requirements include both quality and quantity aspects, and specify separate requirements for new urban development (greenfield) and development within existing urban areas (brownfield).

In terms of quality aspects, the policy generally requires that suspended solids and total phosphorus in the post-development stormwater runoff is reduced by 80% and 45%, respectively, through a combination of BMPs which infiltrate or capture and treat the runoff.

There are three requirements specified in order to achieve the objectives for the control of runoff rate and quantity, typically through the use of attenuation controls. Firstly, it is proposed that downstream channel stability is protected by means of 24 hours of extended detention of the 24-hour storm event which occurs on average once a year. “Nuisance” flooding is controlled by reducing the post-development peak flows of the floods recurring up to once every 10 years to pre-development levels. Major floods, up to peak flows occurring

once every 50 years, should also be reduced to pre-development levels, while the 100-year recurrence interval flood must be evaluated and managed without significant damage to urban properties.

In order to achieve the above-mentioned policy requirements in terms of both quality and quantity, the CCT has initiated a number of catchment-wide stormwater master planning studies which assess existing and future areas of the city, in most cases at a high level looking at regional requirements.

#### **2.6.4 City of Johannesburg**

Johannesburg began as a small mining settlement in 1886, following the discovery of gold on the Witwatersrand. By 1905, the settlement was expanding rapidly and flooding and general road drainage were already becoming a problem (Whitlow and Brooker, 1995). From these early years, piped stormwater networks were installed and rivers were canalized to deal with erosion problems resulting from greater and more frequent flooding caused by catchment “hardening”. This became a trend as urbanisation continued throughout the 20th century. Today, much of the stormwater infrastructure of Johannesburg consists predominantly of pipe systems and modified watercourses.

A study of the hydrological history of Johannesburg shows that flooding and river channel erosion was generally dealt with in a reactive, rather than a proactive manner, and without any consideration for the natural environment. In this regard, Whitlow and Brooker (1995) noted that *“there is clearly a need for analysis of complete catchments or river systems within metropolitan Johannesburg rather than ad-hoc studies of selected problem reaches”*.

From the late 1990s, with the ushering in of the new South African Constitution (Republic of South Africa, 1996) and the subsequent NWA and NEMA, the City of Johannesburg (CJ) began initiating gradual changes in the way the management of rivers and stormwater systems was undertaken (Botha, 2005). However, it was not until the issuing of the *Draft Stormwater By-Laws* in 2009 (City of Johannesburg, 2009) that the CJ could enforce certain stormwater related policies on developers. The policies presented in the By-Laws document are intended to support the broad requirements of the national legislation by providing specific requirements, especially in relation to new development.

According to the By-Laws, *“the purpose of these By-laws is to manage, control and regulate the quantity, quality, flow and velocity of stormwater runoff from any property which it is proposed to develop or is in the process of being developed or is fully developed, in order to prevent or mitigate:*

- *erosion and degradation of watercourses;*
- *sedimentation in ponds and watercourses;*
- *degradation of water quality and fish habitat; and*
- *excess stormwater runoff onto a public road which may pose a danger to life or property or both."*

In order to achieve these objectives, the By-Laws prescribe the use of BMPs and attenuation facilities. In terms of controlling post-development runoff, *"the post-development peak stormwater discharge rate from a development site for a 5- to 25-year recurrence interval design storm event of any duration from 0.25 to 24 hours, may not at any time exceed the pre-development peak stormwater runoff rate from that site for the same design storm event"*. The CJ also has the legal right to increase the specified recurrence interval to 50 years if deemed necessary.

Additionally, the By-Laws specify the requirement for a stormwater drainage plan to be submitted by developers to the CJ for approval, prior to the implementation of the development. The stormwater drainage plan should evaluate the potential impacts resulting from increased runoff and degradation of stormwater quality, and propose measures to mitigate the anticipated impacts.

### **2.6.5 eThekweni Municipality**

The city of Durban, which is governed by the eThekweni Municipality, has a humid subtropical climate with a high summer rainfall. The city is characterised by hilly terrain, and the primary stormwater conveyance system consists of a relatively high density of natural watercourses, including some large rivers such as the Umgeni and Mlazi Rivers. As the city of Durban grew, many areas that were once wetlands or floodplains were filled in and rivers were canalised and sometimes diverted to suit development.

Today the eThekweni Municipality holds full responsibility for the management of stormwater systems within their municipal limits and any new development within the municipal area is subject to approval by the Municipality. In this respect, the Engineering Unit of the eThekweni Municipality's Coastal Stormwater and Catchment Management Department has compiled a design manual entitled *Guidelines and Policy for the Design of Stormwater Drainage and Stormwater Management Systems* (eThekweni Municipality, 2008). This guideline focuses on control of lower order storm runoff (i.e. the 3-year recurrence interval storm) at a site level by means of soakpits and other attenuation measures, however, it also briefly outlines the general requirements for the control of more significant storm events.

The policy generally assumes that the existing stormwater system in residential areas is designed to accommodate lower order storm runoff from catchment areas consisting of up to 40% impervious surfaces. Therefore, development within existing urban areas which has direct access to the municipal stormwater network is generally permitted, without any additional controls, as long as the impervious area does not increase beyond 40% of the total site area. Where development exceeds this limit, the developer is required to provide measures to reduce runoff to acceptable levels. The general rule of thumb, suggested by the *Guidelines*, is that soakpits be constructed, and sized on the basis of 1 m<sup>3</sup> of clear volume for every 40 m<sup>2</sup> of hardened catchment area. Alternative means of attenuation recommended include rainwater tanks and ponds.

Development of greenfield sites without access to existing stormwater infrastructure requires the submission of a stormwater management plan showing the controls and mitigation measures proposed in order to prevent increased flooding in adjacent or downstream areas.

A stormwater management plan, produced by a professional engineer, is also required for more extensive developments. Whereas individual units cannot be expected to provide significant onsite control of major floods (above 10-year recurrence interval), the *Guidelines* require that larger developments, consisting of multiple residential units, provide infiltration and/or attenuation measures for the control of flood runoff up to the 50-year recurrence interval event. In addition, it explicitly states that the rate of outflow from these controls, for any recurrence interval storm event, must be restricted to the equivalent pre-development rate.

## **2.7 International stormwater attenuation regulations and guidelines**

The impacts of stormwater runoff affect all cities around the world and there are many ways of managing these impacts. This section summarises some of the more comprehensive and relevant international policies and guidelines, specifically focussing on stormwater attenuation practices.

### **2.7.1 US guidelines and legislation**

The available national legislation relating to stormwater and flood related practices in the USA is generally covered by the Clean Water Act, the Flood Control Acts, and the National Flood Insurance Act.

The Flood Control Acts, promulgated in approximately 20 acts between 1917 and 1970, are administered at federal level by the United States Army Corps of Engineers (USACE), although, flood control measures are also conducted by state and local agencies. The Acts typically deal with the funding and implementation of measures to reduce damage and loss of life during major flood events. In addition to these acts, the US government established the National

Flood Insurance Program (NFIP) in 1968 which is administered by the Federal Emergency Management Agency (FEMA). The NFIP initiated the mapping of the 1 in 100 year floodplains and high hazard areas and enforced the introduction of local flood management programmes which aim to reduce flood losses by limiting development within floodplains. In addition, the NFIP provides compulsory, subsidised flood insurance for the property owners within the 1 in 100 year floodplain.

It is argued that the USA's flood control policies have serious shortcomings (Haubner et al., 2001; Tarlock, 2012). Most significantly, the NFIP tends to promote development within the floodplain areas due to a false sense of safety. The floodplain areas of smaller watercourses which are not included in the flood mapping programme are also susceptible to development which is exposed to unexpected flood risks. Furthermore, the loss of natural attenuation on river floodplains can also have an impact on developments downstream.

The majority of urban stormwater related regulations governing the USA at federal level are enforced through the Clean Water Act, administered by the Environmental Protection Agency (EPA) in partnership with state environmental agencies. This act, together with a host of related EPA documentation, provides a wide range of information regarding stormwater management, particularly with respect to the reduction of contaminants in stormwater runoff through the implementation of various BMPs, including attenuation of runoff.

In 2007, US Congress recognised the impact of stormwater in the Energy Independence and Security Act of 2007 (EISA) which stipulates that the "sponsor of any development or redevelopment project involving a Federal facility with a footprint that exceeds 5,000 square feet shall use site planning, design, construction, and maintenance strategies for the property to maintain or restore, to the maximum extent technically feasible, the predevelopment hydrology of the property with regard to the temperature, rate, volume, and duration of flow".

Following the promulgation of the EISA, in December 2009 the US EPA published the document *Technical Guidance on Implementing the Stormwater Runoff Requirements for Federal Projects under Section 438 of the Energy Independence and Security Act* (United States Environmental Protection Agency, 2009a) which is a very useful guideline for designers attempting to eliminate all stormwater impacts of a development.

The above-mentioned guideline notes that the implementation of Section 438 of the EISA will address the inadequacies of the traditional practices of attenuation which typically only control the larger storm events. The focus of the document is therefore on the retention and control of frequent rainfall events and the elimination of contaminants, with only a brief mention of the control of peak flows during larger flood events. This perhaps illustrates the short-coming of much of the EPA's documentation regarding stormwater: it neglects to

integrate stormwater quality requirements with the equally important flood risk aspect of stormwater associated with larger storm events.

Ultimately it is the responsibility of local authorities to ensure that the runoff problems occurring as a result of development within their jurisdiction are managed in a way that prevents flooding and mitigates environmental impact (Haubner et al., 2001). This is undertaken through the promulgation of various ordinances and permit requirements usually backed up by an adopted stormwater management manual which would contain all applicable policies and design standards to be enforced by the authorities.

There are a number of publically available stormwater management manuals used around the USA, many of which detail similar design concepts. The primary objective of most of the regulations and design guidelines contained within these manuals is to maintain pre-development conditions, in terms of stormwater quality, as well as flow rates, volumes and velocities, in natural drainage systems once the catchment has been developed. One of the most comprehensive and widely used of these manuals is the *Georgia Stormwater Management Manual* (GSMM) (Haubner et al., 2001). The GSMM provides an integrated approach for addressing both water quality and quantity impacts of post-development runoff by addressing the following key goals:

- Remove stormwater runoff pollutants and improve water quality;
- Prevent downstream bank and channel erosion;
- Reduce downstream overbank flooding; and
- Safely pass or reduce the runoff from extreme storm events.

The above goals are achieved by means of an integrated set of criteria, referred to as the *Unified Stormwater Sizing Criteria*, summarised in **Table 2.1**:

**Table 2.1: Summary of the Unified Stormwater Sizing Criteria (adapted from the GSMM)**

Sizing Criteria	Description
Water Quality	Treat the runoff from 85% of the storms that occur in an average year. Reduce average annual post-development total suspended solids loadings by 80%.
Channel Protection	Provide extended detention of the 1-year recurrence interval storm event released over a period of 24 hours to reduce bank-full flows and protect downstream channels from erosive velocities and unstable conditions.
Overbank Flood Protection	Provide peak discharge control of the 25-year storm event such that the post-development peak rate does not exceed the pre-development rate to reduce overbank flooding.
Extreme Flood Protection	Evaluate the effects of the 100-year storm on the stormwater management system, adjacent property, and downstream facilities and property. Manage the impacts of the extreme storm event through detention controls and/or floodplain management.

The four criteria listed in **Table 2.1** are intended to be implemented in conjunction with each other in order to holistically address many of the potential stormwater impacts associated with a variety of storm events.

### 2.7.2 General floods and stormwater management in Europe

The management of runoff across much of mainland Europe is broadly overseen by the European Commission (EC) and legislated to some extent by the Water Framework Directive and the Floods Directive (European Commission, 2011). Member states of the European Union are required to collaborate across borders to address catchment-wide management objectives from a flood risk and water resource perspective. There are some large river systems which affect multiple communities across a number of countries. A cooperative approach to stormwater management is therefore of utmost importance, since poor management has the potential to have widespread consequences downstream. The EC therefore encourages all member states to assess and mitigate all flood risks and other stormwater impacts with a high level of environmental protection in accordance with the principle of sustainable development as laid down in *Article 37 of the Charter of Fundamental Rights of the European Union* (European Parliament, 2007).

The note by the Directorate-General Environment, *Towards Better Environmental Options for Flood Risk Management*, proposes that flood risk management make use of natural processes through the use of green infrastructure which will slow, store and attenuate flow during flood events (European Commission, 2011). It argues further that this approach not only reduces the risk of flooding when designed appropriately, but it also helps to conserve river and wetland ecosystems, thereby increasing carbon dioxide intake and reducing the impact of climate change.

### 2.7.3 UK guidelines and policies

Policy and funding of stormwater and flood risk related matters in the United Kingdom (UK) are governed at a national level by the Department for Environment, Food and Rural Affairs (DEFRA). DEFRA is supported by the Environment Agency, an executive non-departmental public body, which has compiled a strategic overview in terms of flood risk management for significant water bodies across the UK, as well as providing a framework which informs and supports the implementation of local stormwater risk management strategies. Various drainage authorities are then responsible for ensuring that the impacts of stormwater are managed on the ground through enforced regulations and the implementation, operation and maintenance of various controls.

Current UK legislation places a significant emphasis on sustainable development and the use of SuDS for the management of stormwater runoff and the mitigation of flood risk. The *Flood and Water Management Act 2010* (United Kingdom, 2010) states that flood management “*must aim to make a contribution towards the achievement of sustainable development*” and that it is the responsibility of the relevant cabinet minister to provide guidance as to how this should practically be accomplished by the authorities responsible for the control of urban development as it relates to stormwater management. This would include the publication of national standards for sustainable drainage systems (first published in December 2011 (DEFRA, 2011a)), and the implementation of an approval process which would ensure that developments comply with these standards.

The above-mentioned *National Standards for Sustainable Drainage* stipulates the following objectives in terms of the control of peak runoff rates and volumes:

- There must be no discharge to a surface water body or sewer that results from the first 5mm of any rainfall event.
- The peak flow rates for the:
  - a) 1 in 1 year rainfall event; and
  - b) 1 in 100 year rainfall event;

must not be greater than the equivalent greenfield runoff rates for these events. The critical duration rainfall event must be used to calculate the required storage volume for the 1 in 100 year rainfall event.

- The volume of runoff must not be greater than the greenfield runoff volume from the site for the 1 in 100 year, 6 hour rainfall event.
- The volume of runoff may only exceed that prior to the proposed development where the peak flow rate is restricted to 2 l/s/ha.

The design and implementation of these objectives can be achieved with a number of controls, details of which are in the SuDS Manual (Woods-Ballard et al., 2007) (discussed in **Section 2.5**). Each development requiring approval from the local authority will, from April 2014, require their stormwater management plan to be reviewed and approved by a SuDS Approving Body.

In addition, the DEFRA has initiated a number of other useful guidelines such as *Surface Water Management Plan Technical Guidance* (DEFRA, 2010), and *Guidance for risk management authorities on sustainable development in relation to their flood and coastal erosion risk management functions* (DEFRA, 2011b). In addition, the *National Flood and Coastal Erosion Risk Management Strategy for England* (DEFRA; Environment Agency, 2011) provides the overarching framework for addressing flood risk in England, building on existing approaches, as well as looking at potential future impacts, such as climate change. One of its main objectives is to outline the roles and responsibilities of various stakeholders so that they are empowered to work more effectively together to achieve the goals of sustainable development as it relates to flood and coastal erosion risk management.

A further part of this strategy is to implement surface water management plans (SWMPs) for key catchments in order to reduce flood risk through coordinated action plans and informed capital investment planning. This is typically undertaken through flood risk assessments at an appropriate level of detail for the given area, and often will include modelling of the catchment's surface and sub-surface drainage systems (DEFRA, 2010).

#### **2.7.4 Australian guidelines and policies**

There is involvement from all spheres of government in the Commonwealth of Australia when it comes to the management of floods and stormwater. In terms of overarching environmental aspects of stormwater and river management, the Environmental Protection Authority is responsible for enforcing Commonwealth environmental legislation (Government of Western Australia, 2004). Many of the states and territories have planning structures in place which prescribe procedures and standards to be followed by local government. In turn, local government, or the statutory local water management authority, often establish strategies,

policies and master plans which form the basis for development approvals in their areas of jurisdiction (ANZECC and ARMCANZ, 2000).

Australia is considered to be one of the countries at the forefront in terms of the implementation of best management practice when it comes to urban stormwater. From as early as the 1960s, Australian authorities have increasingly pursued more sustainable means of developing the built environment. Through this process they have effectively evolved from “traditional” stormwater systems which only considered public health and safety risks, to systems which also considered the environmental health and amenity of urban drainage systems (Melbourne Water, 2012). In this process, the concept of WSUD was born in the 1990s.

Initially, WSUD was only associated with stormwater management, and generally focused on water quality aspects at development site level, specifically related to frequent storm events (recurrence intervals of typically less than one year). Because of this, a large portion of the available literature on WSUD is still very narrowly focussed. However, as the concept of WSUD has developed, its definition has broadened significantly to a point where today it deals holistically with all aspects of the urban water cycle. According to Wong (2007), Australia’s *Intergovernmental Agreement on a National Water Initiative* (Council of Australian Governments, 2005) defines WSUD as “the integration of urban planning with the management, protection and conservation of the urban water cycle, that ensures that urban water management is sensitive to natural hydrological and ecological processes”.

One of the main objectives for stormwater management is to maintain pre-development flooding regimes in streams and rivers following urban development, generally through the use of attenuation and infiltration (Melbourne Water, 2012). The Stormwater Management Manual for Western Australia (Government of Western Australia, 2009) recommends that runoff from frequent rainfall events be retained or detained as close to the source as possible with the use of controls such as soakaways, pervious paving, swales and rainwater tanks. For larger flood events, the Manual recommends that runoff from developments be attenuated by means of landscaped detention or retention ponds located in road reserves or public open spaces. The system for larger floods should follow vegetated flow paths and, where practical, flow regimes should mimic pre-development conditions in terms of peak flow rates, as well as flow volumes.

The Queensland Urban Drainage Manual (2007) provides a very comprehensive section on many of the technical considerations related to the implementation of attenuation facilities in an urban catchment and also highlights some of the potential problems associated with attenuation of floods. This manual strongly advocates the use of catchment-wide stormwater

modelling for detailed catchment planning in terms of routing and attenuation of floods and thus does not specify a single solution for all scenarios.

## **2.8 Technical considerations for implementation of attenuation practices**

As shown in the local and international design guidelines and policies presented in **Sections 2.6** and **2.7**, there is no clear consensus with regards to the implementation of attenuation in the control of urban stormwater runoff. The effect that urban development has on runoff is complex and often difficult to predict. As a result, there is no simple solution when designing controls, such as attenuation facilities, which aim to mitigate negative impacts of post-development runoff and achieve the objectives of human safety and environmental preservation downstream. Furthermore, it is equally difficult to predict the effect that the implementation of multiple flood controls will have on a catchment-wide stormwater system. **Sections 2.8.1** to **2.8.6** discuss many of the technical considerations which need to be taken into account in the implementation of attenuation in an urban stormwater system.

### **2.8.1 Pond sizing parameters**

There are a number of factors which need to be taken into account in the design of urban flood attenuation ponds. Firstly, it is important to establish to what degree the attenuation pond is going to reduce post-development flows. Although some international legislation and policies specify release rates based on empirical functions, most leave it up to the designer to ensure that the post-development peak runoff rates are reduced to the peak runoff rate that would have naturally occurred prior to development (Faulkner, 1999).

Furthermore, there are varying assertions as to which design storms the pond should cater for, as it is often impractical to design for every storm event. In the past, many attenuation ponds have only been designed to reduce post-development peak runoff from one or two recurrence interval storm events (Urbonas and Glidden, 1983). The result of this is that usually the less frequent recurrence interval storms, such as the 50- or 100-year events, are effectively controlled, but the smaller, more frequent storm events are often not designed for, resulting in increased lower-order flooding and watercourse channel degradation (Fennessey et al., 2001).

Another important consideration when designing an attenuation facility is whether or not it needs to address water quality aspects as proposed by the *Georgia Stormwater Management Manual* (Haubner et al., 2001), which provides an integrated approach for addressing both water quality and quantity impacts of post-development runoff, as discussed in **Section 2.7.1.2**.

It is noted, however, that some urban stormwater systems fail despite the attempted implementation of designs which address all of the above sizing criteria. Although flood

attenuation practices generally aim to reduce runoff back to pre-development rates, the performance of the control depends largely on the design criteria used; especially the design of the outlet which determines the release rate. Booth and Jackson (1997) note that the actual performance of attenuation ponds depends firstly on how closely they are designed to mimic pre-development runoff conditions, and secondly, on how accurate pre-development conditions are determined. For example, should the designer over-predict the pre-development runoff, thereby designing the pond for a flow larger than what would naturally occur, this will potentially lead to more frequent flooding downstream, and watercourse channel degradation (Fennessey et al., 2001). This highlights the importance of choosing flood hydrology calculation or modelling methods that are applicable to the catchment being analysed and which can preferably be backed up and verified by catchment specific input data and calibration data.

The decision regarding which design storms to attenuate cannot always be made for a single pond in isolation (Queensland Government, 2007). The ultimate aim of any urban stormwater system should be to mimic pre-development flow regimes for multiple storm events, where practical (Monk and Chalmers, 2006). This may mean that upstream controls address more frequent storm events and a downstream attenuation facility only attenuates flow from larger storms.

The means of flow control is through the design of the pond outlet structure, and ultimately, the required volume of the pond is a function of this control mechanism. This is discussed in more detail in **Section 2.8.2**.

### **2.8.2 Pond outlet design**

The design of the outlet structure is an essential component in achieving the functional objectives of the attenuation facility. The outlet structure has to be designed in such a way as to take into account inflow rate into the pond, and the relationship between water head in the pond and outlet discharge.

In the past, many attenuation facilities have been designed with a single design outflow rate in mind. For example, for a given post-development inflow into the pond during a 50-year recurrence interval storm, a culvert outlet would ensure that at the peak inflow rate, the full storage volume of the pond would be utilised, and at this level, the outflow would be restricted to the 50-year pre-development rate. This type of design may ensure that the 50- and 100-year flood hazard zones do not extend into new areas, however, as discussed in **Section 2.8.1**, it does not address lower-order events which can result in more frequent “nuisance” flooding and watercourse channel degradation. A number of examples of these outlets used in the Cape Town area are shown in **Figures 2.6 to 2.8**.



**Figure 2.6: Single culvert-type outlet of a detention pond in the Blaauwberg area**



**Figure 2.7: Detention pond outlet structure in Kraaifontein area**



**Figure 2.8: Retention pond outlet in Durbanville area**

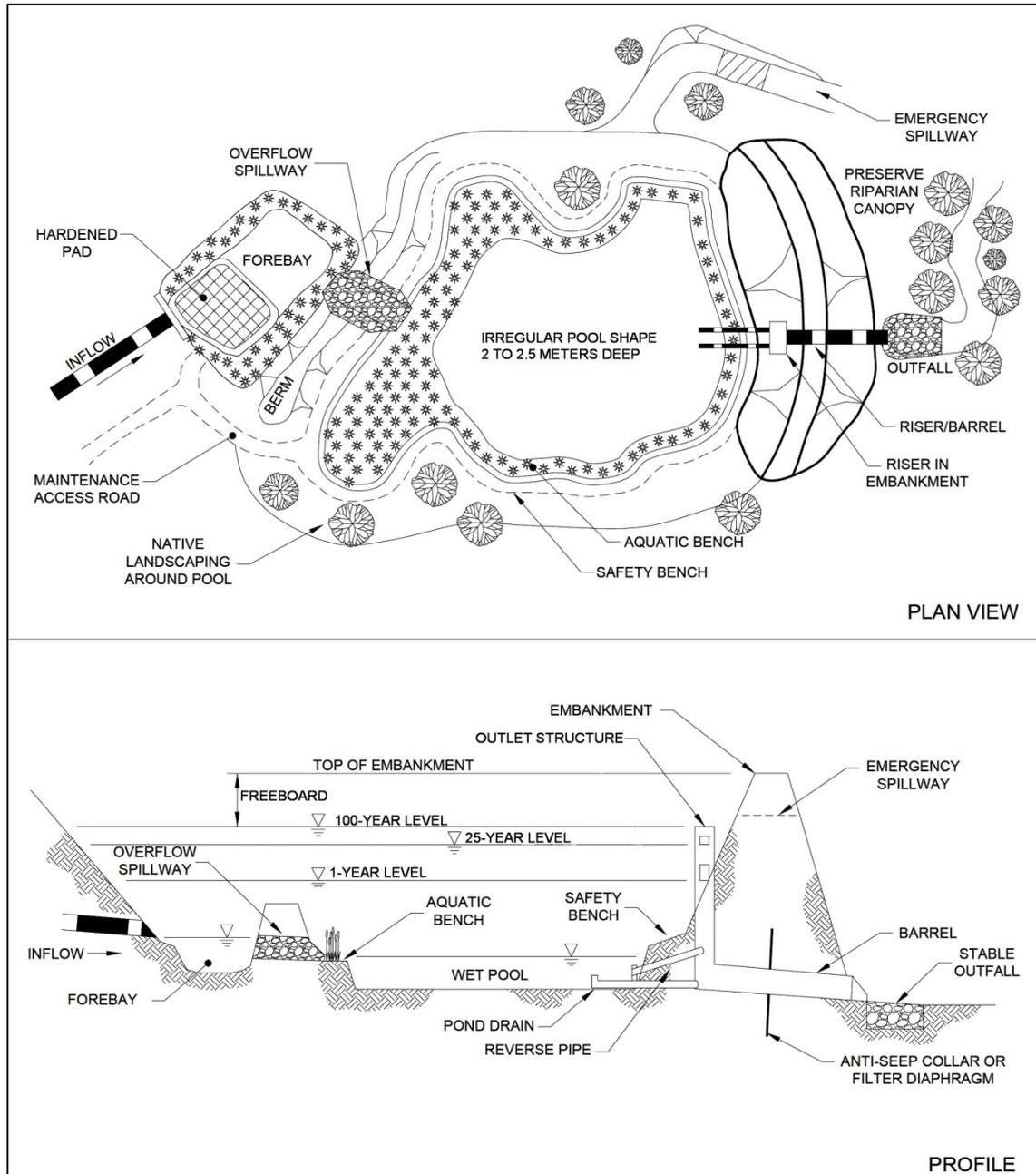
Current legislation in many of South Africa's metropolitan municipalities now requires developers to construct on-site flood control structures which restrict runoff from a number of storm recurrence interval events to pre-development rates (refer to **Section 2.6**). In order to achieve this, outlet flows from attenuation ponds need to be controlled by means of a multi-stage outlet structure, with a head-discharge curve which addresses the full range of design flows. **Figure 2.9** shows a typical layout of an attenuation pond designed for a range of recurrence interval storms (Knox County, 2008).

There are a number of outlet types, including the following:

- Orifice
- Broad-crested weir
- Perforated riser
- V-notch weir
- Pipe/culvert
- Proportional weir
- Sharp-crested weir
- Combination outlet

Each of these outlet types have different conveyance characteristics and are therefore applicable to different design requirements. The control of low flows for water quality and

channel protection are normally handled with smaller, more protected outlet types such as reverse sloped pipes, hooded orifices and V-notch weirs which are less likely to get blocked. Larger discharges during higher-order storm events (2- to 100-year recurrence intervals) are typically handled through a structure with a number of different sized openings or overflow weirs at various levels. **Figures 2.10** and **2.11** show two examples of multi-stage outlet structures currently in use in the Cape Town area.



**Figure 2.9: Typical attenuation pond (adapted from the Knox County Stormwater Management Manual)**



**Figure 2.10: Multi-level outlet structure in Sunningdale area, Blaauwberg**

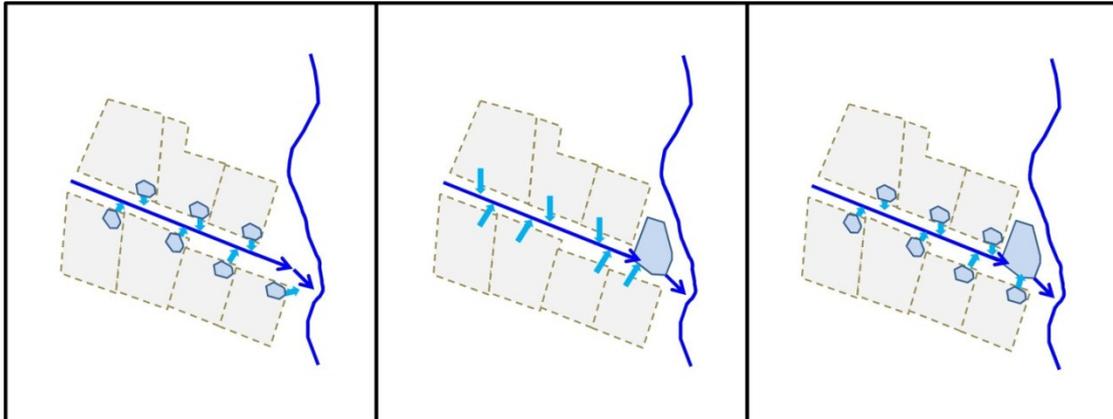


**Figure 2.11: Multi-leveled outlet structure in Atlantis**

### **2.8.3 On-site versus regional attenuation**

There is an on-going debate, both locally and internationally, over whether authorities should implement stormwater attenuation at a regional level so that a single facility serves a number of developments, or whether developers should be required to implement attenuation

facilities on-site when the development area is bigger than a certain size. Alternatively, a combination of on-site and regional ponds could be used where required. These three options are indicated schematically in **Figure 2.12**.



**Figure 2.12: On-site versus regional attenuation**

Current stormwater legislation promulgated in most of the metros in South Africa require that developers finance and implement on-site mitigation measures in terms of runoff impacts caused by the development (refer to **Section 2.6**). This practice, which is also commonly implemented by local authorities worldwide, results in much of the financial burden of stormwater management being taken off the local authority and placed on developers (Buys and Aldous, 2009). Although this benefits local authorities to some degree, there are a number of arguments against on-site attenuation, some of which are summarised as follows (Haubner et al., 2001):

- On-site attenuation results in a direct financial impact on developers who often lose developable land area to stormwater attenuation requirements.
- The design and construction of on-site attenuation facilities is often difficult for authorities to effectively regulate.
- On-site ponds are often not maintained when located within a gated development complex.
- One strategically located regional attenuation facility is arguably more cost effective than a number of on-site facilities both in terms of capital cost and maintenance costs.
- In comparison to on-site attenuation, there is often more opportunity for regional attenuation facilities to serve other community uses such as recreation and over-flow parking areas.

Should regional attenuation facilities be implemented in isolation, the following stormwater management concerns need to be considered:

- Infrastructure upstream of the regional attenuation facility must be able to cope with the unattenuated runoff from the development.
- Smaller storm events and water quality impacts may not be adequately controlled by a regional facility. A more effective way to reduce the concentration of contaminants in stormwater runoff is to implement a series of on-site, local and regional treatment facilities (Armitage et al., 2013a; United States Environmental Protection Agency, 2009a; Woods-Ballard et al., 2007).

These two concerns are the primary reasons why many stormwater management authorities have opted to enforce the implementation of on-site attenuation onto new developments bigger than a specified size. However, with this approach, the onus is on the authorities responsible for stormwater management to regulate the design, construction and maintenance of these attenuation facilities with broader catchment objectives in mind. Some of the technical considerations which must be addressed by authorities are discussed in **Sections 2.8.4 to 2.8.6**.

#### **2.8.4 Runoff timing and catchment dynamics**

Changes to the natural drainage system through the introduction of attenuation facilities can result in unforeseen secondary impacts further downstream. Therefore, the control of urban flooding cannot be effectively implemented without an understanding of the catchment dynamics. The relative timing of sub-catchment response plays a critical role in the flood magnitude experienced downstream and it is therefore important to take this into account when assessing the impact of development and flood attenuation practices (Pattison et al., 2008).

Faulkner (1999) argues that, *“flood risk from new developments is more a function of the modified timings of runoff peaks and their interactions rather than volumes of runoff”*, and goes further to say that *“there appears to be a lack of appreciation of the true consequences of widespread flow attenuation in terms of both (i) the operational performance of the storage structure itself, and (ii) whole catchment effects when considering downstream flooding resulting from the many complex interactions of urban and rural runoff”*.

Faulkner (1999) modelled a theoretical catchment scenario to illustrate how the attenuated runoff from a new development can lead to increased flood risk in areas downstream due to the timing of the watercourse flows. The example shows that although peak flows are effectively controlled at a local scale (i.e. at the new development and immediately downstream), attenuation results in higher peak flows further downstream compared to a scenario where no attenuation is implemented at the new development. This result is due to the timing of peak flows from adjacent sub-catchments. The peak outflow from the new development is not only reduced but also prolonged so that when the peak flow from the

adjacent sub-catchments reach the confluence with the flow being slowly released from the new development, the result is worse than if the flow from the new development was more rapidly released so as to pass downstream prior to the peak from the adjacent catchments. This scenario is shown graphically in **Figure 2.13**.

As mentioned in **Section 2.7.4**, the Queensland Urban Drainage Manual (2007) indicates that issues related to catchment dynamics can be avoided by undertaking detailed planning studies involving total catchment modelling which can inform decisions related to the design and placement of attenuation facilities.

### **2.8.5 Cumulative effect of catchment-wide attenuation**

It has been observed that although flood attenuation controls are able to limit the runoff rate to pre-development levels, they are unable to significantly reduce the total runoff volume to pre-development levels (Petrucci et al., 2011). Goff and Gentry (2006) evaluated the influence of various catchment characteristics on the cumulative impacts of stormwater attenuation and found that even if all the sub-catchments within a larger catchment area release urban runoff at pre-development levels, the elevated volumes result in increased flood peaks downstream and the flooding risk gets worse with a greater number of developed sub-catchments. **Figure 2.14** provides a comparison of the combined flow hydrograph from two tributaries prior to urban development and post-development with attenuation facilities implemented.

In order to compensate for the downstream cumulative effects, outflows from ponds could be reduced to less than pre-development rates. However, even if the downstream composite peak from a number of sub-catchments is adequately reduced to pre-development levels, due to the increased stored volume, peak flow conditions would persist for a longer duration, which would potentially impact the natural river stability (Hardt and Burges, 1976).

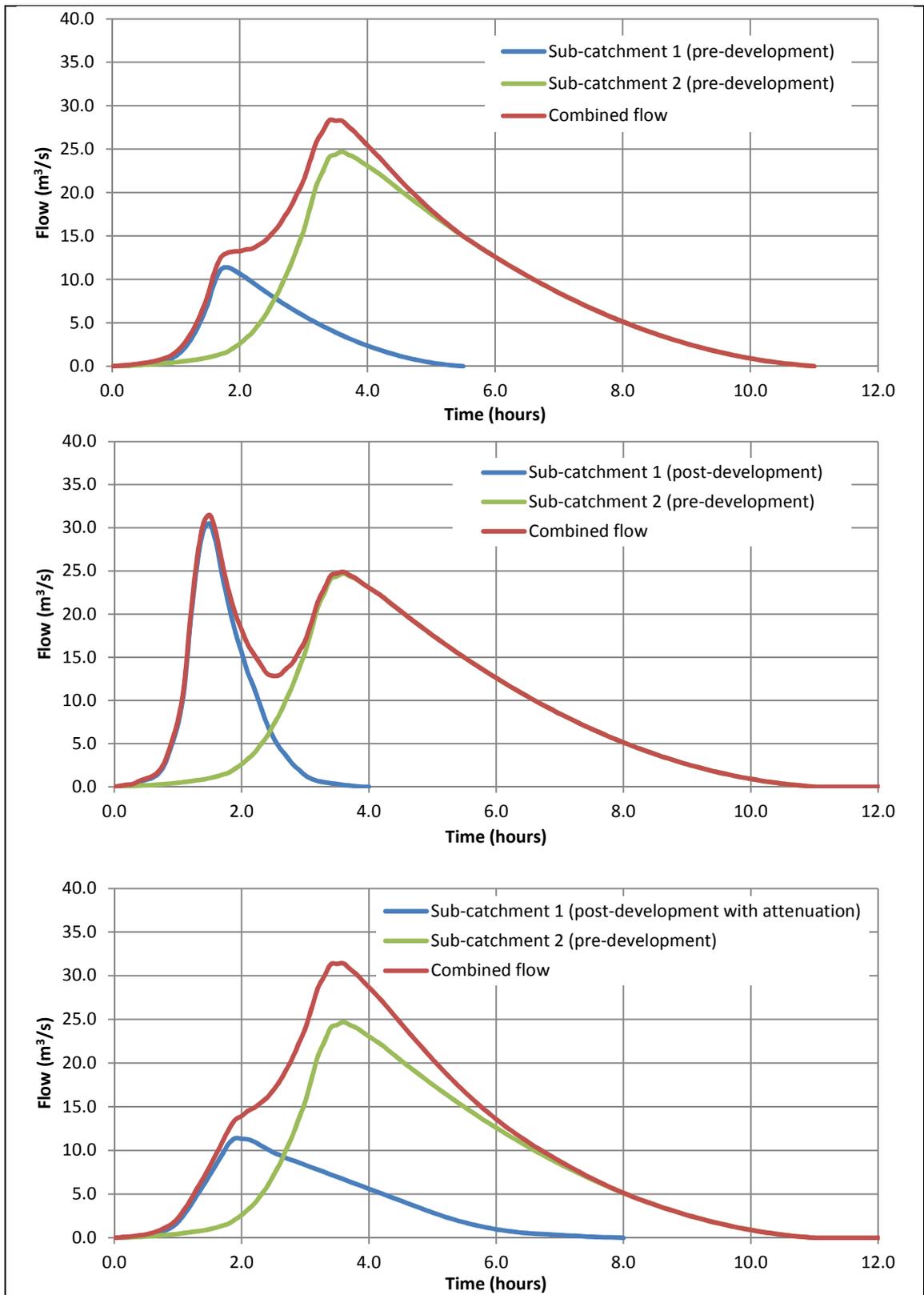
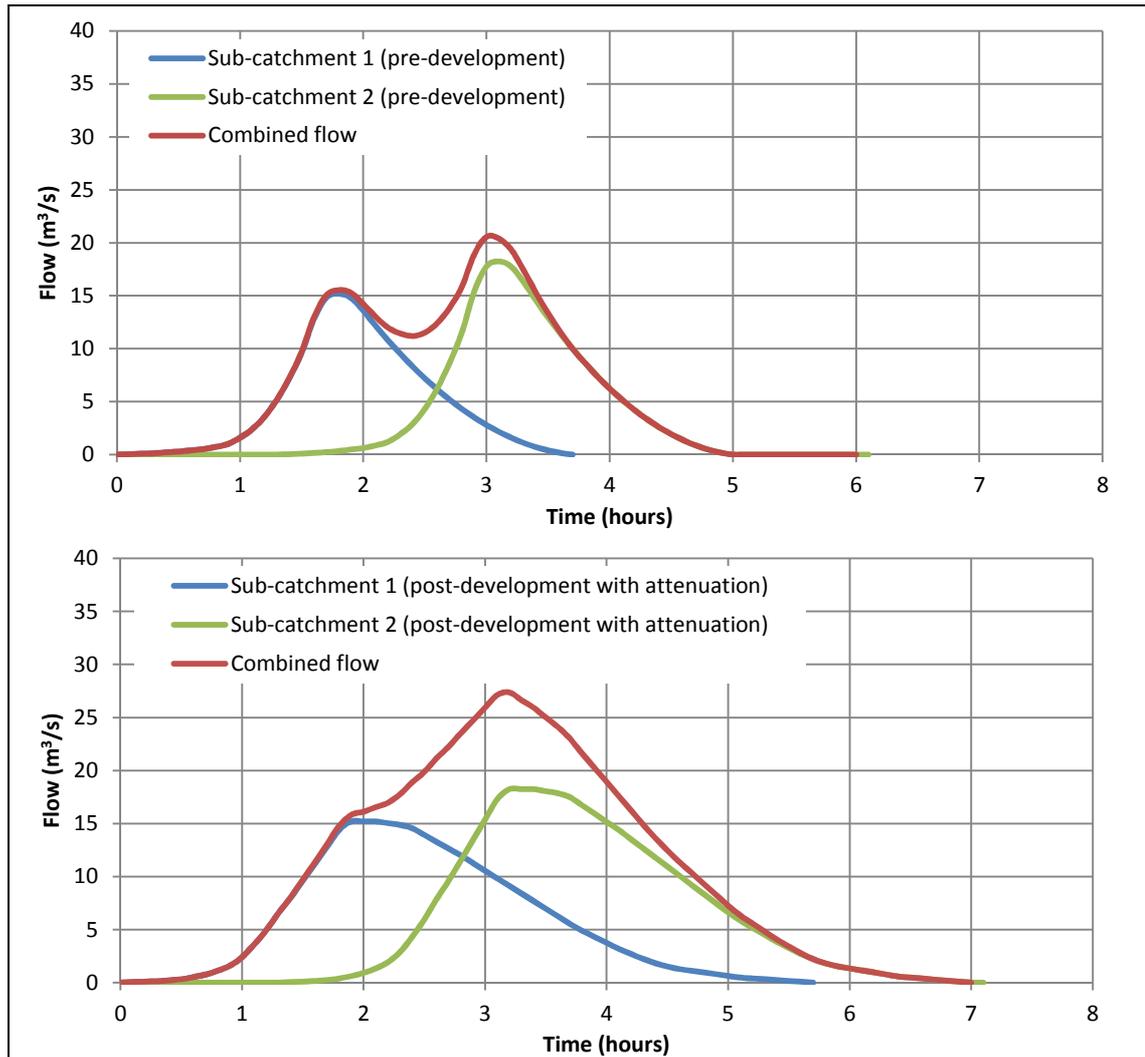


Figure 2.13: Effect of attenuation on runoff timing



**Figure 2.14: Cumulative effect of multiple tributaries with attenuation**

The Queensland Urban Drainage Manual (2007) indicates that research was carried out during the compilation of the Manual in order to determine design criteria for attenuation facilities which would avoid the cumulative impact of multiple attenuation facilities. It, however, concluded that no solution could be found, besides addressing runoff volume through aspects of WSUD which promote infiltration such as permeable paving, swales, filter strips, infiltration trenches and constructed wetlands. However, it is usually impractical or impossible to implement sufficient BMPs to reduce runoff volume to pre-development levels across an entire catchment (Petrucci et al., 2011).

### 2.8.6 Downstream assessments

As alluded to in **Sections 2.8.4** and **2.8.5**, in the design and implementation of an attenuation facility, it is important to not only evaluate the facility in isolation in terms of its effect in the

immediate vicinity of the facility, but to also assess the potential broader effect that it would have operating in conjunction with multiple facilities, both existing and future (Lee et al., 2012).

The best way to gain a better understanding of the unique catchment dynamics is to compile a catchment-wide integrated hydrology-hydraulics model of the urban drainage system (Faulkner, 1999; Queensland Government, 2007). There are a number of highly sophisticated software packages available, however the difficulty lies in sourcing the required data and estimating the model's input parameters to accurately create a realistic representation of the catchment behaviour. The overwhelming amount of resources required to effectively undertake a catchment-wide modelling study means that in the past they have rarely been carried out in most countries (Wang and Jin, 2001).

In the absence of a master plan, the Knox County (2008) and Georgia (2001) Stormwater Management Manuals suggest that the effect of development be determined at a number of locations downstream from the site, down to a point where the development site comprises 10% of the total catchment area (referred to as the "Ten-Percent Rule"). It is important that this assessment looks specifically at critical points downstream from the site, such as at tributary junctions and bridge structures. If all adverse impacts in terms of increased peak flows are mitigated up to this point, it is unlikely that the development will have any impact further downstream.

## **2.9 A paradigm shift for attenuation controls in South Africa**

### **2.9.1 Current flood attenuation practices in South Africa**

Historically, urban stormwater systems, usually in the form of "hard" infrastructure, have been implemented in South Africa without much prior catchment planning, and without much thought to the impacts caused by the significant changes in flow regimes downstream. Uncontrolled urban runoff has the potential to severely alter the frequency of flooding downstream, as well as cause irreversible environmental damage to receiving watercourses as a result of erosion and pollutants (Haubner et al., 2001). The natural progression from this approach has been the implementation of flood mitigation and erosion protection measures on an ad-hoc basis, still with no real concern for the impacts which are effectively shifted downstream. These poor stormwater management practices have left a legacy so that today the large majority of urban watercourses in South Africa are in a very poor condition from an ecological perspective (City of Johannesburg, 2003; eThekweni Municipality, 2006; River Health Programme, 2005). Significant lengths of these urban watercourses have been modified to the extent that they are drastically changed from their natural state in terms of alignment, cross-section, vegetation, water quality, and flow regimes.

**Section 2.6** of the literature review, which discusses current stormwater related guidelines, policies and legislation in South Africa, shows that there has been a definite shift away from the conventional approach to stormwater management in the last 20 years. However, in comparison to the apparent growing technical knowledge and institutional resources available to stormwater practitioners in countries such as the USA, the UK, and Australia, South Africa is still a long way behind. Nevertheless, it appears that there has been positive progress in this regard in recent years, especially evident in the recent publication of *The South African Guidelines for Sustainable Drainage Systems* (Armitage et al., 2013a), which provides a broad overview of many SuDS options and associated conceptual design criteria applicable in South Africa.

Attenuation of runoff through the use of storage facilities with controlled outflows is an important component of sustainable urban drainage systems, especially for the control of moderate to large flood events (greater than the 1-year recurrence interval event). In South Africa, however, the implementation of attenuation facilities has historically followed a simplistic and narrowly focussed approach which does not achieve holistic catchment management goals. Monk and Till (2006) argue that the key to achieving the goals of a sustainable drainage system in terms of water quality and biodiversity, is to maintain, or closely mimic, pre-development flow regimes for all storm events.

Two significant short-comings of past practices with regards to the implementation of flood attenuation facilities are, firstly, only considering the control of a single flood event, and secondly, the lack of catchment-wide modelling to inform the design process. These short-comings are discussed further in **Sections 2.9.2** and **2.9.3**.

### **2.9.2 Design of attenuation facilities for control of multiple storm events**

One of the best ways to achieve the objective of mimicking pre-development hydrology for a range of storm events is through attenuation facilities with multi-stage outlet structures, as discussed previously in **Section 2.8.2**. The *Knox County, Tennessee - Stormwater Management Manual* (Knox County, 2008) provides comprehensive design guidance regarding the multitude of design options available which can be used to achieve the objectives of attenuation facilities.

With the regulatory requirements for on-site attenuation currently in place, it is important that spatial planners, landscapers and stormwater engineers collaborate to conceptualise innovative designs for attenuation facilities which not only assist in mitigating stormwater impacts downstream, but also provide open spaces that enhance the community in which they are placed. Attenuation can be implemented in a number of ways, including in the form of a permanent pond with additional surcharge storage, a constructed wetland, or as a dry

pond which only fills up during certain flood events. In addition, for smaller areas, attenuation can also be implemented in the form of sub-surface storage, e.g. beneath permeable paving.

The placement of attenuation controls should be considered in conjunction with the stormwater system as a whole. *The South African Guidelines for Sustainable Drainage Systems* (Armitage et al., 2013a) recommends that attenuation forms part of a “train” of stormwater controls which are located near the runoff source, at a local (site) level and at a regional level. However, it is important to understand how these controls work in conjunction with each other and how they interact with other sub-catchments draining to the same watercourse.

### 2.9.3 Implementation of catchment-wide modelling

In the past, and to some extent even today, it has been common for attenuation facilities to be designed in isolation using simplistic methods (such as the Rational Method) of calculating peak flows and runoff volumes (Armitage et al., 2013b). However, with current regulatory requirements for on-site attenuation being implemented with most new developments, it is important that authorities have an understanding of the broader changes that are occurring to the catchment dynamics. There is a lot of evidence in the available literature (e.g., in the *Georgia Stormwater Management Manual* (Haubner et al., 2001), the *Queensland Urban Drainage Manual* (Queensland Government, 2007), and the *Red Book* (CSIR, 2000), amongst others) for the value of conducting catchment management plans (or stormwater master plans), including stormwater modelling of entire catchments which form part of urban areas.

Probably the main reason why catchment-wide modelling is not undertaken more often by local authorities in South Africa is because detailed stormwater modelling is a very resource intensive exercise. It can be costly and sometimes inaccurate if there is not sufficient reliable data, as indicated by Urbonas and Wre (2007), discussed previously in **Section 2.8.6**. Nevertheless, with the complexities presented by urban runoff and stormwater routing through multiple conveyance structures, it is argued that catchment-wide modelling, with the support of the required input data, is a critical requirement for effective stormwater management.

In addition, catchment-wide modelling should be a tool which is used to inform future development planning and associated stormwater infrastructure design. As catchments develop further, or as more data becomes available, these models should ideally be updated regularly and recalibrated to ensure that stormwater impacts are proactively and holistically managed.

### 3. Case studies

#### 3.1 Introduction

The literature review in **Chapter 2** has described the complex interaction between urban development and stormwater hydraulics, as well as how appropriately designed flood attenuation controls can assist in limiting the negative effects of post-development stormwater runoff.

Following from the literature review, three catchments in the vicinity of Cape Town, described in **Section 3.2**, were simulated with a selected stormwater modelling software package in order to evaluate various flood attenuation practices which are currently in use in South Africa. The choice of the modelling software and the main assumptions with respect to the modelling parameters are discussed in **Sections 3.3** and **3.4**, respectively. **Sections 3.5, 3.6,** and **3.7** present the model verification procedures and parameter sensitivity assessments undertaken for the models of the three case studies.

The main aims of the case studies are summarised as follows:

- To quantify the effects that attenuation of flows can have further downstream in the system by means of detailed catchment-wide stormwater modelling.
- To evaluate the impact that development has on a catchment's flood hydrology and test the impact in terms of its sensitivity to the catchment's coverage of impervious surfaces.
- To evaluate the effect of various attenuation practices in terms of their capability to mimic pre-development hydrology.
- To quantify the effect that attenuation can have on flow hydrographs in terms of total catchment dynamics.

The above outcomes were achieved by varying the way in which floods are routed through attenuation facilities based on a number of design scenarios, discussed in more detail in **Section 3.8**.

#### 3.2 Selection of study areas

As mentioned in **Section 3.1**, three catchments in the vicinity of Cape Town were used for the purpose of the research work done in this thesis. The author of this thesis has been involved in a number of stormwater master planning and infrastructure design projects across the City of Cape Town Metropolitan Area in the past, which has provided some insight into some of the concerns and challenges related to stormwater management in different geographical areas of the city.

In particular, the project commissioned by the City of Cape Town (CCT) entitled *Investigation into the Medium to Long Term Growth Options for Cape Town – Bulk Infrastructure Review* (City of Cape Town, 2012a) highlighted the need for clear planning and stormwater management guidelines as future development continues to impact on catchments which are currently mostly undeveloped. The future development identified in the above-mentioned study is concentrated in areas to the north of the existing urban edge along two corridors – one further east, north of Durbanville and Kraaifontein, and one further west, north of Blaauwberg, as indicated in **Figures 3.1** and **3.2**.

Based on the above-mentioned planning, the first study area, the Mosselbank River Catchment, was chosen since it falls within the eastern development corridor. The second study area, the Bayside Canal Catchment, falls within the western development corridor and represents very different catchment characteristics in comparison to the Mosselbank River Catchment. The third study area, the Upper Kuils River Catchment, was evaluated in terms of the performance of existing attenuation facilities in an area which is already almost completely developed. **Sections 3.2.1** to **3.2.3** describe the study areas in more detail.

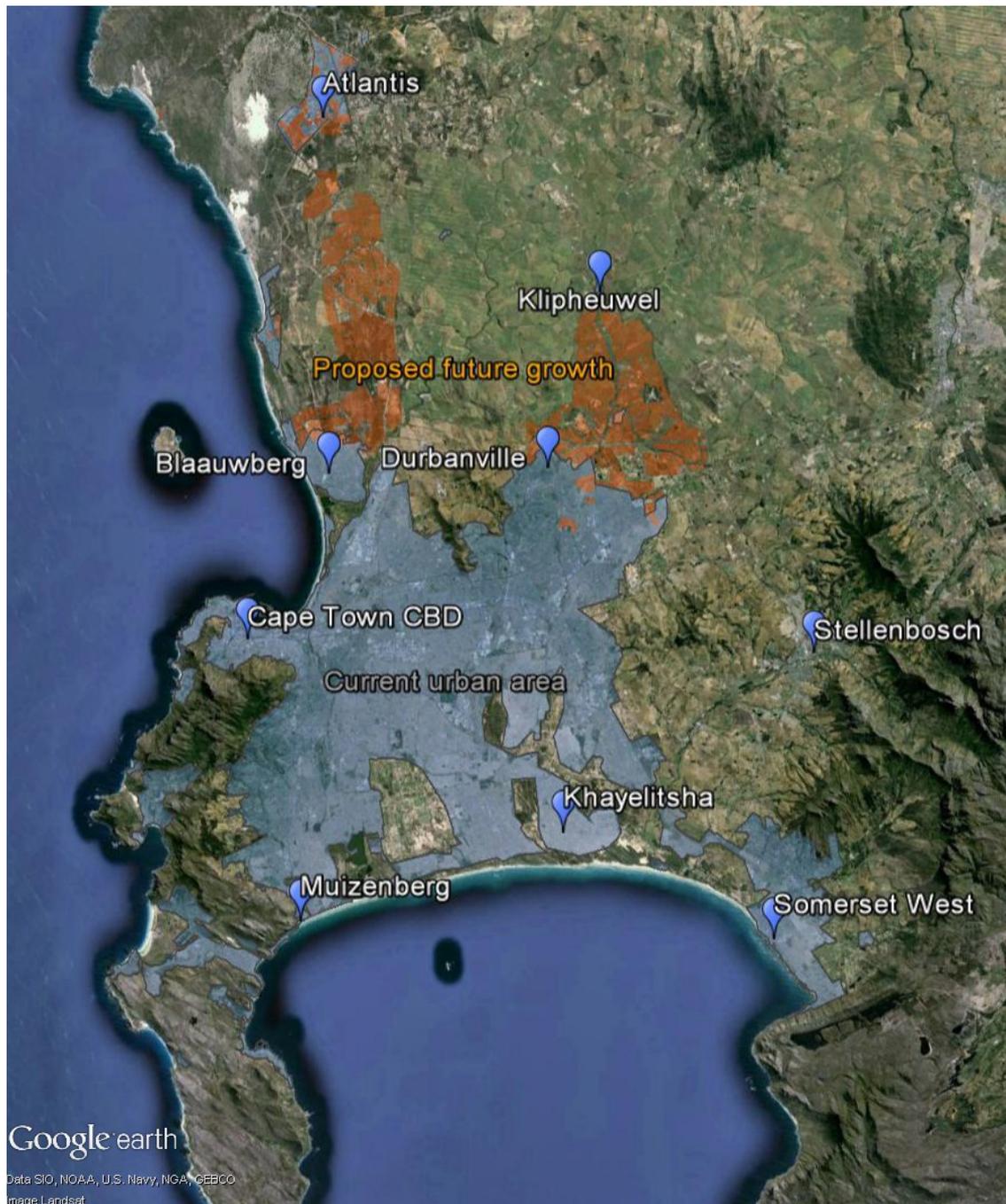
### **3.2.1 Mosselbank River Catchment – Kraaifontein/Klipheuwel**

The base model of the Mosselbank River Catchment was initially compiled by the author as an employee of AECOM SA (Pty) Ltd (AECOM). The base model formed part of the CCT's project entitled *Investigation into Medium to Long Term Growth Options for Cape Town – Bulk Infrastructure Review* (City of Cape Town, 2012a), with its initial purpose being to estimate the required bulk stormwater requirements for the Mosselbank River Catchment associated with the anticipated medium to long-term development illustrated in **Figure 3.2**. With permission from the CCT and AECOM, the base model has been used, and updated by the author as required, for the research outlined in this thesis.

#### **3.2.1.1 Description of catchment**

The Mosselbank River is one of the major tributaries of the Diep River which drains a total catchment area of 1 550 km<sup>2</sup>, discharging into Rietvlei at Table View/Milnerton, before flowing into Table Bay. The Mosselbank River Catchment is approximately 164 km<sup>2</sup>, and comprises of areas of Durbanville and Kraaifontein in the upper reaches, and drains in a northerly direction towards Klipheuwel where it joins the Klipmuts River.

Currently, only approximately 20% of the Mosselbank River Catchment area consists of urban development, with the remaining area consisting predominantly of agricultural lands cultivated with crops such as wheat. The catchment is generally quite flat or gently undulating and the soil texture is classified as sandy clay loam resulting in moderate stormwater runoff potential.



**Figure 3.1: Future growth corridors for Cape Town as proposed by the CCT (City of Cape Town, 2012a)**

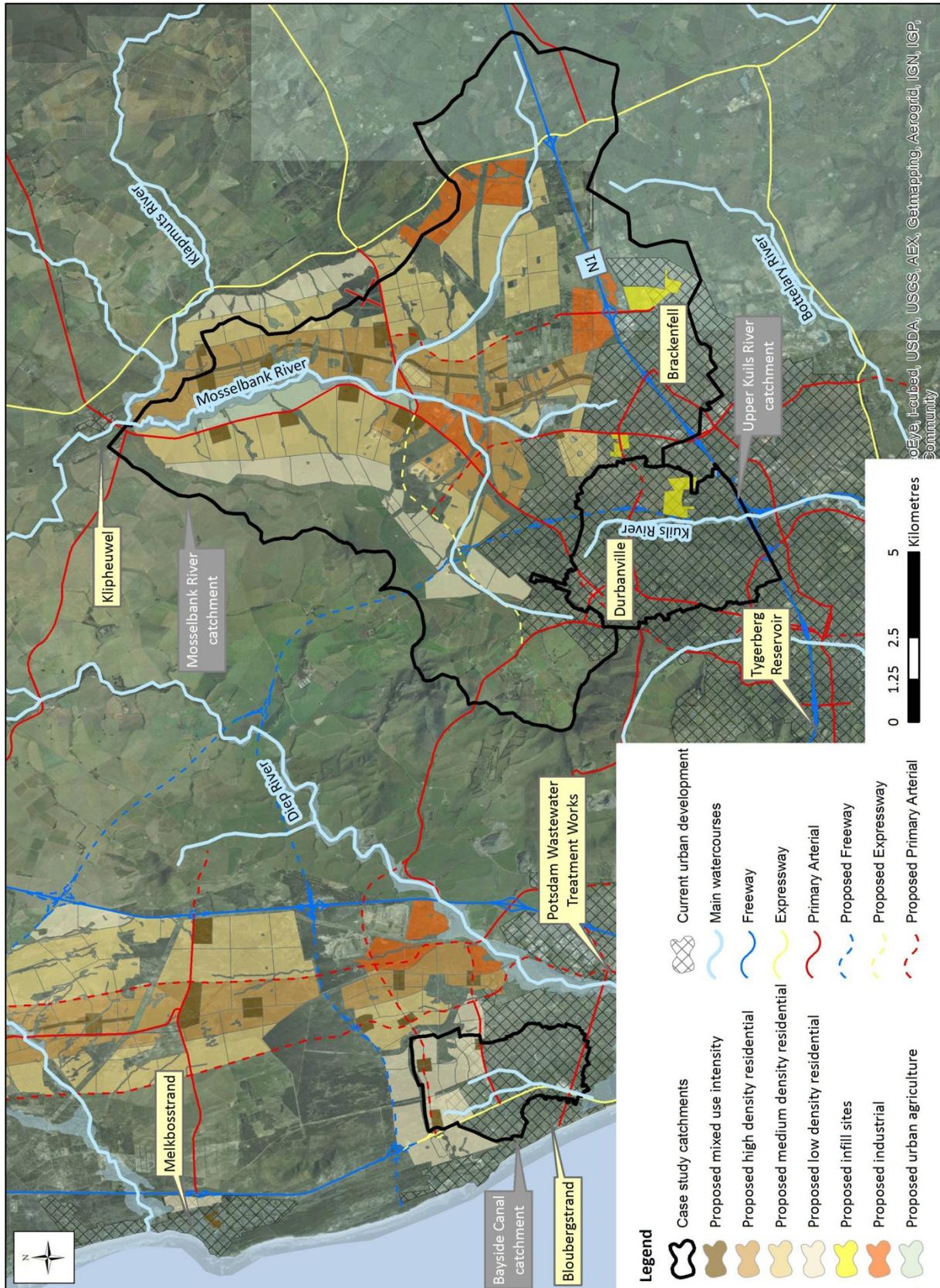


Figure 3.2: Locality plan of the three case study catchments (Mosselbank River, Bayside Canal and Upper Kuils River)

### 3.2.1.2 Development within the catchment

Cape Town is one of the fastest growing cities in South Africa with the 2011 National Census estimating the population to be approximately 3.7 million, up from the previous estimate of 2.9 million in 2001. In the areas which fall within the Mosselbank River Catchment area, the CCT estimates that the population has increased by approximately 70% between 2001 and 2011 (data from [www.capetown.gov.za](http://www.capetown.gov.za)). This growth has in turn created a market for development opportunities.

Over the last two decades, areas which have historically been used for agricultural purposes in the vicinity of Durbanville and Kraaifontein have been developed, primarily for medium to high-income residential use. There are also a number of large development proposals in the area which are going through the planning and authorisation processes (City of Cape Town, 2011). The CCT have identified the area north of Durbanville and Kraaifontein as one of the most logical areas for expansion of the current urban edge of Cape Town. In fact, almost the entire Mosselbank River Catchment may be developed in the long-term according to the CCT's study entitled *Investigation into the Medium to Long Term Growth Options for Cape Town* (City of Cape Town, 2012a)

The above-mentioned study also provided a preferred future development layout which took into account desirable land use proportions for the region, including a range of residential densities for different household income categories, as well as commercial and industrial development nodes which would provide work opportunities in the region. This development layout, illustrated in **Figure 3.2**, was used as a basis for the modelling of post-development scenarios of the Mosselbank River Catchment.

### 3.2.2 Bayside Canal Catchment – Blaauwberg

The model of the Bayside Canal Catchment was initially compiled as part of the design for the upgrading of the Bayside Canal undertaken by AECOM (then BKS (Pty) Ltd) in 2009. The model was subsequently updated by the author for the purpose of this research based on master planning information sourced from the CCT (City of Cape Town, 2006).

#### 3.2.2.1 Description of catchment

The Bayside Canal Catchment is approximately 12.6 km<sup>2</sup> and drains a large portion of the Blaauwberg area in a southerly direction to Rietvlei, part of the Diep River's estuary system.

The current urban area draining into the Bayside Canal system is approximately 9.5 km<sup>2</sup>, a portion of which has been developed since 2002. There are also short to medium term plans to develop the catchment to a total area of approximately 13.4 km<sup>2</sup>. Urban development within the catchment is predominantly medium to high income housing with a number of

commercial and mixed use areas. The bulk stormwater system consists of a range of underground and open channel components, including a number of attenuation facilities.

The Blaauwberg area generally has less rainfall compared to most other areas of Cape Town, and lower rainfall intensity. In addition, the natural ground surface has a low stormwater runoff potential since it generally consists of well-established sand dunes covered by low shrubs (fynbos). The result of this is that there are very few natural watercourses, and a lot of rainfall either infiltrates or drains to low-lying basins which become temporary wetlands during the wet winter period.

Prior to the commencement of development in this area the ground surface often needs to be reshaped so that it is possible for urban infrastructure to be constructed. This includes stormwater infrastructure which must be installed to avoid unwanted ponding and associated flooding. Post-development stormwater runoff ultimately needs to be conveyed to constructed watercourses or to large wetlands which feed aquifers.

In the case of the Bayside Canal catchment area, a system of canals has been constructed to convey most of the runoff to Rietvlei. Some sections of these canals have been constructed to appear as natural streams and have provided the community with beautiful public open spaces.

### 3.2.2.2 Development within the catchment

As mentioned to in **Section 3.2.2.1**, a portion of the existing urban area in the Bayside Canal Catchment has been developed in the previous 10 to 15 years. Development is ongoing in the upper portion of the catchment and it is anticipated that the entire catchment will be developed in the short to medium term. Based on the census data for the CCT Ward 107, which includes the areas of Parklands, Sunningdale, and a portion of Tableview, the CCT estimates that the population has increased by approximately 193% between 2001 and 2011 with over 10 500 additional dwelling units being constructed in the area (data from [www.capetown.gov.za](http://www.capetown.gov.za)).

The CCT's study entitled *Investigation into the Medium to Long Term Growth Options for Cape Town* (City of Cape Town, 2012a) provided a preferred future development layout for the undeveloped areas of the Bayside Canal Catchment which took into account desirable land use proportions for the region, illustrated in **Figure 3.2**. This development layout was used as a basis for the modelling of post-development scenarios of the Bayside Canal Catchment.

### 3.2.3 Upper Kuils River Catchment – Durbanville

The model of the Upper Kuils River Catchment was initially compiled by AECOM (then BKS (Pty) Ltd) as part of the study entitled *Eastern Catchments High Level Masterplan*

commissioned by the CCT (City of Cape Town, 2013). The model based on the existing development scenario was not compiled by the author, however, various other modelling scenarios were derived from this model by the author for the purpose of this study in order to illustrate the effect of current attenuation in the stormwater system.

### 3.2.3.1 Description of catchment

The Upper Kuils River Catchment, north of the N1 National Road (N1), is almost completely developed, consisting of a large portion of the Durbanville area, including the central business district of Durbanville. Although the town of Durbanville dates back to the 1800s, the majority of the Upper Kuils River Catchment has developed in the last 50 years, with a small portion being developed only in the previous 20 years.

The catchment consists of gently undulating hills which generally have a moderate runoff potential in the higher areas and a low runoff potential towards the lower drainage corridors. The existing stormwater system within the catchment consists of a complex network of infrastructure which has been implemented with a mix of new and old design standards.

### 3.2.3.2 Development within the catchment

As mentioned in **Section 3.2.3.1**, the majority of the Upper Kuils River Catchment is currently developed, however, there is one significant site which has been identified for “infill” development by the CCT, as shown in **Figure 3.2**.

## 3.3 Selection of hydraulic modelling software

The evaluation of various attenuation practices requires extensive analysis of various design scenarios through the use of a stormwater modelling software package. The requirements for the software package included the following:

- A hydrology component that can simulate runoff generated from time-varying rainfall falling on multiple sub-catchment areas.
- A hydraulic routing component that can simulate flow through storage areas, watercourses and other open and closed conduits.

Based on the above criteria, there are a large number of suitable software packages available. Some of the more widely used ones are listed in **Table 3.1** below:

**Table 3.1: Hydrological modelling software packages**

Developer	Software package
US Environmental Protection Agency (EPA)	SWMM 5
Computational Hydraulics International (CHI)	PCSWMM
Danish Hydraulic Institute (DHI)	Mike Urban / Mike SWMM
XPSolutions	XPSWMM
Innovyse	Infoworks CS
Deltares	SOBEK
US Army Corps of Engineers	HEC-HMS

The US EPA's Stormwater Management Model (SWMM) software package has been in existence in some form from the early 1970s and is one of the most widely used and tested models of its type in the world today (Lockie, 2006). Since its inception, the software has been available in open source, and free of charge. This has meant that although the fundamentals of the model are technically sound, the software has a relatively poor user interface and minimal data management tools. This short-coming has been overcome by a number of software companies through the development of alternative SWMM Graphical User Interface (GUI) wrappers such as PCSWMM, MIKESWMM, and XPSWMM. Other, more advanced modelling packages, such as Infoworks CS and SOBEK offer various components of SWMM as an option when choosing a modelling approach in the software.

Of the above-mentioned software packages, the one which is arguably most widely used in South Africa for urban stormwater system modelling is PCSWMM. PCSWMM is well suited to the task of evaluating the effect of attenuation in a stormwater system, both at site level, and broadly across a large urban catchment. It uses the US EPA SWMM hydrology and hydraulic engine and has a user-friendly GIS mapping interface which integrates well with other GIS software and with Microsoft Excel. The software is specifically suited to urban catchment modelling and can be used for both single event and long-term (continuous) simulation of runoff quantity and quality (United States Environmental Protection Agency, 2009b).

The hydrology component of PCSWMM simulates the runoff generated from time-varying rainfall occurring over multiple sub-catchment areas. The quantity and quality of runoff from a given catchment will depend on the sub-catchment characteristics, such as impervious area, infiltration potential, rainfall interception, depression storage, and overland routing.

Runoff from sub-catchments will typically join stormwater systems consisting of a combination of closed and open conduits, and hydraulic structures, such as natural channels, culverts, storage facilities, weirs, and outlet structures, all of which can be simulated with the

hydraulic component of PCSWMM for various flow regimes, such as backwater, surcharging, reverse flow and surface ponding.

Other useful functions included with the PCSWMM software package are sensitivity analysis and calibration tools which allow the modeller to compare output data in graphical and numerical format for a range of input parameters. This is very useful when attempting to match recorded stream flow data to the output hydrograph from the model.

In addition to the above arguments in favour of using PCSWMM in the context of this study, it is noted that the author has a number of years of experience and has attended technical training in the use of the PCSWMM software package. PCSWMM is therefore well suited to assist with achieving the objectives of the case studies addressed in this thesis.

### **3.4 Modelling parameters**

#### **3.4.1 Calibration data limitations**

As discussed in **Section 3.3**, PCSWMM has both a hydrology component and a hydraulic routing component, each of which requires substantial reliable input data in order to provide accurate results. Even with the availability of most of the required input data, as is the case in Cape Town, the complex nature of hydrology and stormwater routing hydraulics inevitably means that there are still unaccounted-for unknowns in the model which should be tested through a calibration process whenever possible.

Although the CCT has a number of rainfall stations which are collecting reliable rainfall data in 5 minute intervals (refer to **Section 3.4.7**), there is a significant lack of reliable stream flow data with which calibration can be undertaken. This is noted as a major short-coming of many of the CCT's past stormwater master planning and flood assessment studies. As an example, the original Upper Kuils River Model (refer to **Section 3.2.3**), used for the purpose of this study, formed part of a larger model which simulated the entire Kuils River Catchment, including the Bottelary and Eerste Rivers (City of Cape Town, 2013). The original study made an attempt to calibrate the model with recorded data from a flow gauge just downstream of the confluence of the Kuils River and Bottelary River. This gauge was originally managed by the Department of Water Affairs in the 1970s and 1980s, but has subsequently been taken over by the CCT. However, the data from this station over the previous 15 years has either not been recorded or has been deemed unreliable. Due to this lack of reliable data, only one relatively small storm event from October 2004 was suitable for calibration purposes, however, due to concerns regarding the reliability of the data, and changes in the catchment since 2004, the model was not adjusted from its original set-up. Further discussion with respect to the calibration and verification of this model is presented in **Section 3.7**.

There was also an attempt to source flow data for the Bayside Canal and Mosselbank River. No data was available for the Bayside Canal, and although there was previously a flow gauge in operation near Klipheuwel, downstream of the confluence of the Mosselbank and Klapmuts Rivers, this has been out of operation since 1985, and no continuous data records could be obtained. Further discussion with respect to the verification of this model is presented in **Section 3.5**.

Due to the lack of reliable stream flow data, the models compiled as part of this study were therefore not calibrated. It is argued, however, that the absence of calibration data does not negatively affect the findings of this research since conclusions are drawn from the comparison of a number of hypothetical attenuation scenarios and not necessarily the individual model output data in isolation. In addition, the attenuation scenarios are simulated from a theoretical basis which looks to future options which are not yet in operation and could not have been calibrated even if there was data available.

The input data used in both the base hydrology and hydraulic routing components of the models was based on the best available information, described in **Sections 3.4.2 to 3.4.9**. **Sections 3.5, 3.6, and 3.7** present various procedures undertaken to verify the results of the three models and check the validity of the default modelling parameters.

#### **3.4.2 Sub-catchment delineation**

The sub-catchment boundaries for the three case studies were generally left unchanged from the base PCSWMM models, as described in **Section 3.2**. The original sub-catchment delineation process is summarised as follows:

- In the case of the Mosselbank River Catchment, the entire catchment area was divided into sub-catchments, typically less than 1 km<sup>2</sup> in area. Sub-catchments were generally defined using 5 m contour information sourced from the National Directorate of Geo-spatial Information, a division of the Department of Rural Development and Land Reform. The sub-catchment boundaries in existing urban areas were in some cases adjusted slightly to account for the changes in the natural drainage patterns resulting from the formal stormwater system. In areas which are currently undeveloped, the sub-catchments were assumed to remain unchanged with the development of a formal stormwater system in future. This is in line with the philosophy of the CCT's current *Management of Urban Stormwater Impacts Policy* (City of Cape Town, 2009b), which promotes minimal changes to the pre-development drainage patterns.
- In the case of the Bayside Canal Model, the natural topography and ground conditions are such that catchment boundaries are not easily defined, as alluded to in **Section 3.2.2**. Therefore, the sub-catchment boundaries were defined according to the master planning

conducted by CIVtech Consultants cc (City of Cape Town, 2006) which was typically based on development site boundaries and the placement of attenuation facilities to which each sub-catchment drained.

- The Upper Kuils River Model sub-catchments were defined using the CCT's LiDAR survey data and the available GIS data showing the existing stormwater system. The sizes of the sub-catchments defined as part of this model varied since they were typically only defined upstream of attenuation facilities.

### 3.4.3 Infiltration and interception

The runoff component of the PCSWMM models used the widely accepted Horton infiltration method (Horton, 1940; James et al., 2010). This method assumes that infiltration decreases exponentially during a period of rainfall from a maximum infiltration rate to a minimum rate. The rate at which this decrease occurs is based on a decay coefficient, and following a rainfall event, the time it takes for saturated soil to completely dry is given by a drying time parameter.

The infiltration rates applicable to each sub-catchment were broadly based on a geographical delineation of various soil types informed by the following sources:

- Report and associated GIS mapping data from the study *Water Resources of South Africa 2005* (Water Research Commission, 2005).
- GIS mapping data received from the Institute for Soil, Climate and Water (an institute of the Agricultural Research Council).
- Report entitled *Stormwater and River Management Stormwater Asset Management Plan (Phase 2B): High Level Master Planning for the Eastern Catchments - Aquifer Infiltration Potential and Contamination Risk Assessment* Compiled by Umvoto (Pty) Ltd on behalf of Arcus Gibb (Pty) Ltd for the City of Cape Town (City of Cape Town, 2012b).

Based on the above-mentioned sources, the runoff potential and range of infiltration rates are summarised for each of the case studies as follows:

**Table 3.2: Infiltration parameters**

Case study	Maximum infiltration rate (mm/hr)	Runoff potential description
Mosselbank River	2.2 – 3.2	Medium to high
Bayside Canal	8.4	Low
Upper Kuils River		
- Higher areas	2 – 5	Medium to high
- Lower areas	7 – 9	Low

Typical parameters for overland flow and depression storage were based on the recommendations of the User's Guide to SWMM5 (James et al., 2010), and have been summarised in **Table 3.3**.

**Table 3.3: Runoff parameters**

Modelling parameter	Value
<b>Manning "n"</b>	
- Impervious area	0.02
- Pervious area	0.2
<b>Depression storage (mm)</b>	
- Impervious area	1.3 – 2.0
- Pervious area	5.0 – 5.1

#### 3.4.4 Sub-catchment slopes

For both the Mosselbank River Model and the Upper Kuils River Model, the following empirical formula was used to determine the average slope of each sub-catchment (Giricke and du Plessis, 2012):

$$S_2 = \frac{M \cdot \Delta H \cdot 10^{-4}}{A}$$

Where

$S_2$	=	Average catchment slope (m/m)
M	=	Total length of all contour lines within the sub-catchment (m)
$\Delta H$	=	Contour interval (m)
A	=	Sub-catchment area (km <sup>2</sup> )

The above method could not be used in the case of the Bayside Canal Model since the natural pre-development topography is very varied due to the sand dunes which make up much of the area. In order to develop this area, bulk earthworks are generally undertaken to even out the

ground surface so that infrastructure and building development can be implemented. Therefore, the general slope of the catchment area, once bulk earthworks have been completed was estimated to be approximately 1%. A sensitivity assessment undertaken with PCSWMM indicated that variations to the catchment slope between 0.5% and 2% do not result in significant changes to the sub-catchment runoff response. Therefore, a slope of 1% was assumed for all sub-catchments in this area for the purpose of this study.

#### 3.4.5 Sub-catchment width

The sub-catchment width parameter effectively describes the shape of the sub-catchment which affects the runoff lag time, and hence, the hydrograph shape. It is calculated by dividing the catchment area by the average length of flow path which runoff needs to take between the watershed and main stream draining the sub-catchment. Therefore, a high sub-catchment width typically results in a faster runoff response time, while a low sub-catchment width typically results in a slower runoff response time and a flatter hydrograph. Model calibration can, to a certain extent, be adequately achieved by adjusting this parameter.

#### 3.4.6 Percent imperviousness

Typical values for the percentage of impervious areas within different land use developments were determined by analysing a number of existing developments within the Cape Town surrounds. **Table 3.4** shows the values adopted for the areas of the Mosselbank River and Bayside Canal models which have been identified for future development. In addition, also indicated in **Table 3.4** is the default percentage of runoff from impervious areas routed through pervious areas, as opposed to impervious areas draining directly into the stormwater system. The portion of runoff routed from impervious areas through pervious areas accounts for a number of the typical BMPs which are expected to be implemented within the sub-catchments (e.g. roof downpipes draining to lawns rather than directly to a stormwater system, bioretention areas, swales, etc.).

**Table 3.4: Stormwater catchment land use characteristics**

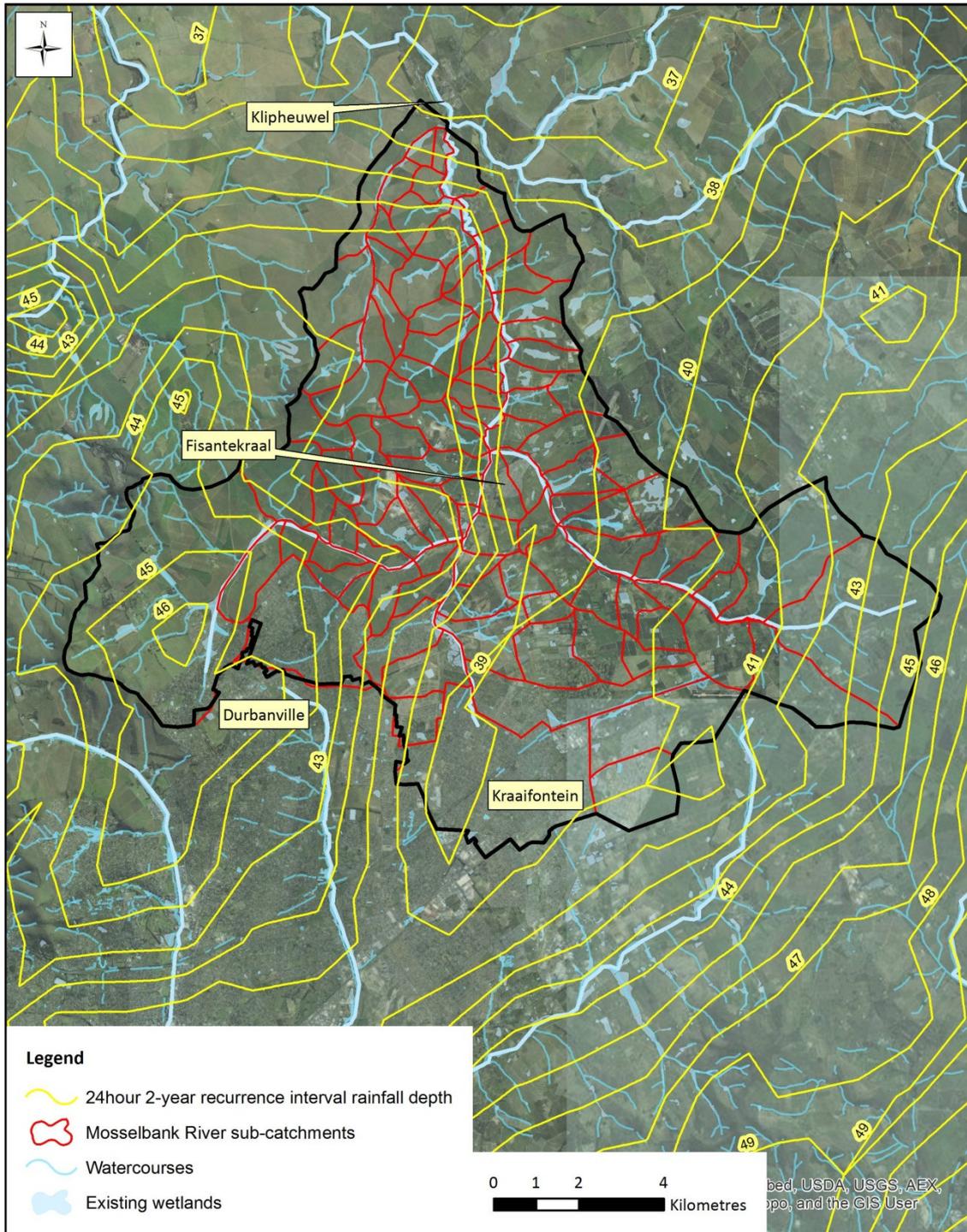
Runoff parameter	Land use				
	High density residential	Medium density residential	Low density residential	Mixed use intensity	Industrial
Imperviousness	75%	65%	40%	85%	95%
Runoff from impervious areas routed to pervious areas	30%	35%	45%	20%	10%

### 3.4.7 Rainfall

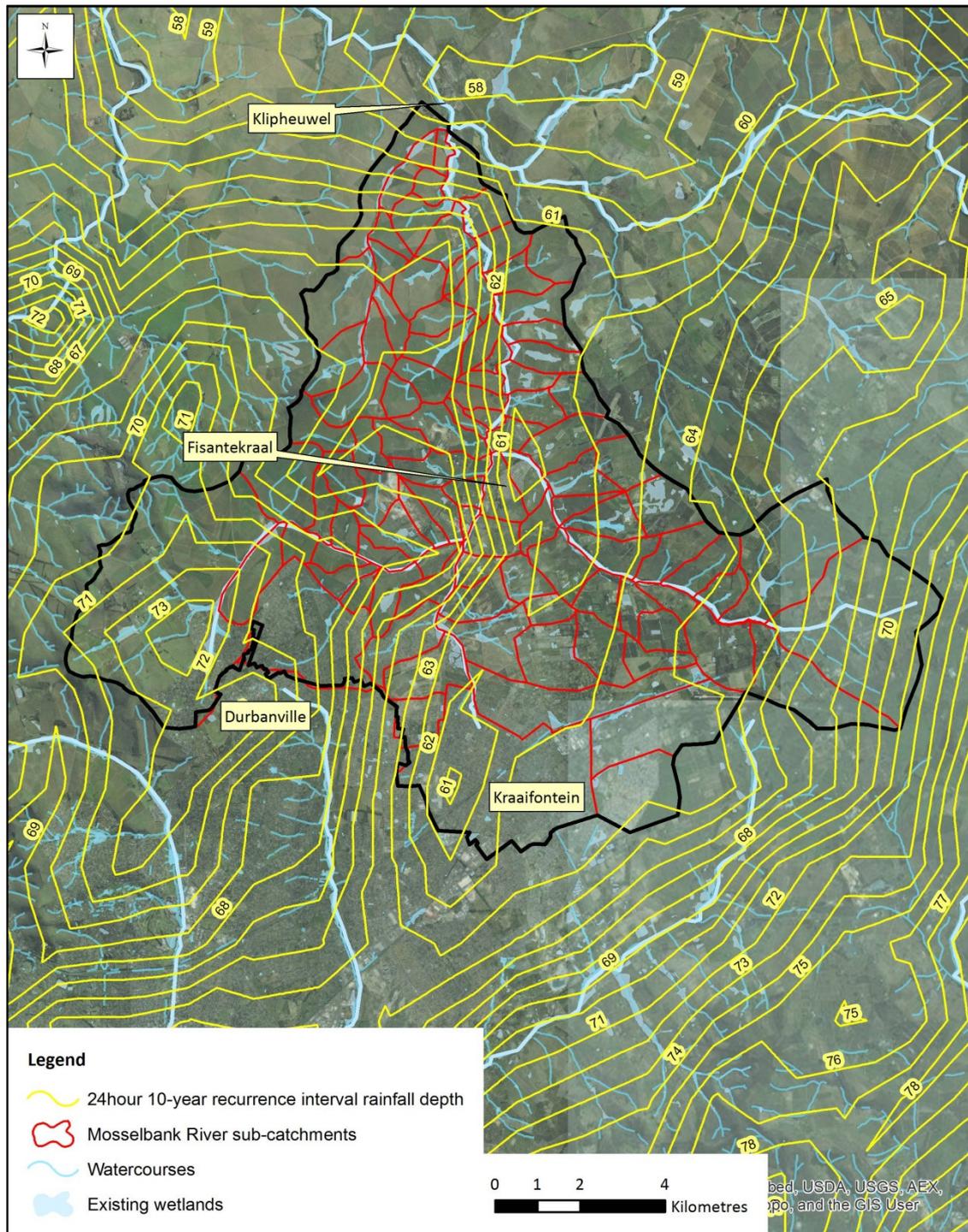
The average point rainfall values used for the models were estimated based on the rainfall data grid compiled for Cape Town as part of the study entitled *Impacts of Projected Climate Change on Design Rainfall and Streamflows in the Cape Town Metro Area* (Schulze et al., 2010), which was commissioned as part of the study entitled *Stormwater Infrastructure Asset Management Plan (Phase 2A) Rainfall Analysis and High Level Masterplanning, Salt River Catchment*, undertaken by SRK Consulting (South Africa) (Pty) Ltd for the CCT (City of Cape Town, 2012c). It is noted that this study explicitly addressed climate change and proposed that, for design purposes, short-term rainfall depths be increased by 15% from those determined with historical data; however, for the purpose of this research, rainfall depths have not been adjusted for potential climate change impacts.

All of the attenuation scenarios were evaluated based on the 2-year, 10-year and 50-year recurrence interval 24-hour South African SCS Type-1 design storms. These three recurrence intervals represent a wide range of possible storm magnitudes which will provide valuable insight with respect to the evaluation of flow attenuation practices. The South African SCS Type-1 design storm was found to provide a very close correlation to the available short-term data in the above-mentioned study.

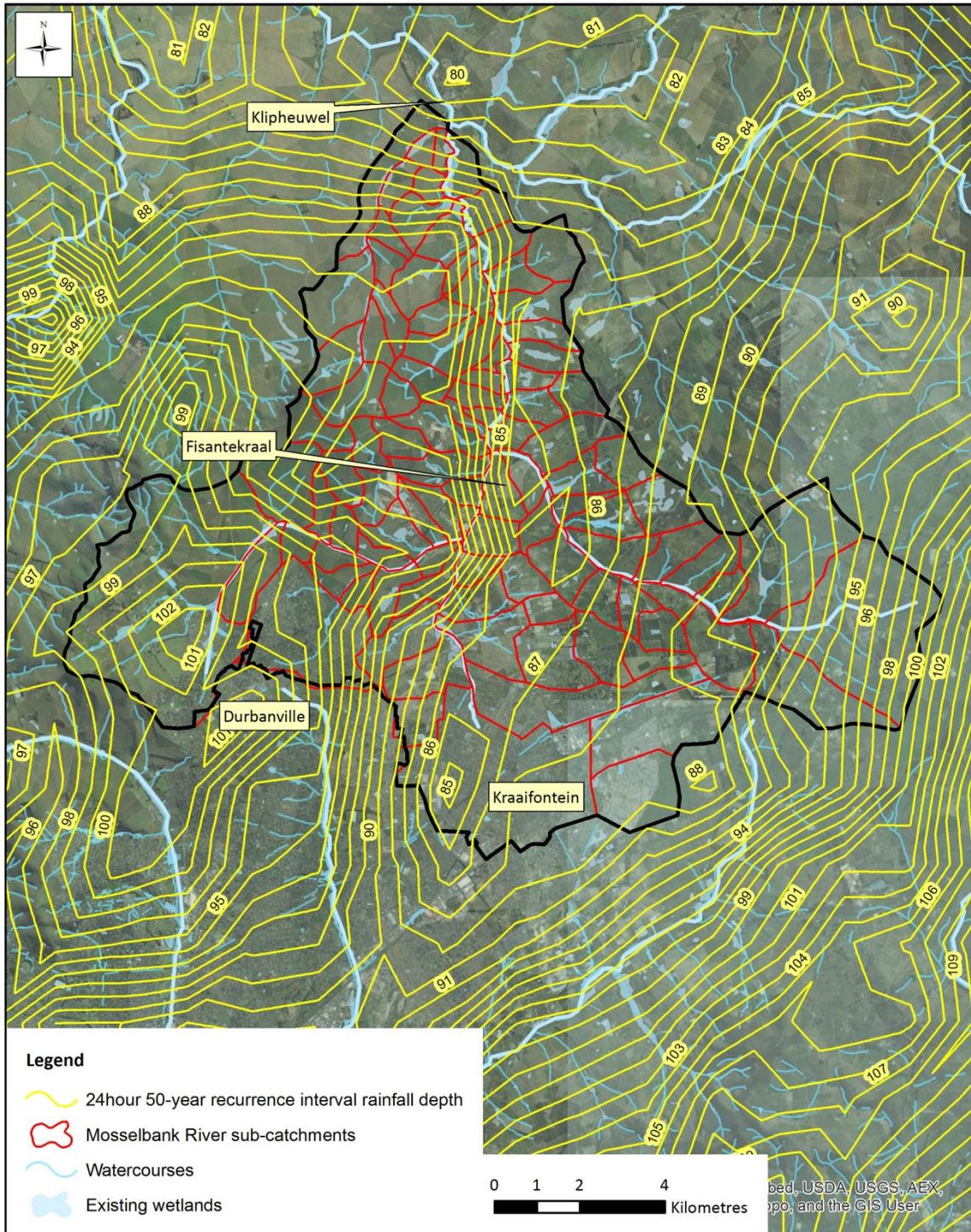
The above-mentioned rainfall data grid compiled for Cape Town was plotted in GIS and converted to isohyets depicting the total 24-hour rainfall depths for the 2-, 10- and 50-year recurrence interval storm events, as shown in **Figures 3.3 to 3.11**. The rainfall depths shown in the figures, rounded to the nearest millimetre, were assigned to each sub-catchment of the three case study models according to their location in relation to the isohyets.



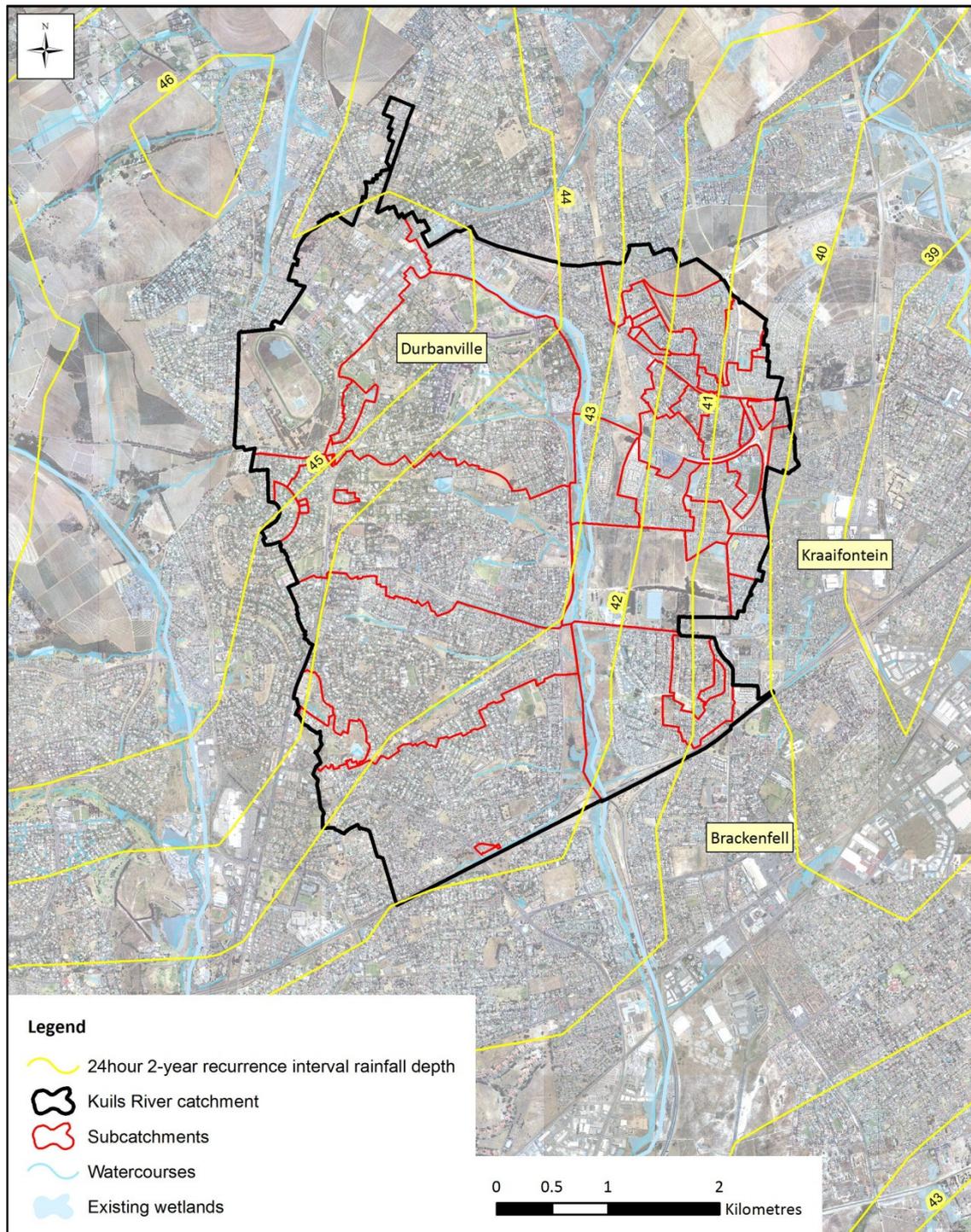
**Figure 3.3: 24-hour 2-year recurrence interval rainfall depths adopted for the Mosselbank River Model**



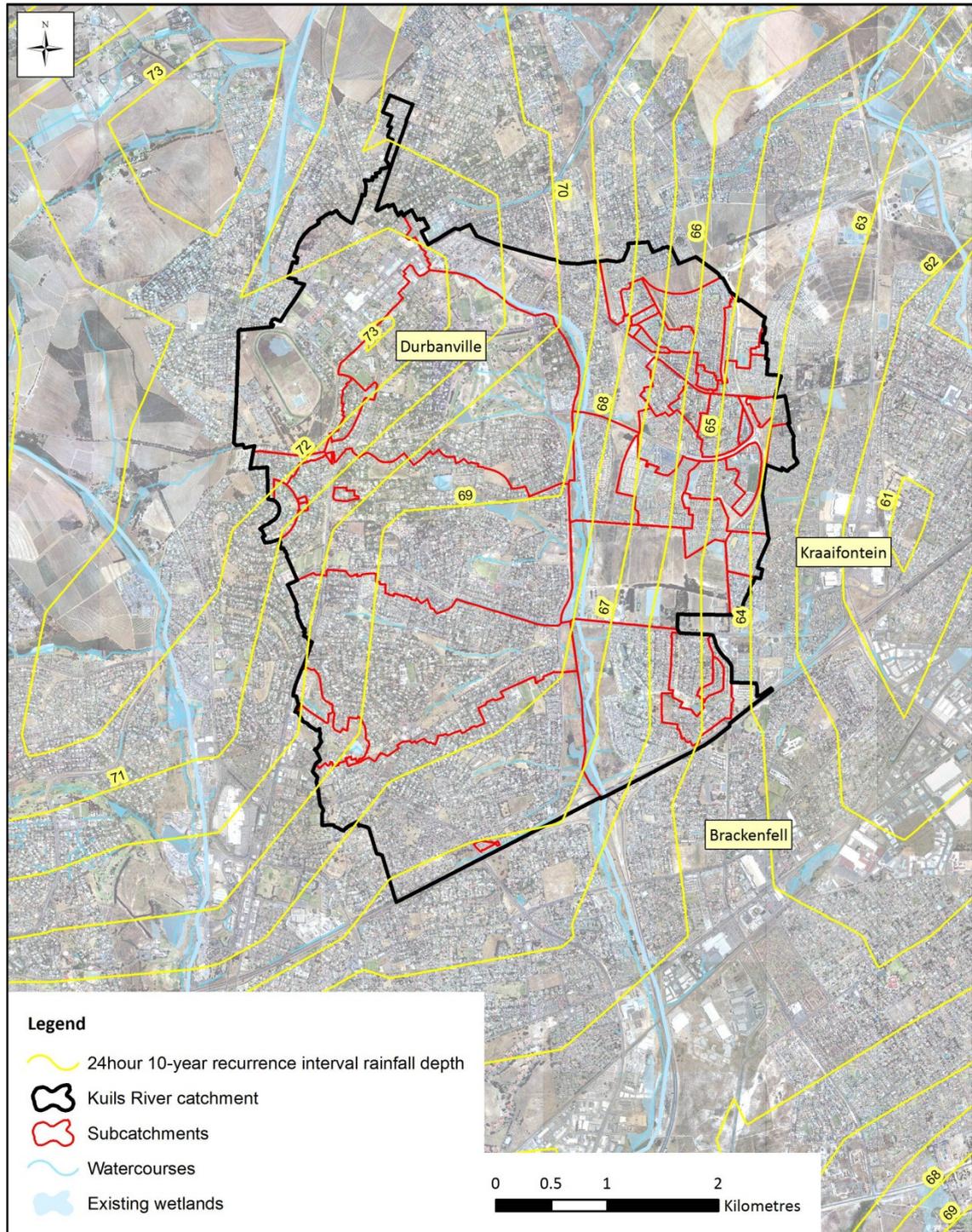
**Figure 3.4: 24-hour 10-year recurrence interval rainfall depths adopted for the Mosselbank River Model**



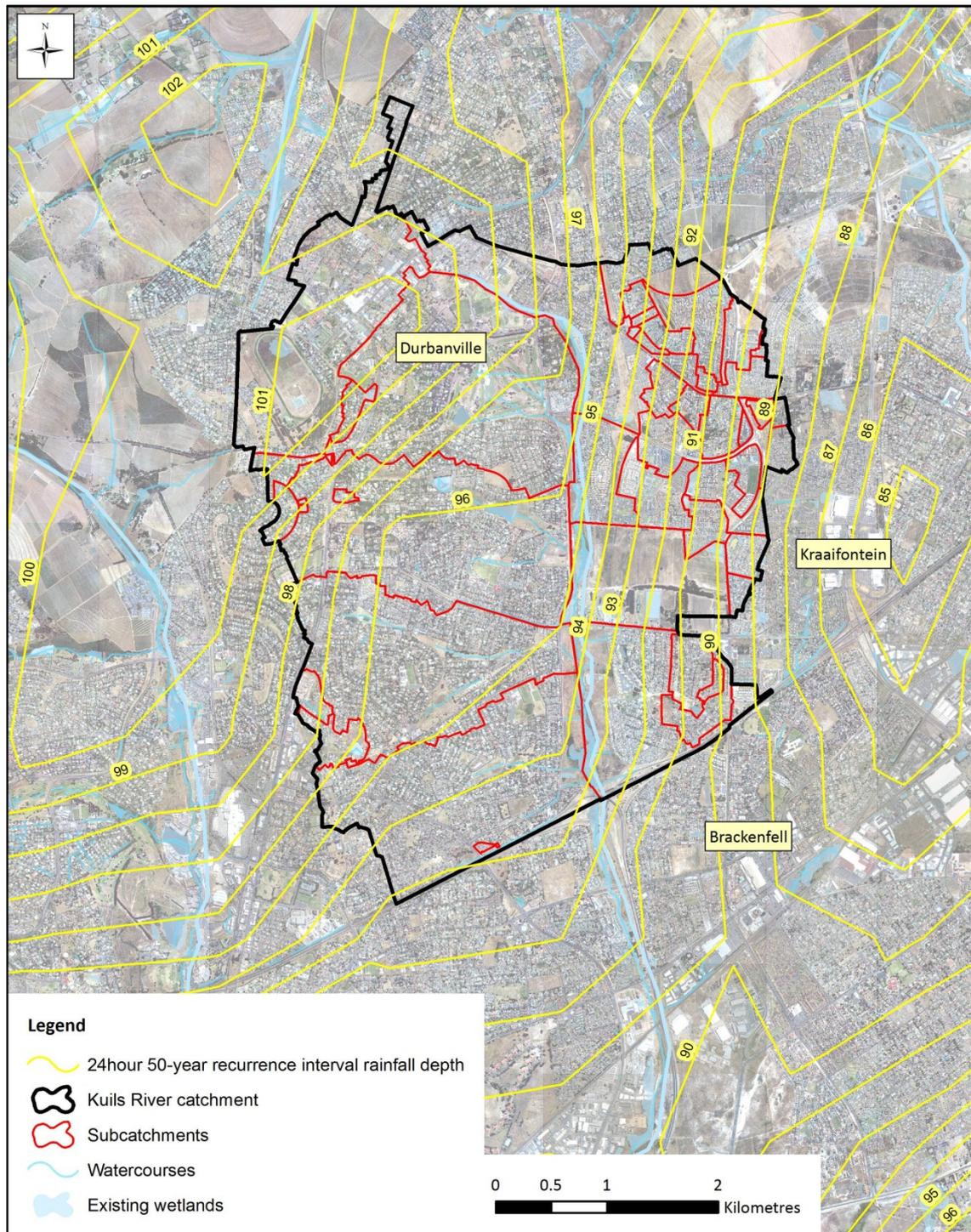
**Figure 3.5: 24-hour 50-year recurrence interval rainfall depths adopted for the Mosselbank River Model**



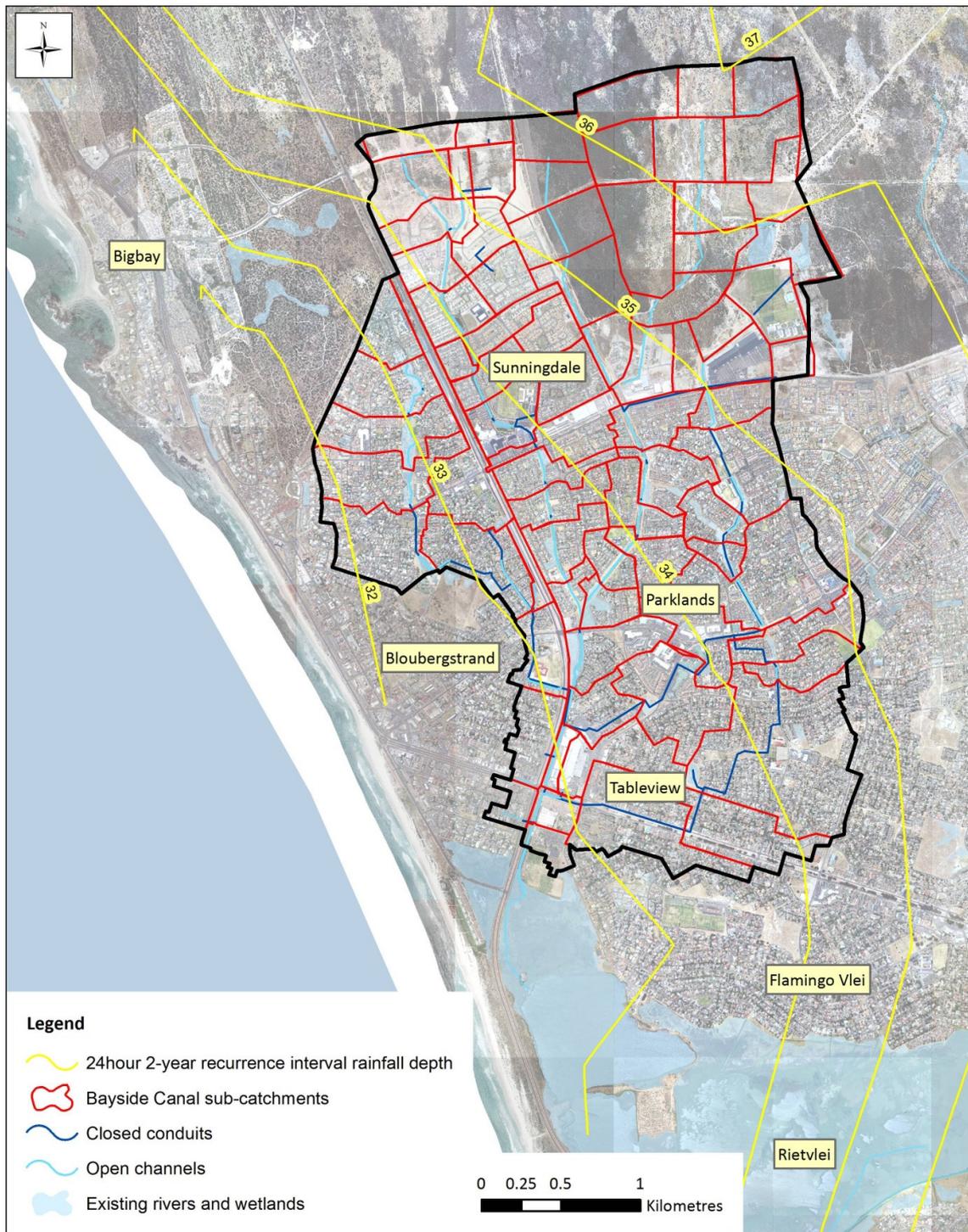
**Figure 3.6: 24-hour 2-year recurrence interval rainfall depths adopted for the Upper Kuils River Model**



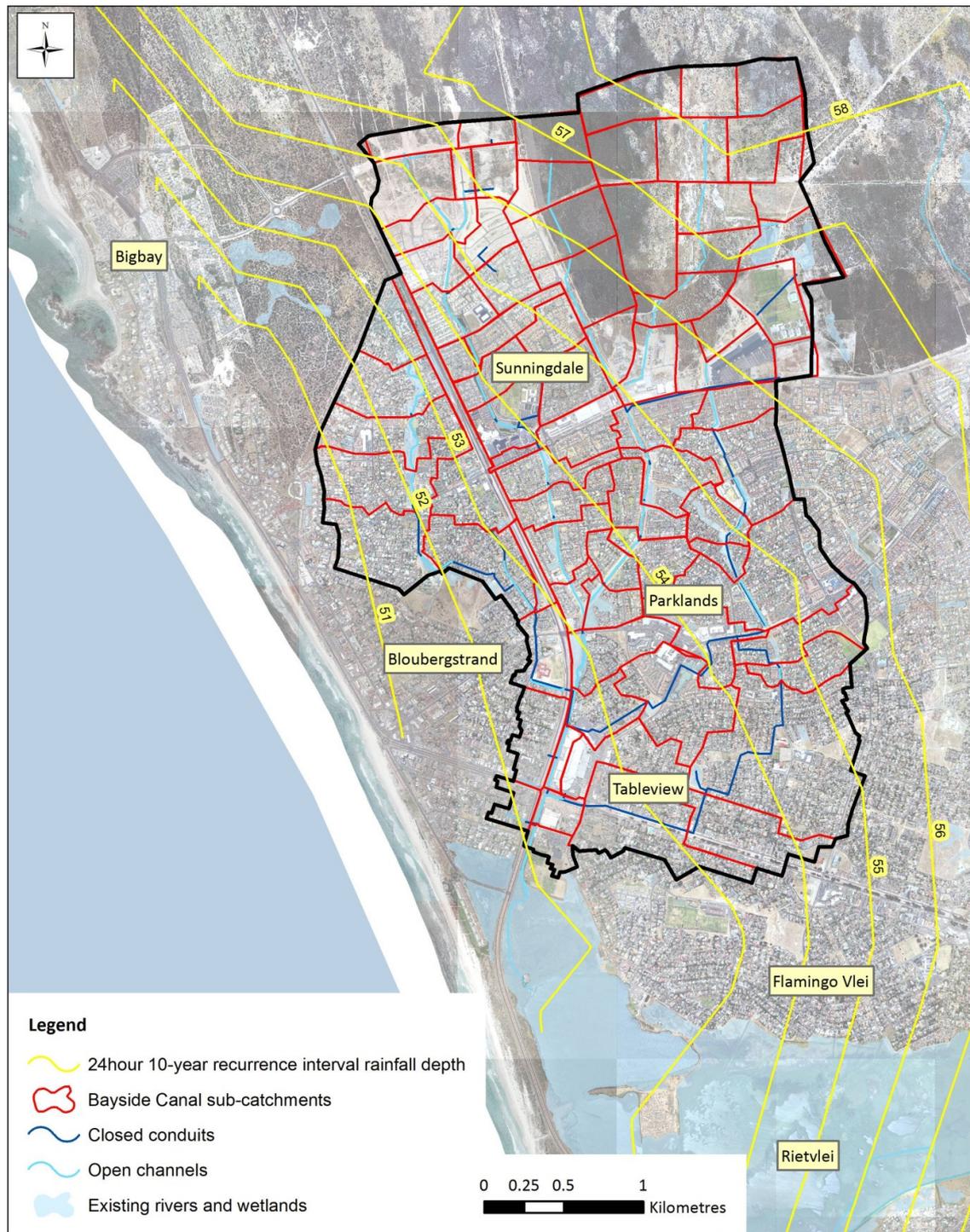
**Figure 3.7: 24-hour 10-year recurrence interval rainfall depths adopted for the Upper Kuils River Model**



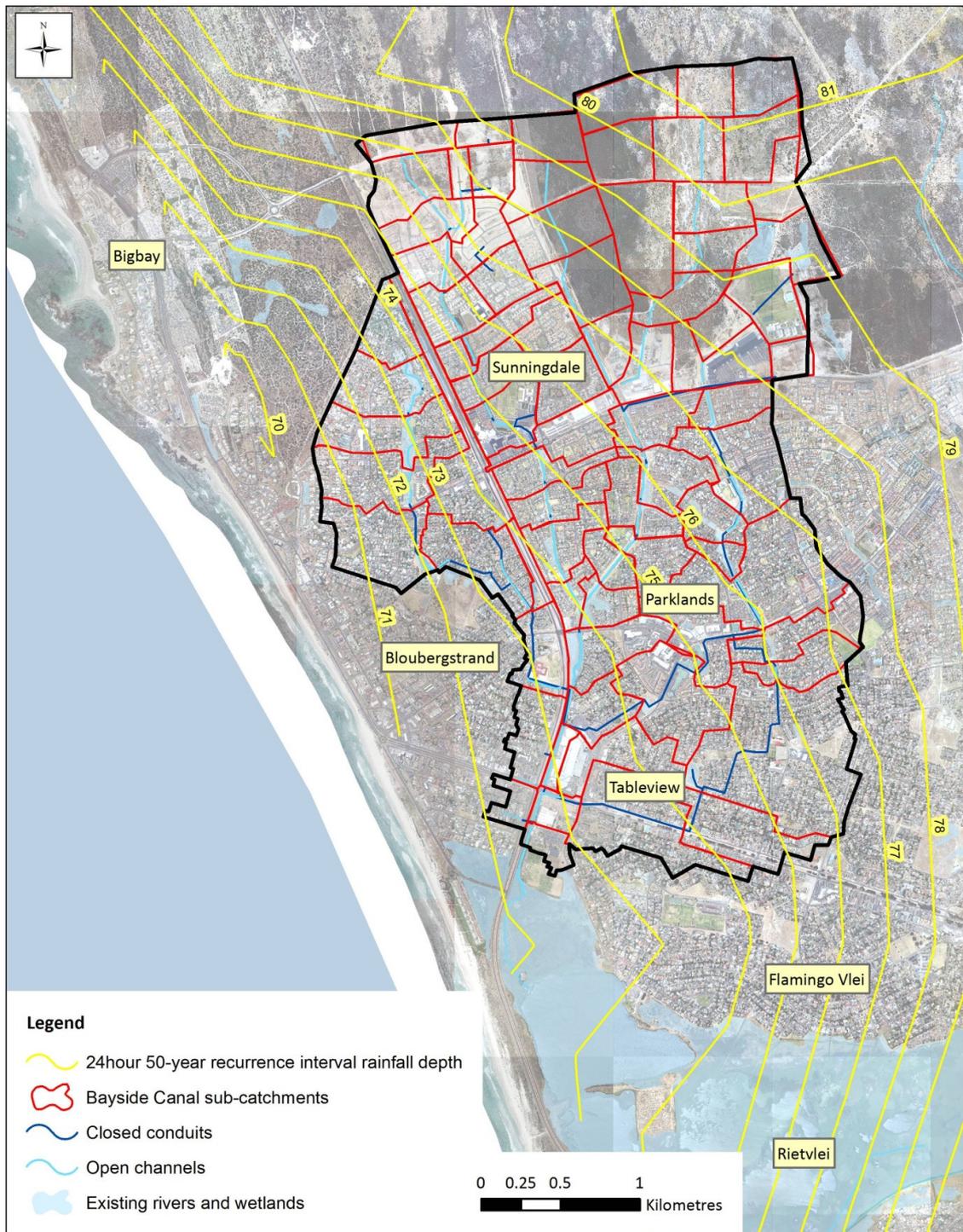
**Figure 3.8: 24-hour 50-year recurrence interval rainfall depths adopted for the Upper Kuils River Model**



**Figure 3.9: 24-hour 2-year recurrence interval rainfall depths adopted for the Bayside Canal Model**



**Figure 3.10: 24-hour 10-year recurrence interval rainfall depths adopted for the Bayside Canal Model**



**Figure 3.11: 24-hour 50-year recurrence interval rainfall depths adopted for the Bayside Canal Model**

In addition to the single-event modelling conducted, rainfall data recorded at 5-minute intervals over a 7 month winter period (May to November 2013) was also used to evaluate the various attenuation scenarios during a number of small to medium recorded storm events (data received from Mr B de Wet of the CCT).

The data from a number of rainfall stations was assessed in terms of its reliability by comparing the total rainfall depths over a number of storm periods and month periods. **Figure 3.12** shows the locations of the City of Cape Town and SA Weather Service rainfall stations in relation to the case study catchment areas.

The use of City of Cape Town's rainfall data was preferred since this was readily available and in the same format for various rainfall stations with depth readings taken at 5-minute intervals. For the Mosselbank River Model, there are no rainfall stations towards the north of the catchment, however, there was reliable data at the Kraaifontein Roads Depot and at the Tygerberg Reservoir. The data from the Maastricht Farm and Dagbreek Reservoir was incomplete and therefore could not be used. It is noted from **Figures 3.3** to **3.5** that the trend in terms of rainfall depths is relatively uniform along a north-south axis through the Mosselbank River Catchment, and the most variance occurs from east to west. Therefore, it can be argued that similar rainfall will generally occur to the north of the Kraaifontein Roads Depot and Tygerberg Reservoir rainfall stations which are located towards the south of the catchment.

The data from the Kraaifontein Roads Depot and Tygerberg Reservoir rainfall stations were also used for the Upper Kuils River Catchment. In order to distribute the rainfall for the Mosselbank and Upper Kuils River sub-catchments, a perpendicular bisector line was drawn between the two rainfall stations, with the sub-catchments to the north-east of the line being assigned the rainfall data from the Kraaifontein Roads Depot station, and the sub-catchments to the south-west of the line being assigned the rainfall data from the Tygerberg Reservoir.

For the Bayside Canal Model, the closest rainfall station was at the Potsdam Wastewater Treatment Works which provided reliable data for the modelling period.

The indicative 24-hour rainfall depths from the Kraaifontein Roads Depot, Tygerberg Reservoir, and Potsdam Wastewater Treatment Works are shown graphically in **Figures 3.13**, **3.14**, and **3.15**, respectively.

It is noted that the case study models were not calibrated for continuous modelling in terms of simulating sub-surface base-flow that continues to feed the watercourses during dry periods. The purpose of the continuous modelling of a 7-month period was rather to use actual rainfall events to evaluate various attenuation scenarios.

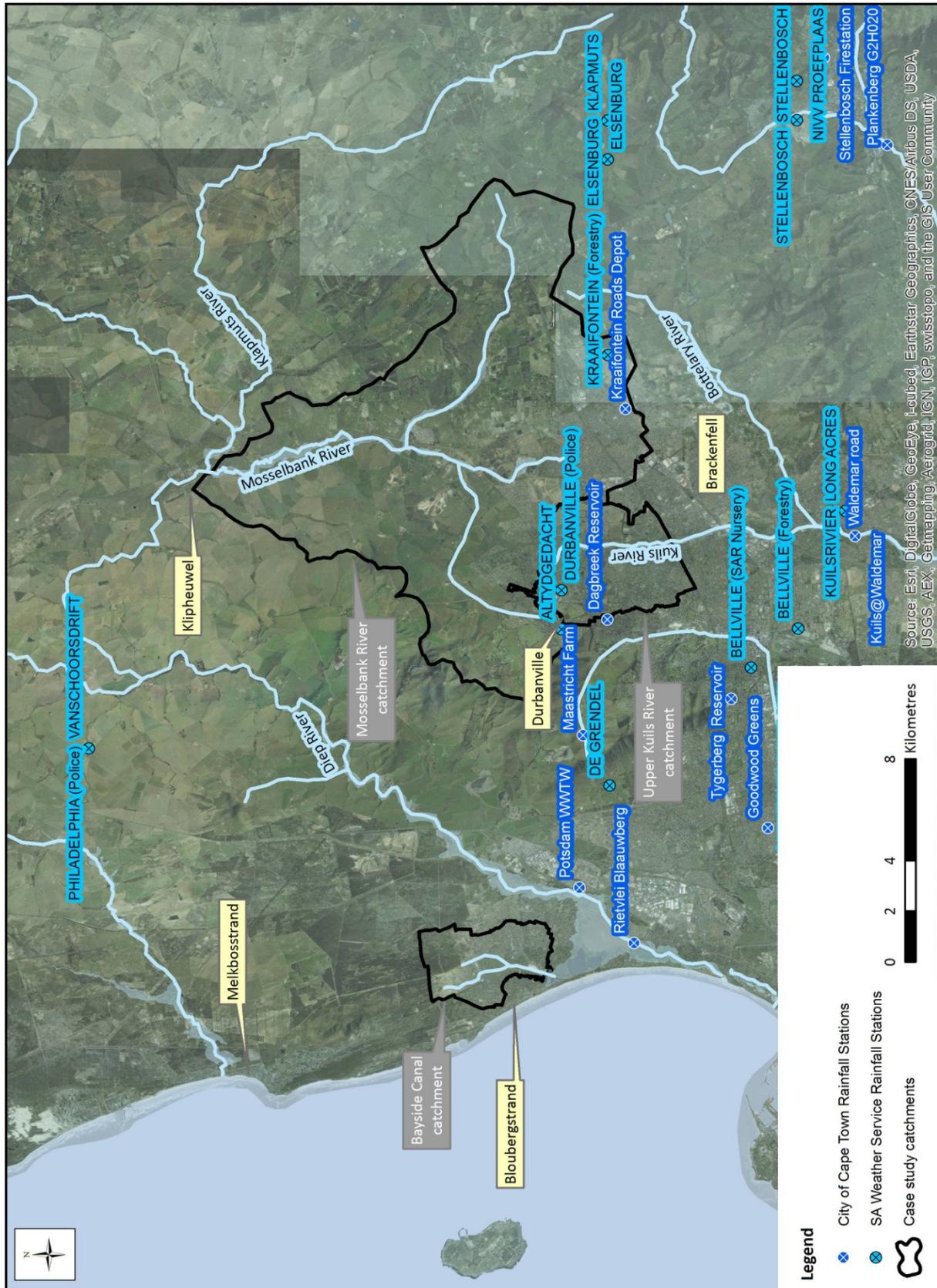


Figure 3.12: City of Cape Town and SA Weather Service Rainfall Stations

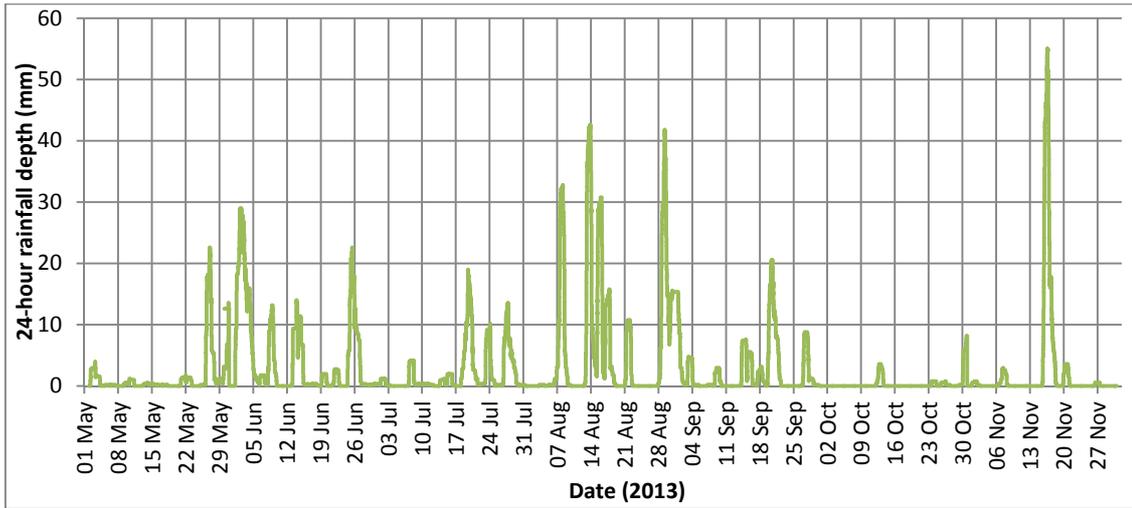


Figure 3.13: 24-hour rainfall depths at the Kraaifontein Roads Depot

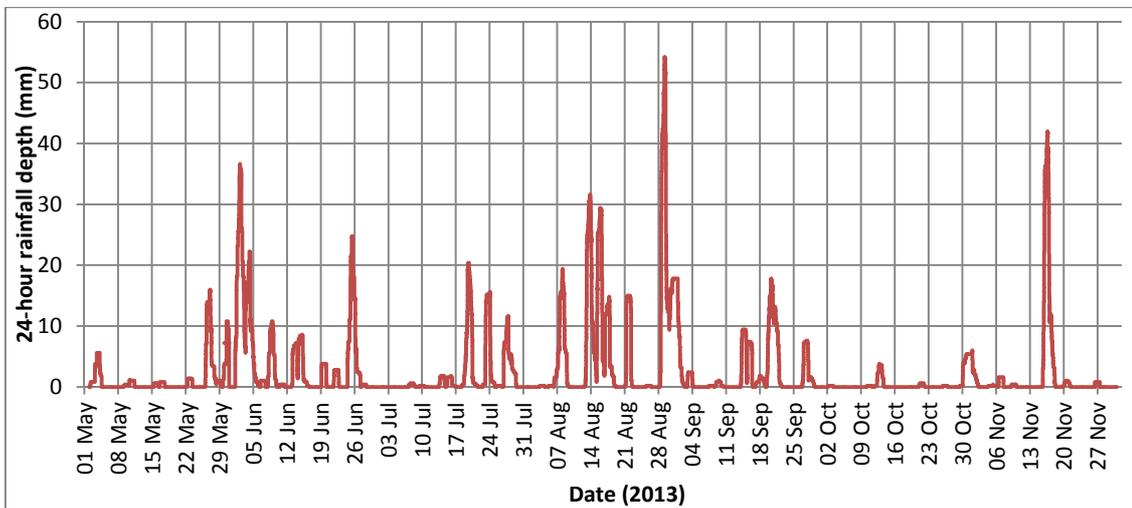


Figure 3.14: 24-hour rainfall depths at the Tygerberg Reservoir

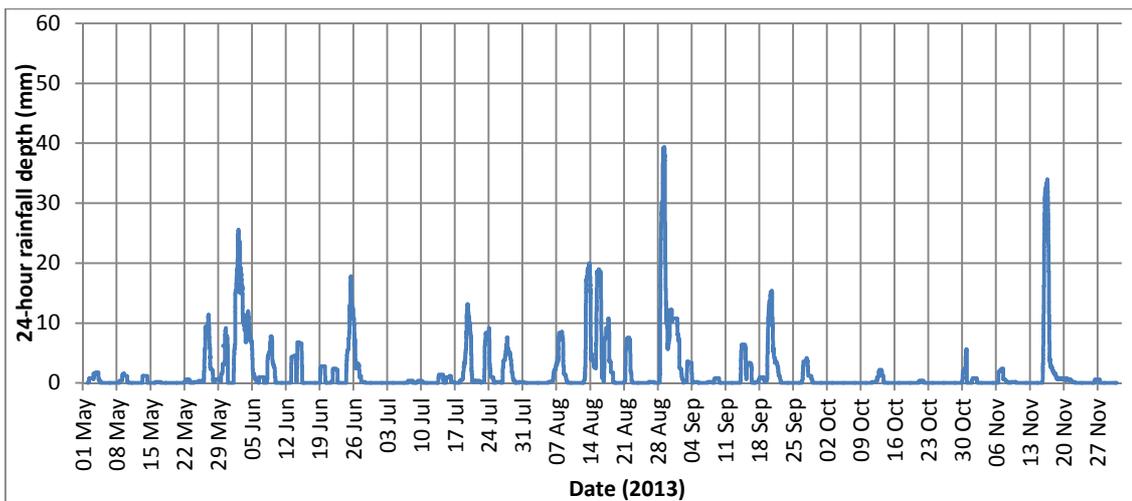


Figure 3.15: 24-hour rainfall depths at the Potsdam Wastewater Treatment Works

### 3.4.8 Watercourse geometry

Cross-sectional data for the existing watercourses was determined from available survey information received from the CCT's Geomatics Services Department. This included a photogrammetrically derived digital elevation model used in the case of the Mosselbank and Bayside Canal Models, and a LiDAR survey used in the case of the Upper Kuils River Model.

The roughness values assumed in the model were based on the recommended Manning "n" values sourced from SANRAL's *Drainage Manual* (SANRAL, 2013) and the *HEC-RAS River Analysis System Hydraulic Reference Manual* (USACE, 2010).

### 3.4.9 Attenuation routing

Post-development flow control was addressed in the models by routing catchment runoff into attenuation facilities in the form of storage nodes in PCSWMM. In the case of existing attenuation facilities, the control of the flow being released downstream was simulated through the use of outlet conduits, orifices or weirs according to the existing outlet conditions.

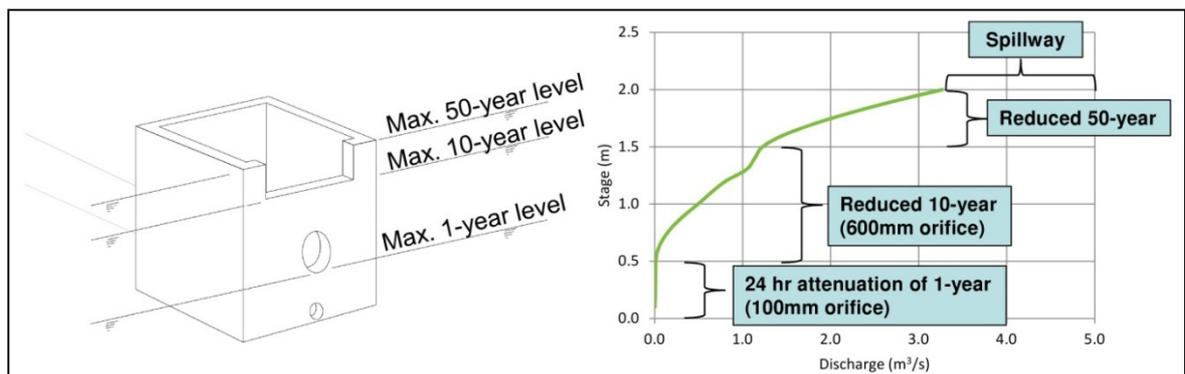
For sub-catchments where future development has been proposed, outlets were generally simulated with a rating curve describing flow for various storage depths. The storage volume was determined with an iterative process in order to achieve a given maximum depth during the design storm which in turn related to a predetermined outlet flow.

The above-mentioned process is relatively simple when designing for a single storm event where outflow at various depths was determined with the same procedure that one would use in designing a culvert structure assuming inlet control conditions. For attenuation facilities designed to control multiple storm events, the following procedure was undertaken for each facility to determine the attenuation storage volume and outlet control hydraulics:

- The maximum pond depth was chosen based on the estimated storage volume (typically 1.0 m, 1.5 m or 2 m).
- The shape of the storage facility was assumed to conform to the formula:  
$$\text{Water surface area} = \text{Coefficient} \times \text{Depth}^{0.5}$$
Where the Coefficient is varied according the required storage volume.
- Outflow from each facility controlled by a multi-stage outlet structure was assumed to follow the following design procedure:
  - Small orifice near pond invert level for control of the 1-year design storm.
  - Larger orifice at a level higher than the pond invert level for the control of the 10-year design storm.
  - Weir/grid inlet for the control of the 50-year design storm.

- An emergency spillway would accommodate flows resulting from storms exceeding the 100-year event.
- The size of each facility was determined through an iterative process by routing the post-development runoff hydrograph through the pond and determining the level to which ponding occurs, and then adjusting the size until the correct design level is reached for each design storm event.

**Figure 3.16** provides typical detail of an outlet structure and its particular outflow stage-discharge curve.

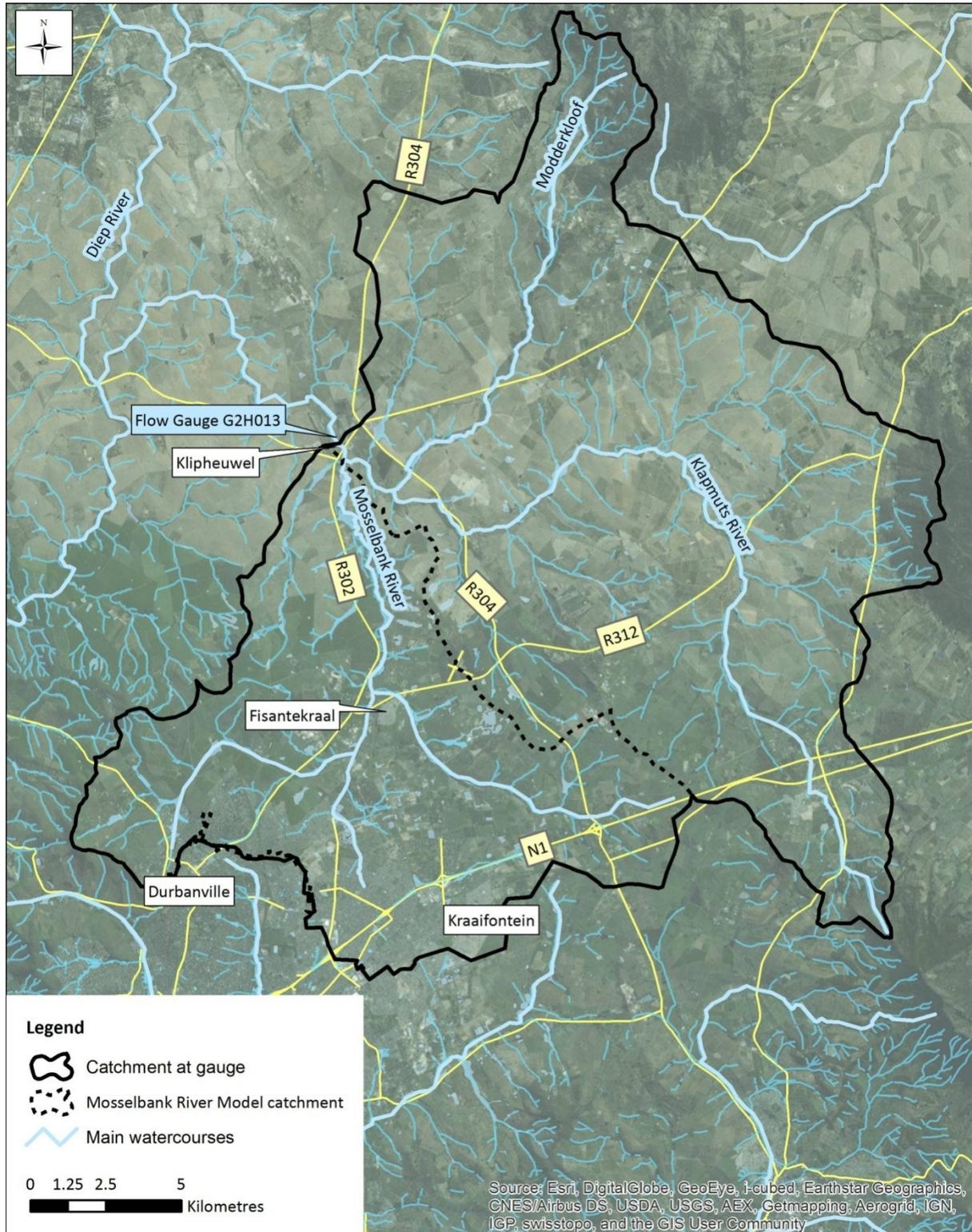


**Figure 3.16:** Typical multi-stage outlet structure used for modelling purposes

### 3.5 Mosselbank River Model verification

#### 3.5.1 Available logging data

As mentioned in **Section 3.4.1**, there was no available continuous stream flow data with which calibration could be undertaken. However, the annual maximum flow series for a 20 year period was available from the Department of Water Affairs for Flow Gauge G2H013 located downstream of the confluence of the Mosselbank and Klapmuts Rivers, as shown in **Figure 3.17**. Unfortunately, the rating curve for the gauge only goes to a maximum flow of  $23.08 \text{ m}^3/\text{s}$  at a level of 1.22 m and an assumption needed to be made regarding flows corresponding with levels higher than 1.22 m. With the lack of information regarding the geometric properties of the river cross-section during the time period over which the flow data was measured, the best assumption was to extrapolate the existing rating curve using a best-fit polynomial function. **Figure 3.18** shows the original rating curve and extrapolated rating curve which was used to produce a complete annual maximum time series for the gauge, as shown in **Table 3.5**.



**Figure 3.17: Location of Department of Water Affairs Flow Gauge G2H013**

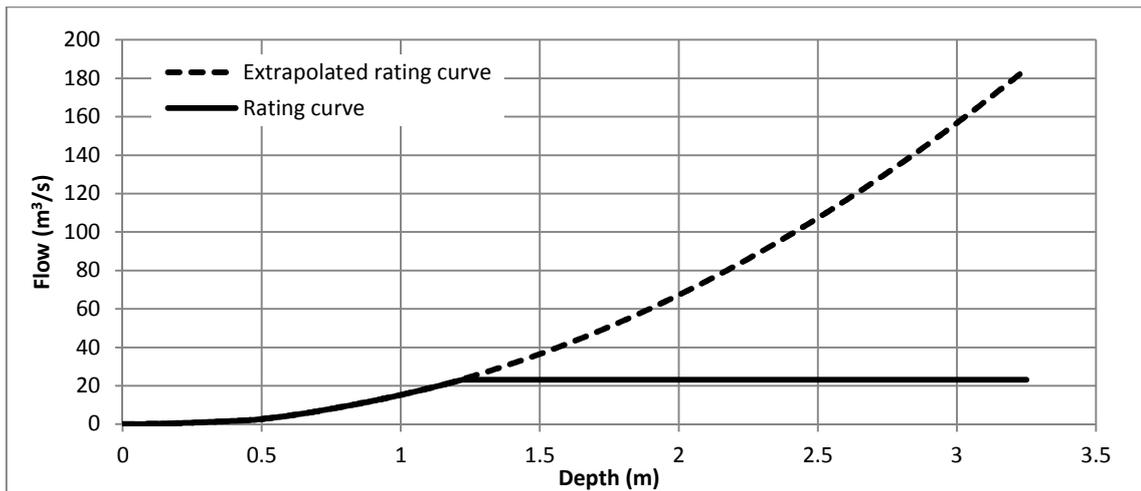


Figure 3.18: Flow Gauge G2H013 rating curve

Table 3.5: Annual maximum time series for Flow Gauge G2H013

Date	Level at peak (m)	Peak flow from DWA rating curve (m <sup>3</sup> /s)	Peak flow from extrapolated rating curve (m <sup>3</sup> /s)
1966-08-05	1.527	26.64	37.6
1967-07-03	0.838	10.69	10.7
1968-07-26	2.117	26.64	75.3
1969-09-10	0.424	1.91	1.9
1970-07-21	1.227	25.89	23.1
1971-07-16	0.463	2.22	2.2
1972-08-04	0.474	2.35	2.3
1973-08-10	0.299	1.07	1.1
1974-08-21	3.180	26.64	176.8
1975-07-20	2.290	23.08	89.4
1976-06-07	1.620	23.08	43.0
1977-06-26	2.000	23.08	67.3
1978-09-01	0.343	1.34	1.3
1979-08-08	0.367	1.50	1.5
1980-06-28	0.391	1.66	1.7
1981-08-11	1.204	22.47	22.5
1982-07-23	0.758	8.23	8.2
1983-06-27	2.108	23.08	74.5
1984-05-16	1.782	23.08	52.6
1985-07-06	3.245	23.08	183.8

Following from **Table 3.5**, a statistical analysis was performed and plotted using the Log Pearson Type 3 distribution which was found to best represent the data, as shown in **Figure 3.19**. The results of the analysis are shown in **Table 3.6** together with the results determined for only the Mosselbank River Catchment, upstream of its confluence with the Klapmuts River. The latter results were determined from the statistical data by reducing the figures obtained for the flow gauge using the so-called “Square-Area” Method, shown as follows. **Table 3.6** also shows the results obtained from the PCSWMM model set up for current catchment development characteristics using default modelling parameters.

$$Q_{Mosselbank} = Q_{G2H013} \sqrt{\frac{A_{Mosselbank}}{A_{G2H013}}}$$

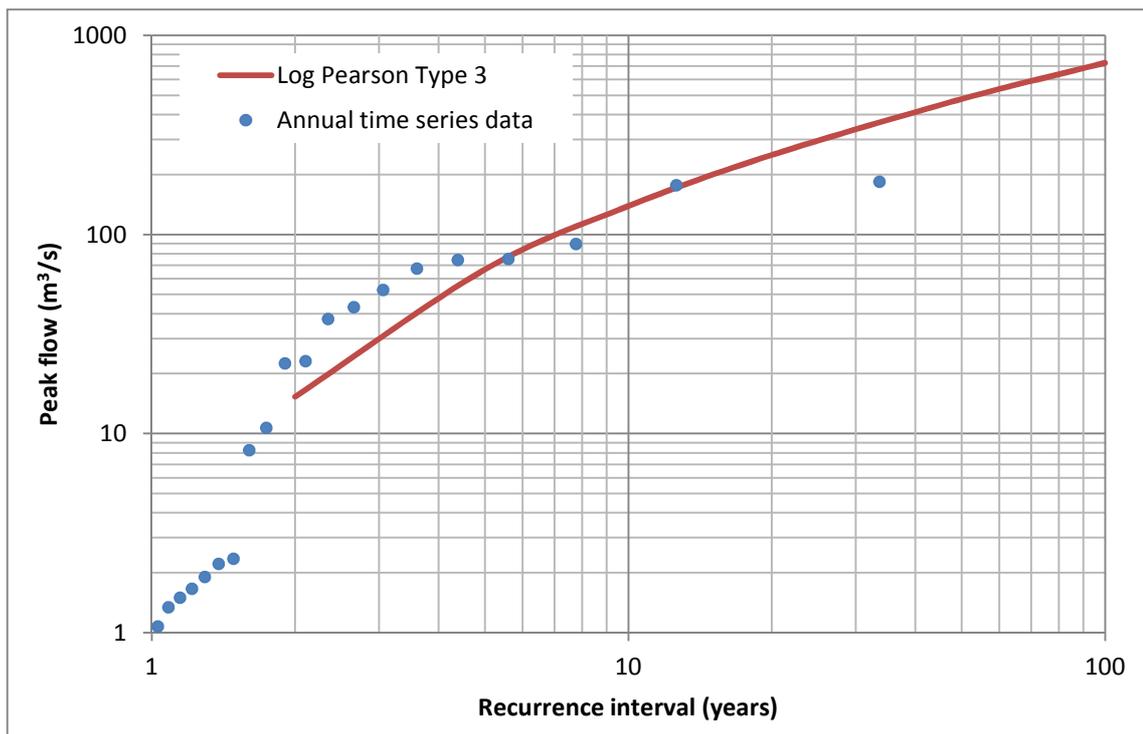
Where:

$Q_{Mosselbank}$  = Peak discharge from the Mosselbank River Catchment (m<sup>3</sup>/s)

$Q_{G2H013}$  = Peak discharge at Flow Gauge G2H013(m<sup>3</sup>/s)

$A_{Mosselbank}$  = Area of Mosselbank River Catchment (i. e. 164 km<sup>2</sup>)

$A_{G2H013}$  = Area of catchment at Flow gauge G2H013 (i. e. 473 km<sup>2</sup>)



**Figure 3.19: Statistical plot for Flow Gauge G2H013 using Log Pearson Type 3 distribution**

**Table 3.6: Comparison of results from statistical analysis and PCSWMM model**

Recurrence interval (years)	Peak flow at gauge (catchment area = 473 km <sup>2</sup> ) (m <sup>3</sup> /s)	Peak flow of Mosselbank River (catchment area = 164 km <sup>2</sup> ) (m <sup>3</sup> /s)	PCSWMM model results (catchment area 164 km <sup>2</sup> ) (m <sup>3</sup> /s)
2	15	9	14
5	67	39	-
10	139	82	98
20	251	147	-
50	478	281	210
100	728	428	-

From **Table 3.6**, it is noted that the PCSWMM modelling results using default modelling parameters are in the same range as the figures derived from the statistical analysis.

### 3.5.2 Peak flow verification using deterministic flood hydrology methods

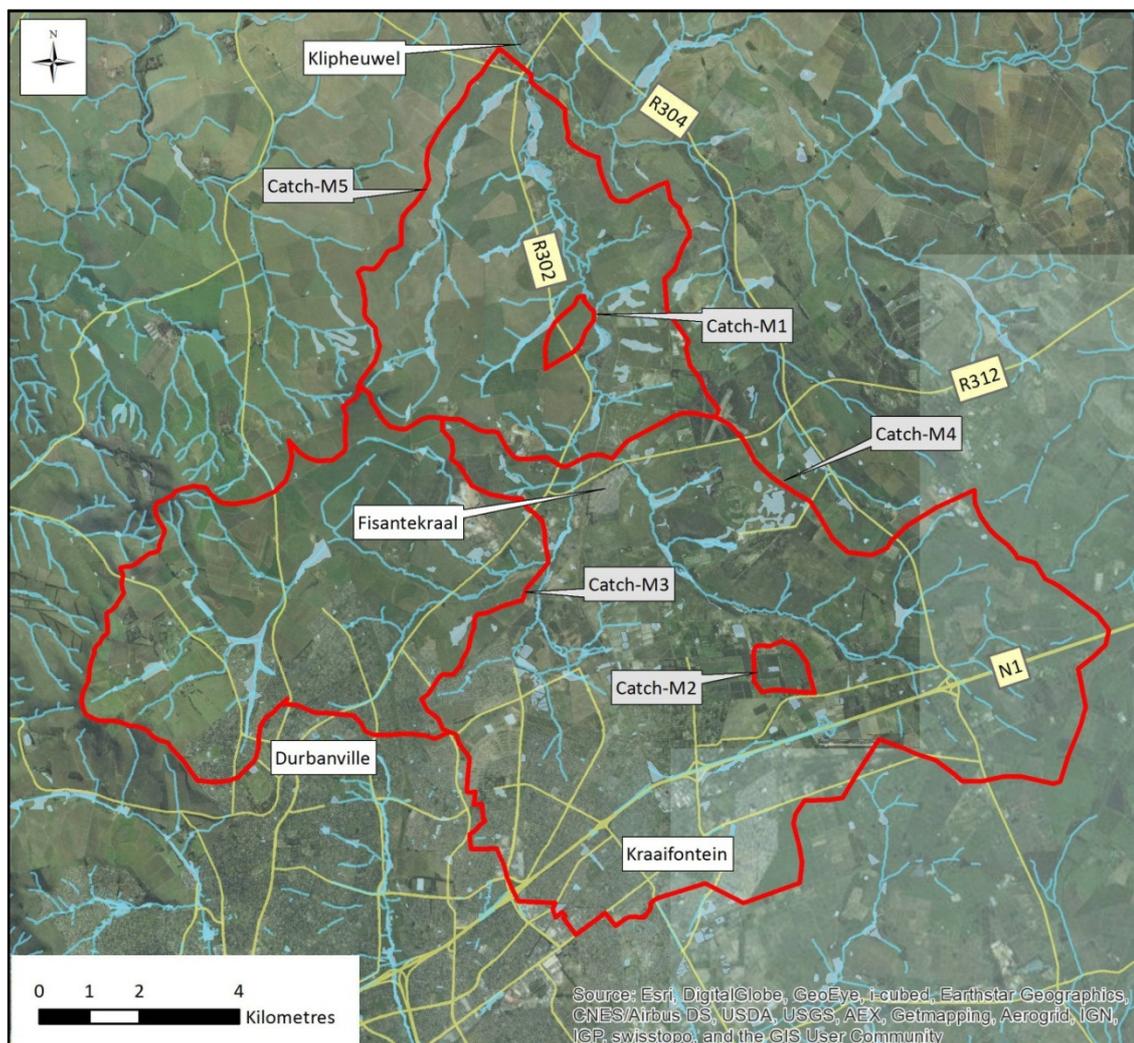
Since there was a lack of reliable stream flow records, it was deemed necessary to conduct a number of other checks to assess the results obtained from the PCSWMM model depicting the current catchment conditions (referred to as the pre-development model), and using default modelling parameters (refer to **Section 3.4**). Three of the most widely used deterministic flood hydrology methods, the Rational Method (Alternative 3 as described in the *Drainage Manual* (SANRAL, 2013)), the Unit Hydrograph Method and the SCS Method, were used at a number of locations within the Mosselbank River Catchment and compared to the results obtained using PCSWMM. The deterministic methods were based on procedures outlined in the *Drainage Manual* (SANRAL, 2013) and the Rational and SCS Methods made use of rainfall intensities derived from the short-term rainfall depths produced as part of the study entitled *Impacts of Projected Climate Change on Design Rainfall and Streamflows in the Cape Town Metro Area* (Schulze et al., 2010), previously discussed in **Section 3.4.7**. The SCS Method calculations were undertaken with the *Visual SCS-SA* software developed by the University of KwaZulu Natal and the Unit Hydrograph Method calculations were undertaken with the *Utility Programmes for Drainage* software developed by Sinotech cc.

In terms of applicability, the Rational Method was originally developed for catchment areas smaller than 15 km<sup>2</sup> due to the method's assumption that constant rainfall intensity occurs across the catchment for the duration of a storm event, which would not necessarily be true for larger catchment areas. However, with the use of short term rainfall data, and the application of an area reduction factor, the Rational Method formula has been successfully used by experienced flood hydrologists in South Africa for small and large catchments (van der Spuy and Rademeyer, 2014). Therefore, the Rational Method was used for all the catchments assessed below, from the 0.7 km<sup>2</sup> catchment to the 164 km<sup>2</sup> catchment.

The Unit Hydrograph Method is suitable for catchment sizes of between 15km<sup>2</sup> and 5000 km<sup>2</sup> (SANRAL, 2013), but should be used with caution for catchments at the lower end of the range. One of the key input parameters of the Unit Hydrograph Method is the “Veld Type” which is a regionalised runoff parameter, and therefore, if the physical characteristics of catchment being assessed are not particularly typical of the wider area within the Veld Type region, this may result in an overestimate or underestimate of the flood peaks.

The SCS Method was developed for small catchments, and is typically not recommended for catchments larger than 30 km<sup>2</sup> (Gericke and du Plessis, 2013).

The catchments assessed for verification purposes are shown in **Figure 3.20** and the main parameters used for the analyses are provided in **Table 3.7**. It is noted that the parameters used were, as far as possible, kept equivalent to those used in the PCSWMM model.



**Figure 3.20: Catchments used for verification purposes using deterministic flood hydrology methods**

**Table 3.7: Deterministic method input parameters**

Parameter	Catchment				
	Catch-M1	Catch-M2	Catch-M3	Catch-M4	Catch-M5
Catchment area (km <sup>2</sup> )	0.7	1.0	40.5	127.0	163.7
Longest watercourse (km)	1.5	1.5	11.4	13.4	26.7
Average watercourse slope (%)	3.72	1.10	1.09	0.93	0.35
Distance to centroid (km)	N/A	N/A	3.4	3.7	11.9
Average catchment slope (%)	4.6	1.9	7.4	4.3	4.4
Mean annual precipitation (mm)	498	542	565	546	539
Area reduction factor	1.00	1.00	0.97	0.91	0.94
Runoff coefficient (c-value)	0.25	0.22	0.31	0.31	0.30
SCS Curve number	81	81	N/A	N/A	N/A

Following from **Table 3.7**, a comparison of the results determined using the deterministic methods and the PCSWMM model is shown in **Table 3.8**, and further details of the deterministic method calculations are provided in **Appendix A**.

**Table 3.8: Results of flood hydrology using deterministic methods**

Location	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s)			
			Rational Method	SCS Method	Unit Hydrograph Method	PCSWMM Model
Catch-M1	0.7	2	0.7	0.7	-	0.4
		10	1.3	1.7	-	1.8
		50	2.2	2.9	-	3.9
Catch-M2	1.0	2	0.7	0.8	-	0.2
		10	1.4	1.8	-	1.3
		50	2.2	3.0	-	3.0
Catch-M3	40.5	2	19.9	-	41.2	12.7
		10	36.3	-	72.3	46.6
		50	59.0	-	117.4	99.0
Catch-M4	127.0	2	49.6	-	111.4	17.8
		10	90.3	-	194.6	95.1
		50	146.8	-	313.1	214.8
Catch-M5	163.7	2	34	-	82.1	13.6
		10	62	-	144.3	98.1
		50	100	-	234.4	209.5

From **Table 3.8**, it is noted that the results from the Mosselbank River PCSWMM model were generally found to produce comparatively higher peak flows for 50-year flood event compared to the Rational Method, and lower peak flows for the 2-year flood event. The Unit Hydrograph Method results were typically higher than the Rational Method and PCSWMM results, however, quite close to the PCSWMM 50-year results. The results for Catch-M1 and Catch-M2 show that the PCSWMM results are in a similar range to those determined with the Rational and SCS Method calculations.

### 3.5.3 Previous flood studies conducted for the Mosselbank River

The following previously conducted flood studies were reviewed in terms of their flood hydrology results, and compared to the result of the Mosselbank River PCSWMM model:

- *Mosselbank & Klapmuts Rivers: Determination of Floodlines* compiled by Africon Engineering International for the City of Cape Town in March 2001 (City of Cape Town, 2001). This study is hereafter referred to as the Africon 2001 Study.

- *Durbanville North Stormwater Masterplan and Floodline Determination* compiled by Bergstan Consulting and Development Engineers for the City of Cape Town in August 2009 (City of Cape Town, 2009c). This study is hereafter referred to as the Bergstan 2009 Study.

The Africon 2001 Study primarily made use of the Unit Hydrograph Method for determining flood peak flows for the purpose of floodline determination along the Mosselbank and Klapmuts River. The Bergstan 2009 Study focused on an upper tributary of the Mosselbank River (Catchment Catch-M3, as shown in **Figure 3.20**). This study used a PCSWMM model to determine flows at various locations within this catchment. The model was verified and adjusted based on the data obtained from a temporary stream flow logging device installed at the bottom of the above-mentioned catchment which measured maximum water levels during a number of relatively small storm events (< 1-year recurrence interval).

In **Table 3.9**, a summary of the results of the Africon 2001 and Bergstan 2009 studies is presented and compared to the results of the PCSWMM model set up for current catchment development characteristics using default modelling parameters.

**Table 3.9: Results of the Africon 2001 and Bergstan 2009 studies**

Location	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s)		
		Africon 2001	Bergstan 2009	PCSWMM Model
<b>Catch-M3: Upper Mosselbank River (catchment area = 40.5 km<sup>2</sup>)</b>	2	-	3	13
	5	49	4	-
	10	-	-	47
	20	79	15	-
	50	104	59	99
	100	130	77	-
<b>Catch-M5: Mosselbank River at Klipheuwel (catchment area = 163.7 km<sup>2</sup>)</b>	2	-	-	14
	5	107	-	-
	10	-	-	86
	20	171	-	-
	50	224	-	203
	100	176	-	-

Following from **Table 3.9**, **Figures 3.21** and **3.22** present a comparison of the flood hydrology results from the Africon 2001 Study, the Bergstan 2009 Study, and the results obtained with deterministic flood hydrology methods (refer to **Section 3.5.2**) and by means of statistical analysis (refer to **Section 3.5.1**).

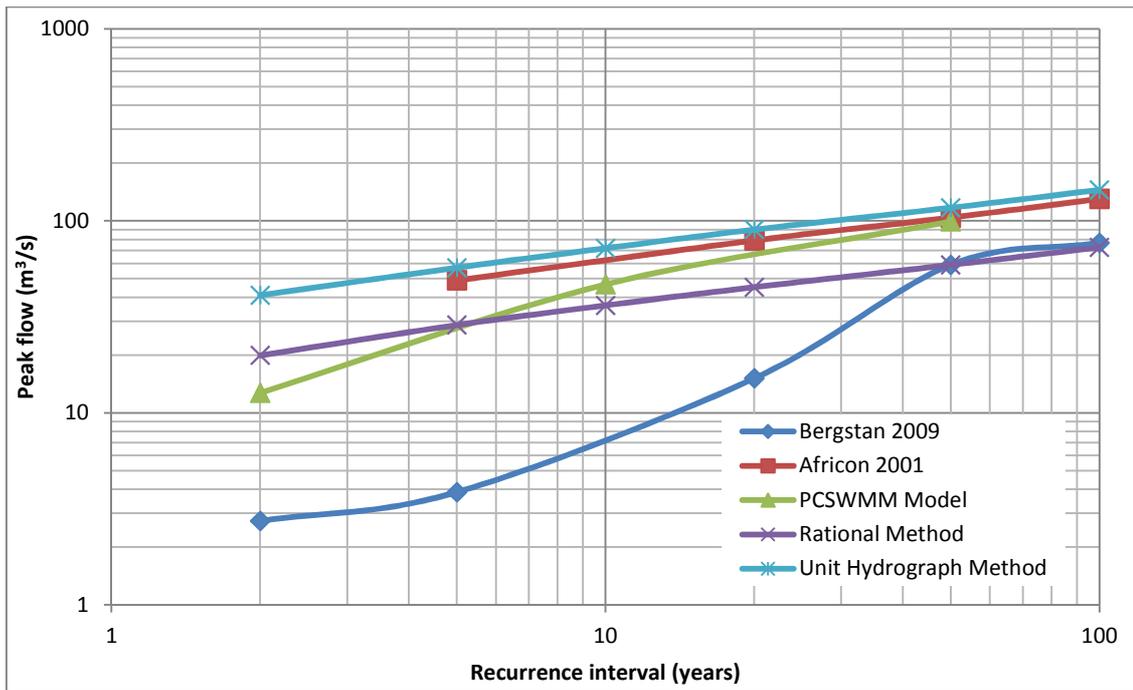


Figure 3.21: Log-log plot comparison of flood hydrology results for the Upper Mosselbank River (Catch-M3)

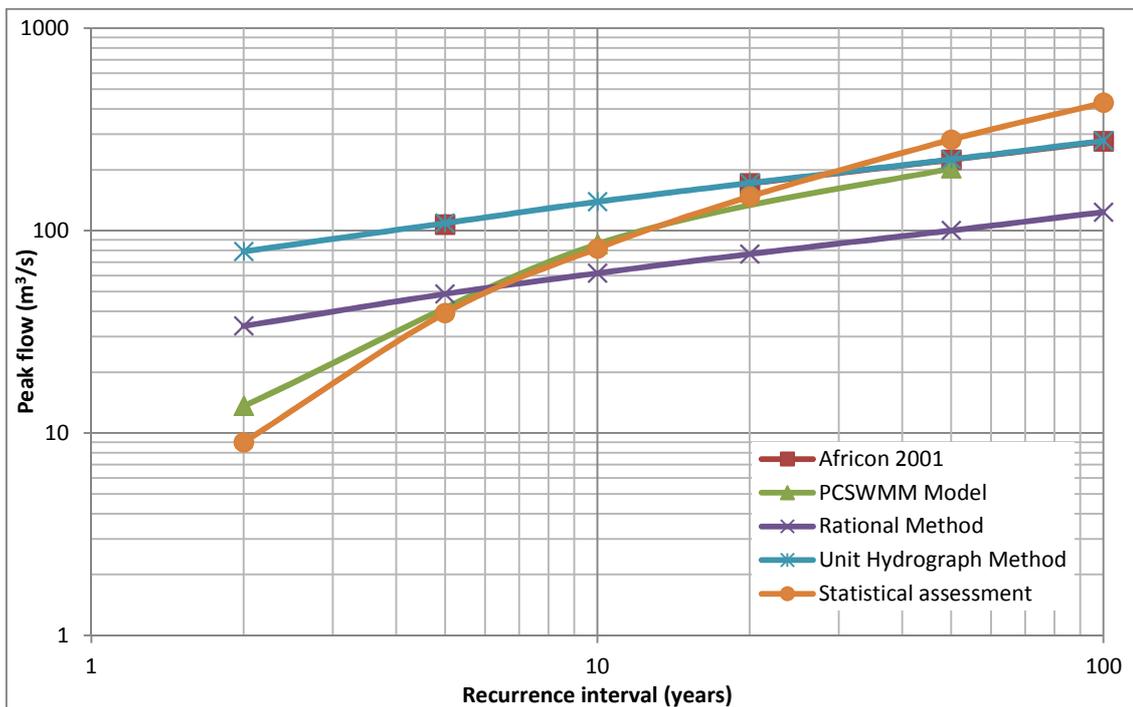


Figure 3.22: Log-log plot comparison of flood hydrology results for the Mosselbank River (Catch-M5)

From **Figures 3.21** and **3.22** it can be concluded that the results of the Mosselbank River PCSWMM model using default modelling parameters appear to be reasonable for the purpose of providing a comparative basis for assessment of attenuation practices within the catchment.

#### **3.5.4 Mosselbank River Model parameter sensitivity analysis**

In addition to the model verification procedure presented in **Section 3.5.3**, a selection of modelling parameters were varied and assessed in terms of their effect on peak flow results at various locations downstream:

- Watercourse Manning roughness coefficient – pre-development scenario
- Catchment width – pre-development scenario
- Infiltration rate – pre-development scenario
- Infiltration rate – post-development scenario
- % imperviousness – post-development scenario

The selected parameters listed above are ones which could be compared to available data, calculations or physical observations. Other parameters such as catchment roughness and depression storage, although affecting catchment runoff, were based on recommended values for typical catchments and could not be compared to available data, calculations or physical observations. The results of the sensitivity analysis are shown in **Tables 3.10** to **3.14**.

**Table 3.10: Test for sensitivity of Manning roughness coefficient**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default roughness	200% of default (all watercourses)	50% of default (all watercourses)
Catch-M3	40.5	2	12.7	12.7 (0.0)	12.6 (-0.8)
		10	46.6	46.7 (+0.2)	46.5 (-0.2)
		50	99.9	100.1 (+0.2)	99.4 (-0.5)
Catch-M4	127.0	2	20.2	20.2 (0.0)	20.2 (0.0)
		10	96.9	96.9 (0.0)	96.7 (-0.2)
		50	211.1	211.7 (+0.3)	210.5 (-0.3)
Catch-M5	163.7	2	13.6	13.7 (+0.7)	13.6 (0.0)
		10	98.1	98.3 (+0.2)	98.0 (-0.1)
		50	209.5	209.7 (+0.1)	209.2 (-0.1)

**Table 3.11: Test for sensitivity of catchment width**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default catchment width	20% increase (all catchments)	20% decrease (all catchments)
Catch-M3	40.5	2	12.7	13.6 (+7.1)	11.7 (-7.9)
		10	46.6	50.0 (+7.3)	43.5 (-6.7)
		50	99.9	104.1 (+4.2)	93.5 (-6.4)
Catch-M4	127.0	2	20.2	21.8 (+7.9)	18.6 (-7.9)
		10	96.9	104.6 (+7.9)	89.7 (-7.4)
		50	211.1	224.0 (+6.1)	196.8 (-6.8)
Catch-M5	163.7	2	13.6	14.6 (+7.4)	12.7 (-6.6)
		10	98.1	103.7 (+5.7)	92.1 (-6.1)
		50	209.5	221.0 (+5.5)	196.3 (-6.3)

**Table 3.12: Test for sensitivity of infiltration rate – pre-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default infiltration rates	50% increase (all catchments)	50% decrease (all catchments)
Catch-M3	40.5	2	12.7	7.0 (-44.9)	23.8 (+87.4)
		10	46.6	12.5 (-73.2)	70.0 (+50.2)
		50	99.9	77.4 (-22.5)	120.0 (+20.1)
Catch-M4	127.0	2	20.2	9.6 (-52.5)	47.5 (+135.1)
		10	96.9	65.8 (-32.1)	154.0 (+58.9)
		50	211.1	157.8 (-25.2)	286.4 (+35.7)
Catch-M5	163.7	2	13.6	6.1 (-55.1)	44.4 (+226.5)
		10	98.1	55.0 (-43.9)	153.9 (+56.9)
		50	209.5	152.6 (-27.2)	309.9 (+47.9)

**Table 3.13: Test for sensitivity of infiltration rate – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default infiltration rates	50% increase (all catchments)	50% decrease (all catchments)
Catch-M3	40.5	2	18.3	17.8 (-2.7)	27.7 (+51.4)
		10	54.9	43.7 (-20.4)	76.4 (+39.2)
		50	109.0	91.7 (-15.9)	124.7 (+14.4)
Catch-M4	127.0	2	47.2	38.5 (-18.4)	67.5 (+43.0)
		10	135.6	112.3 (-17.2)	188.2 (+38.8)
		50	269.9	224.0 (-17.0)	329.7 (+22.2)
Catch-M5	163.7	2	38.0	30.4 (-20.0)	64.6 (+70.0)
		10	131.1	100.2 (-23.6)	184.4 (+40.7)
		50	273.2	217.6 (-20.4)	341.2 (+24.9)

**Table 3.14: Test for sensitivity of % imperviousness – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default % imperviousness	20% increase (all catchments)	20% decrease (all catchments)
Catch-M3	40.5	2	18.3	20.3 (+10.9)	16.7 (-8.7)
		10	54.9	58.2 (+6.0)	52.1 (-5.1)
		50	109.0	111.6 (+2.4)	106.4 (-2.4)
Catch-M4	127.0	2	47.2	55.9 (+18.4)	39.9 (-15.5)
		10	135.6	149.9 (+10.5)	127.3 (-6.1)
		50	269.9	282.5 (+4.7)	257.3 (-4.7)
Catch-M5	163.7	2	38.0	46.1 (+21.3)	32.7 (-13.9)
		10	131.1	146.9 (+12.1)	116.6 (-11.1)
		50	273.2	290.5 (+6.3)	255.6 (-6.4)

Following from **Tables 3.10 to 3.14**, the following is noted:

- Changes to the watercourse roughness parameters result in negligible changes to peak flow rates downstream. A higher roughness resulted in a minor increase in peak flows, while a lower roughness resulted in a minor decrease in peak flows for the three catchments evaluated. It is noted that further upstream in the catchments, changes to watercourse roughness had the opposite effect on peak flows. This apparent contradiction in results can be attributed to catchment-specific routing dynamics which cause a negligible increase in peak flows when runoff from the various tributaries is lagged slightly.
- A 20% change in the sub-catchment widths of all the model sub-catchments results in a change in the peak flow of catchments Catch-M3, Catch-M4 and Catch-M5 of less than 8%. Therefore, this parameter is not considered very sensitive.
- A 50% change in the infiltration rate parameters used in the pre-development model results in significant changes to the flow rates in the Mosselbank River, especially for the 2-year recurrence interval event. It is therefore very important that the most accurate soil characteristics are determined and used in the hydrological component of PCSWMM, especially for undeveloped catchments.

- The post-development model results are less sensitive to infiltration rate changes since a large portion of the flow is now running off hardened surfaces instead of the natural ground.

Another point to note is that the extent of impervious catchment surface area has quite a large impact on runoff. A reduction of hardened surfaces, and hence, post-development runoff, can be achieved through innovative spatial planning and urban design. This is addressed further as one of the primary modelling scenarios (Scenario 5) evaluated as part of this study, described in Section 3.8.

Based on the results of the Mosselbank River Model verification procedure and sensitivity analysis, it was not deemed necessary to adjust any of the modelling parameters since these are based on the best available data and appear to provide reasonable results which can be used for the assessment of attenuation practices within the catchment. The default modelling parameters, as discussed in **Section 3.4**, were therefore left unchanged.

### 3.6 Bayside Canal Model verification

#### 3.6.1 Peak flow verification using deterministic flood hydrology methods

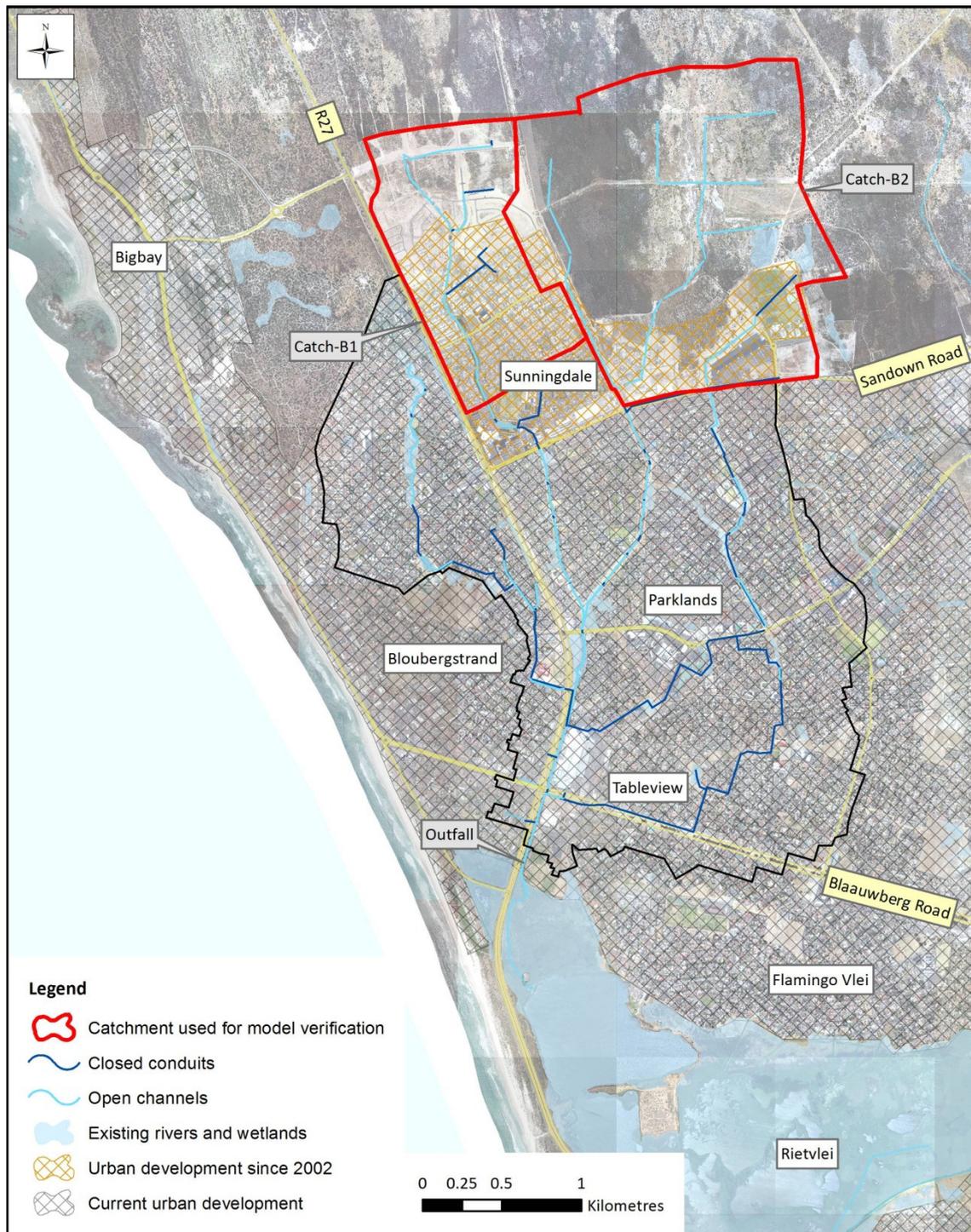
In order to verify the results of the Bayside Canal Model, a number of deterministic runoff calculations were conducted following the same procedure described in **Section 3.5.2** for the Mosselbank River Model.

The catchments assessed for verification purposes are shown in **Figure 3.23** and the main parameters used for the analyses are provided in **Table 3.15**. It is noted that the parameters used were, as far as possible, kept equivalent to those used in the PCSWMM model.

**Table 3.15: Deterministic method input parameters**

Parameter	Catchment number	
	Catch-B1	Catch-B2
Catchment area (km <sup>2</sup> )	1.2	3.3
Longest watercourse (km)	1.7	2.7
Average watercourse slope (%)	0.5	0.4
Average catchment slope (%)	1.0	1.0
Mean annual precipitation (mm)	399	420
Area reduction factor	1.00	1.00
Runoff coefficient (c-value)	0.10	0.10
SCS Curve number	49	49

Following from **Table 3.15**, a comparison of the results determined using the deterministic methods and the PCSWMM model is shown in **Table 3.16**, and further details of the deterministic method calculations are provided in **Appendix A**.



**Figure 3.23: Catchments used for verification purposes using deterministic flood hydrology methods**

**Table 3.16: Results of flood hydrology using deterministic methods**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s)		
			Rational Method	SCS Method	PCSWMM Model
Catch-B1	1.2	2	0.26	0.01	0.00
		10	0.50	0.09	0.08
		50	0.95	0.28	0.48
Catch-B2	3.3	2	0.54	0.03	0.00
		10	1.03	0.24	0.13
		50	1.97	0.70	1.61

From **Table 3.16**, it is noted that the PCSWMM model results are generally lower than the Rational Method and SCS Method results for the 2- and 10-year recurrence interval flood events. This is to be expected since the model takes into account the high infiltration rate associated with the undeveloped catchment which consists of undulating dune systems. As described in **Section 3.2.2**, this area does not have any well-defined watercourses, and it is only during large flood events that significant runoff is to be expected. For the 50-year recurrence interval event, the SCS Method results are lower than both the Rational Method and PCSWMM model results.

### 3.6.2 Previous flood studies conducted for the Bayside Canal

The following previously conducted stormwater master planning study was reviewed in terms of the flood hydrology results presented therein, and compared to the result of the Bayside Canal PCSWMM model:

- *Stormwater Masterplan for Blaauwberg Development Area* compiled by CIVtech Consultants for the City of Cape Town in September 2006 (City of Cape Town, 2006). This study is hereafter referred to as the CIVtech 2006 Study.

The above-mentioned study assessed the bulk stormwater requirements for the full development of the catchment draining to the Bayside Canal, and made recommendations regarding the necessary attenuation to reduce post-development runoff to a maximum permissible flow during the 50-year flood event. The maximum permissible flow was not directly related to pre-development flows, but rather to the limited capacity of the canal system downstream.

The following maximum permissible flows were prescribed in the CIVtech 2006 Study for Catchments Catch-B1 and Catch-B2, respectively:

- Catch-B1: 0.85 m<sup>3</sup>/s
- Catch-B2: 2.80 m<sup>3</sup>/s

The post-development modelling scenarios with attenuation control, discussed further in **Section 3.8**, also took these flows into account.

### 3.6.3 Bayside Canal Model parameter sensitivity analysis

In addition to the model verification procedure presented in **Section 3.6.2**, the following modelling parameters were varied and assessed in terms of their effect on peak flow results at various locations downstream:

- Watercourse Manning roughness coefficient – post-development scenario
- Catchment width – post -development scenario
- Infiltration rate – pre-development scenario
- Infiltration rate – post-development scenario
- % imperviousness – post-development scenario

It is noted that generally the post-development scenario was used for the testing of parameter sensitivity as the pre-development scenario flows were considered too low for adequate comparison. The results of the sensitivity analysis are shown in **Tables 3.17 to 3.21**.

**Table 3.17: Test for sensitivity of Manning roughness – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default roughness	200% of default (all watercourses)	50% of default (all watercourses)
Catch-B1	1.2	2	0.76	0.51 (-32.9)	0.84 (+10.5)
		10	1.66	1.03 (-38.0)	1.85 (+11.4)
		50	3.48	1.74 (-50.0)	3.77 (+8.3)
Catch-B2	3.3	2	2.49	1.70 (-1.7)	2.79 (+12.0)
		10	4.90	2.89 (-41.0)	5.34 (+9.0)
		50	6.31	3.35 (-46.9)	7.09 (+12.4)

**Table 3.18: Test for sensitivity of catchment width – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default catchment width	20% increase (all catchments)	20% decrease (all catchments)
Catch-B1	1.2	2	0.76	0.79 (+3.9)	0.73 (-3.9)
		10	1.66	1.72 (+3.6)	1.56 (-6.0)
		50	3.48	3.69 (+6.0)	3.38 (-2.9)
Catch-B2	3.3	2	2.49	2.56 (+2.8)	2.42 (-2.8)
		10	4.90	5.05 (+3.1)	4.70 (-4.1)
		50	6.31	6.38 (+1.1)	6.16 (-2.4)

**Table 3.19: Test for sensitivity of infiltration rate – pre-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default catchment width	50% increase (all catchments)	50% decrease (all catchments)
Catch-B1	1.2	2	0.00	0.00 (-)	0.01 (+)
		10	0.08	0.00 (-)	0.41 (+412.5)
		50	0.48	0.18 (-62.5)	2.27 (+372.9)
Catch-B2	3.3	2	0.00	0.00 (-)	0.01 (+)
		10	0.13	0.00 (-)	1.33 (+923.1)
		50	1.61	0.48 (-70.2)	3.86 (+139.8)

**Table 3.20: Test for sensitivity of infiltration rate – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default catchment width	50% increase (all catchments)	50% decrease (all catchments)
Catch-B1	1.2	2	0.76	0.68 (-10.5)	0.82 (+7.9)
		10	1.66	1.59 (-4.2)	1.86 (+12.0)
		50	3.48	3.41 (-2.0)	3.75 (+7.8)
Catch-B2	3.3	2	2.49	2.46 (-1.2)	2.73 (+9.6)
		10	4.90	4.68 (-4.5)	5.33 (+8.8)
		50	6.31	6.13 (-2.9)	6.80 (+7.8)

**Table 3.21: Test for sensitivity of percent imperviousness – post-development scenario**

Catchment number	Catchment area (km <sup>2</sup> )	Recurrence interval (years)	Flood peak (m <sup>3</sup> /s) (% change)		
			Default % imperviousness	20% increase (all catchments)	20% decrease (all catchments)
Catch-B1	1.2	2	0.76	0.92 (+21.1)	0.63 (-17.1)
		10	1.66	1.94 (+16.9)	1.24 (-25.3)
		50	3.48	5.40 (+55.2)	2.47 (-29.0)
Catch-B2	3.3	2	2.49	3.12 (+25.3)	2.07 (-16.9)
		10	4.90	5.47 (+11.6)	4.20 (-14.3)
		50	6.31	6.67 (+5.7)	5.88 (-6.8)

Following from **Tables 3.17 to 3.21**, it is noted that the sensitivity analysis undertaken for the Bayside Canal Model resulted in similar findings reported on for the Mosselbank River Model in **Section 3.5.4**. The parameter which has the greatest impact on runoff when adjusted is the sub-catchment infiltration rate. However, this parameter becomes less sensitive in post-development modelling scenarios.

Based on the results of the Bayside Canal Model verification procedure and sensitivity analysis, it was not deemed necessary to adjust any of the modelling parameters since these are based on the best available data and appear to provide reasonable results which can be used for the assessment of attenuation practices within the catchment. The default modelling parameters, as discussed in **Section 3.4**, were therefore left unchanged.

### 3.7 Upper Kuils River Model verification

As mentioned in **Section 3.2.3**, the Upper Kuils River Model was not originally compiled by the author but was merely used to provide insight into the effect of existing attenuation facilities in a catchment which is almost fully developed. The Upper Kuils River Model was initially compiled by AECOM (then BKS (Pty) Ltd) as part of the study entitled *Eastern Catchments High Level Masterplan* commissioned by the CCT (City of Cape Town, 2013). Part of this study involved the calibration and verification of the model. This included the following procedures:

- Calibration was undertaken using rainfall and stream flow data for a single storm event which occurred on 20 October 2004. This was the only storm event which appeared to provide reliable stream flow and rainfall data at the Waldemar Road gauge in the suburb of Kuils River. Based on this storm event, the model produced peak flow results in excess of the recorded data, but the shape of the hydrograph was reasonably similar.
- The model's results were also compared with previous flood studies and with the results of deterministic flood hydrology methods which typically provided peak flows comparatively higher than the model.

Based on the above calibration and verification undertaken, the Kuils River Model was adjusted where necessary, and the model was accepted to provide reasonably accurate results. Therefore, for the purpose of this study, the modelling parameters were kept unchanged.

### 3.8 Attenuation scenarios

From the results of the model verification assessments described in **Sections 3.5, 3.6 and 3.7**, for the three case studies, it was concluded that the models are providing realistic results which are suitable for the purpose of comparing a number of hypothetical attenuation scenarios, and therefore no further adjustments were made to the default input parameters of the PCSWMM models as discussed in **Section 3.4**. The following attenuation scenarios were specifically modelled and compared as part of this study:

- Scenario 1: Pre-development  
This scenario models the catchment in natural or rural conditions prior to development.
- Scenario 2: Post-development  
The post-development scenario takes into account the significant increase in the areas now covered with impervious surfaces, such as roads, paving, roofs etc. It also takes into

account the conveyance of flow via concrete pipes and channels with no flood attenuation facilities implemented in the drainage system.

- Scenario 3: Post-development – Attenuation with single culvert type outlet  
This scenario shows the effect of attenuation implemented at sub-catchment level designed to attenuate the post-development peak flow resulting from the 50-year design storm with a single culvert type outlet structure. This type of attenuation is common in South African cities in the form of dry detention ponds. The volume of the pond is determined by calculating the difference between the flow volume coming into the pond over time, and the outlet flow volume over time.
  
- Scenario 4: Post-development – Attenuation with multi-stage outlet structure  
This scenario simulates attenuation implemented at sub-catchment level designed to attenuate multiple storm events (1-year, 10-year and 50-year) as per the CCT's *Management of Urban Stormwater Impacts Policy* (City of Cape Town, 2009b) (refer to **Section 2.6.3**).
  
- Scenario 5: Post-development – Attenuation and stormwater BMPs  
This scenario simulates attenuation implemented at sub-catchment level designed to attenuate multiple storm events, as in Scenario 4, however it assumes that stormwater BMPs and SuDS controls are more widely used within the sub-catchment drainage network. These BMPs include the following:
  - Green roofs
  - Permeable paving
  - Better site design, e.g. reduced impervious roadway widths
  - Clustering of buildings, leaving more open spaces
  - Grass channels and swales
  - Bioretention areas
  - Infiltration devices (e.g. highly permeable trenches)
  - Soakaways
  - Small ponds and wetlands
  - Prevention of runoff from hard surfaces entering directly into stormwater system

It would be impractical and beyond the scope of this study to assess the effect of all of the above BMPs individually, however, one can make assumptions as to the effect that these BMPs would have on the percent imperviousness of the sub-catchments, as well as the portion of runoff that would be routed from impervious areas to pervious areas, instead of directly to the stormwater system, as shown in **Table 3.22** (based on similar factors used in

the study *Investigation into the medium to long term growth options for Cape Town* (City of Cape Town, 2012a)).

**Table 3.22: Impact of BMPs on stormwater runoff parameters**

Best Management Practice	Land use				
	High density residential	Medium density residential	Low density residential	Mixed use intensity	Industrial
<b>Percentage reduction in impervious area</b>					
Green roofs	-5%	0%	0%	-10%	-5%
Permeable paving	-5%	-5%	-5%	-5%	-10%
Better site design, e.g. reduced impervious roadway widths	0%	-5%	-5%	0%	0%
Clustering of buildings and leaving more extensive open spaces	-5%	-5%	0%	0%	0%
Bioretention areas	-1%	-1%	-1%	-1%	-1%
Infiltration devices	-1%	-1%	-1%	-1%	-1%
<b>Percentage increase in runoff routed from impervious areas to pervious areas</b>					
Prevent runoff from hard surfaces directly entering stormwater system	10%	20%	30%	10%	10%

**Table 3.23** provides a comparison of the runoff parameters used for Scenarios 2, 3 and 4 in comparison to Scenario 5.

**Table 3.23: Comparison of stormwater runoff parameters for various scenarios**

Runoff parameter	Scenario	Land use				
		High density residential	Medium density residential	Low density residential	Mixed use intensity	Industrial
Imperviousness (%)	2, 3, 4	75%	65%	40%	85%	95%
	5	58%	48%	28%	68%	78%
Runoff from impervious areas routed to pervious areas (%)	2, 3, 4	30%	35%	45%	20%	10%
	5	40%	55%	75%	30%	20%

Besides the pre-development scenario (Scenario 1), all of the other scenarios assumed full development of the modelled catchments. For comparative purposes, the catchment and watercourse modelling parameters of all of the post-development scenarios (i.e. Scenarios 2 to 5) were kept consistent for the respective simulations and only the attenuation control

parameters were varied. One exception to this statement is that the percent imperviousness and routing to pervious areas was adjusted for Scenario 5, as described above.

The following modelling approaches specific to each case study are noted:

- In the case of the Mosselbank River Model, for the purpose of this study, most of the existing catchment development in the Kraaifontein area was not modelled in detail but rather compiled so that sub-catchment outflows corresponded with the results of the previous stormwater master planning studies undertaken by Ninham Shand Consulting Services for the City of Cape Town in 2002 (City of Cape Town, 2002b). These catchments were left unchanged for the various modelling scenarios.
- In the case of the Bayside Canal Model, all development which has been constructed since 2002, that is all development north of Sandown Road (refer to **Figure 3.23**), was ignored in the pre-development scenario (Scenario 1). For the post-development scenarios, the sub-catchment parameters took into account the existing development in this area, as well as the proposed future development in the remaining areas of the catchment, however, the existing attenuation facilities north of Sandown Road were modelled in accordance with the various above-mentioned scenarios, and not as they exist currently.

As mentioned in **Section 3.2.2**, the natural pre-development conditions of the Bayside Canal Catchment result in very little overland runoff and therefore it is often impossible or impractical to attenuate flow to accurately mimic pre-development conditions. Therefore, the attenuation facilities were designed to attenuate flow so that the 50-year post-development flow was attenuated to a level typically less than 10 l/s per hectare. In the case of Scenario 4, the 10-year post-development runoff was typically reduced to less than 5 l/s per hectare and the 2-year post-development runoff was typically attenuated over a 24-hour period.

- The existing stormwater system within the Upper Kuils River Catchment consists of a complex network of infrastructure which has been implemented with a mix of new and old design standards. It was therefore not practical as part of this study to assess the effect of various attenuation practices implemented retrospectively in a catchment which is already almost completely developed. Instead, this model was used to evaluate the performance of the existing system in terms of attenuation, and comment on which areas of the catchment perform better when compared to pre-development hydrology, and how this relates to some of the observations made from the Bayside Canal and Mosselbank River models. Therefore, the scenarios modelled for this case study only included Scenario 1,

Scenario 2, an “existing attenuation” scenario, and Scenario 5 which assessed the option of implementing retrofit BMPs within the existing catchment areas.

The results of the modelling simulations are provided in **Sections 3.9, 3.10 and 3.11** for the three case studies and a discussion of the results is presented in **Chapter 4**.

### 3.9 Mosselbank River Model results

#### 3.9.1 Attenuation results at sub-catchment level

For Scenarios 3, 4 and 5, each sub-catchment within the Mosselbank River Model was drained to a hypothetical attenuation facility, modelled according to the assumptions described in **Section 3.8**. As mentioned in **Section 3.4.9**, the design of each of these attenuation facilities was undertaken by means of an iterative process which sized the outlet structures and storage volume based on the on the pre-development and post-development runoff peaks resulting from each individual sub-catchment. As part of this process, each sub-catchment was assessed in terms of its performance in comparison to pre-development runoff. **Table 3.24** provides the average figures for the percent difference of the maximum attenuated outlet flows from the sub-catchments compared to pre-development runoff peaks.

**Table 3.24: Comparison of sub-catchment post-development and pre-development peak runoff**

Recurrence interval (years)	Average % difference of maximum post-development outlet discharge from individual sub-catchments in comparison to pre-development peak runoff			
	Scenario 2	Scenario 3	Scenario 4	Scenario 5
2	497%	213%	35%	-8%
10	195%	28%	-20%	-34%
50	138%	-22%	-26%	-35%

From **Table 3.24**, the following is noted:

- The implementation of attenuation facilities with single-stage outlet structures (Scenario 3) only tends to reduce the 50-year post-development runoff peak to less than the equivalent pre-development runoff peak at each sub-catchment (i.e. on average, the outlet flow from each attenuation facility is 22% lower than the pre-development runoff peak from the same catchment). The post-development peak discharge from the Scenario 3 attenuation facilities is on average 28% and 213% higher than the equivalent pre-development runoff peak for the 10-year and 2-year recurrence interval storms, respectively.
- The multi-stage outlet structures simulated in Scenario 4 effectively reduce the 50-year and 10-year post-development runoff peaks to lower than the pre-development runoff peaks, as required by the CCT’s *Management of Urban Stormwater Impacts Policy* (City of

Cape Town, 2009b). In addition, runoff from the 1-year storm event is released over 24 hours as per the above Policy (not explicitly reported on as part of this study). However, despite the slow release of the lower recurrence interval events (1-year and 2-year events), on average, the peak discharge from the Scenario 4 attenuation facilities is 35% higher than the pre-development runoff peak.

- The attenuation implemented in conjunction with additional BMPs (Scenario 5) effectively reduced the post-development runoff peak to less than pre-development levels for the 50-, 10- and 2-year recurrence interval events.

### 3.9.2 Mosselbank River Model points of interest

Five points of interest, shown in **Figure 3.24**, were identified in the Mosselbank River Model for the purpose of comparison and assessment of peak flows at various locations within the catchment. The most pertinent characteristics of the catchments draining to these points of interest are provided in **Table 3.25**.

**Table 3.25: Mosselbank River Model points of interest**

Location	Total catchment size (km <sup>2</sup> )	Post-development % Imperviousness	Average catchment slope (%)
M-1	1.0	63	1.87
M-2	31.6	22	3.67
M-3	127.0	30	4.30
M-4	142.2	32	4.34
M-5	163.7	32	4.38

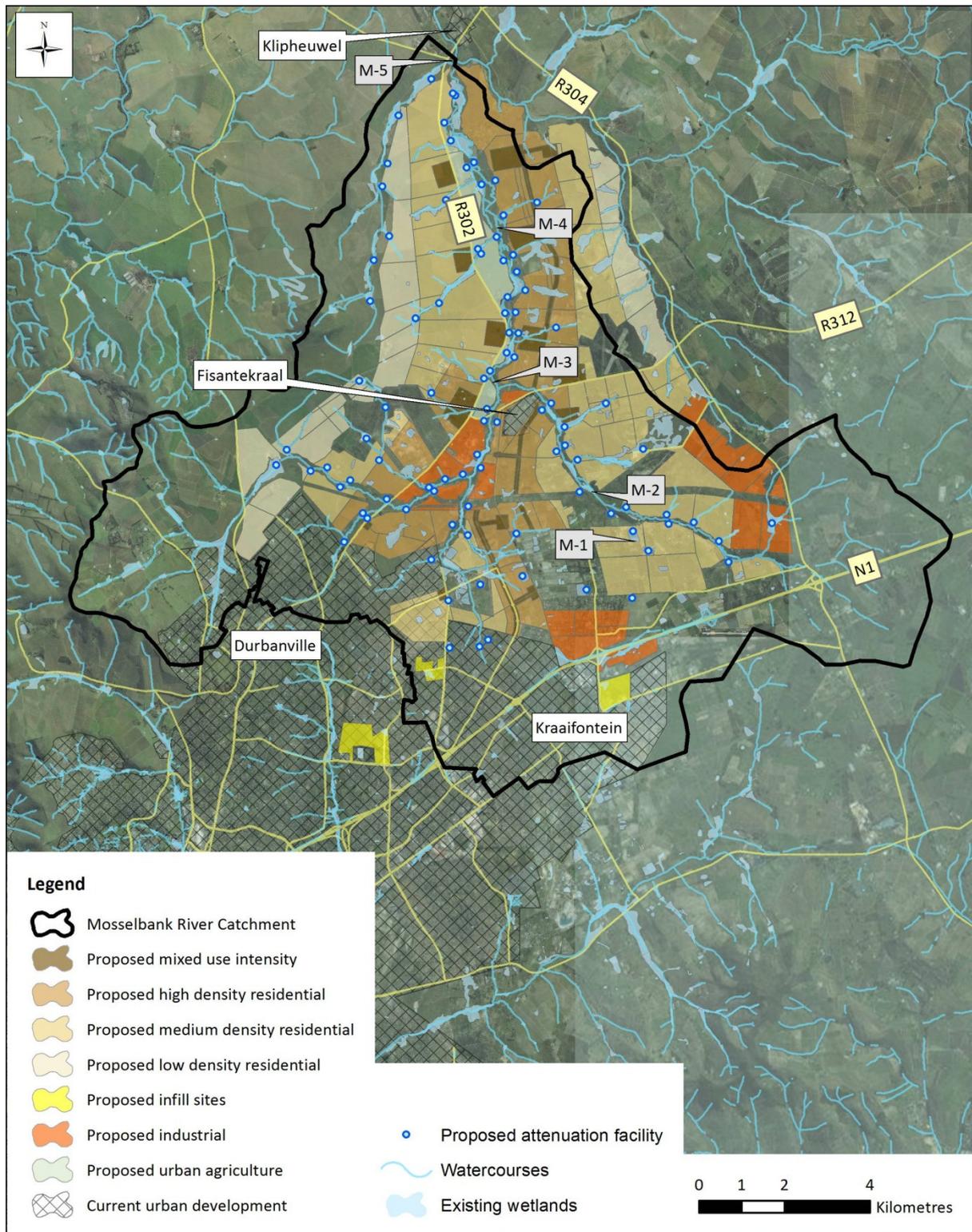


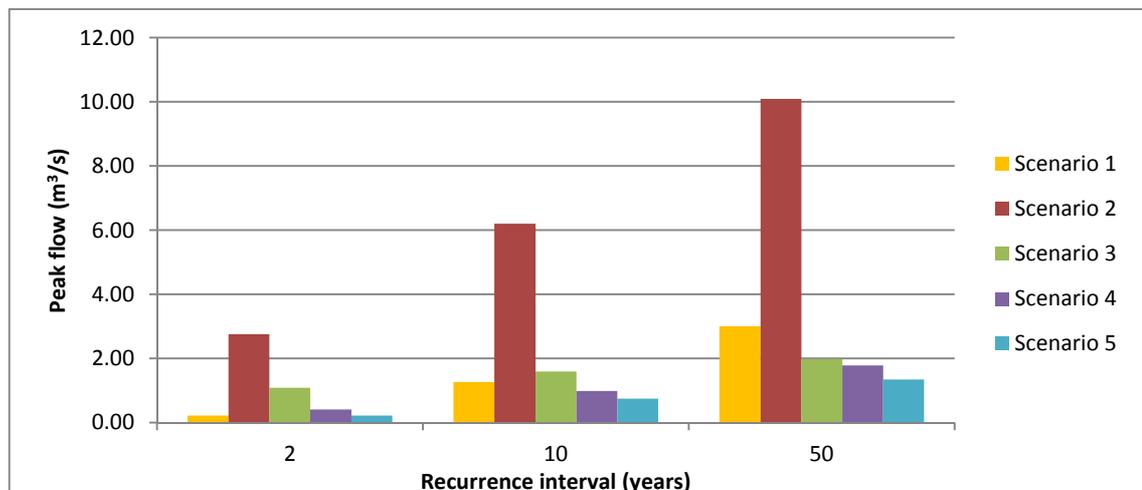
Figure 3.24: Mosselbank River Model

### 3.9.3 Mosselbank River design storm modelling results

The results of the five modelling scenarios listed in **Section 3.8** are provided in **Table 3.26** and shown graphically in **Figures 3.25** to **3.29** for the 2-year, 10-year and 50-year recurrence interval storm events at the five points of interest previously identified. A layout plan of the Mosselbank River Model is provided in **Appendix B (Figure B.1)** and a full list of results from the Mosselbank River Model is provided in **Appendix C**.

**Table 3.26: Summary of Mosselbank River design storm modelling results**

Location	Recurrence interval (years)	Peak flow (m <sup>3</sup> /s)					Total flow volume passing point of interest (1000 m <sup>3</sup> )		
		Scenario					Scenario 1	Scenario 2, 3, 4	Scenario 5
		1	2	3	4	5			
M-1	2	0.22	2.76	1.08	0.41	0.22	1.53	20.61	10.70
	10	1.26	6.20	1.59	0.98	0.75	11.30	39.32	26.50
	50	3.00	10.09	1.98	1.79	1.35	26.07	60.88	42.75
M-2	2	4.20	14.23	13.01	7.77	5.58	54.46	269.06	169.48
	10	25.31	38.67	36.13	27.74	25.45	370.48	687.92	558.71
	50	59.68	79.51	63.36	56.73	51.96	869.58	1259.04	1087.55
M-3	2	20.21	47.23	49.46	38.12	30.20	571.58	1429.27	1033.65
	10	96.90	135.61	141.96	117.94	107.47	2153.98	3424.61	2910.10
	50	211.14	269.88	253.64	221.08	207.14	4423.39	5852.78	5122.14
M-4	2	17.83	47.23	49.96	37.30	29.73	593.36	1685.15	1182.58
	10	95.18	139.68	148.64	126.70	114.07	2372.94	3966.69	3318.75
	50	214.83	272.80	267.18	234.01	218.96	4917.35	6739.11	5893.89
M-5	2	13.64	37.95	39.91	32.41	21.35	647.23	1780.04	1141.85
	10	98.14	131.07	138.06	108.93	95.96	2713.80	4393.08	3584.01
	50	209.51	273.16	275.08	222.50	203.08	5644.16	7610.90	6604.59



**Figure 3.25: Comparison of peak flows at Location M-1**

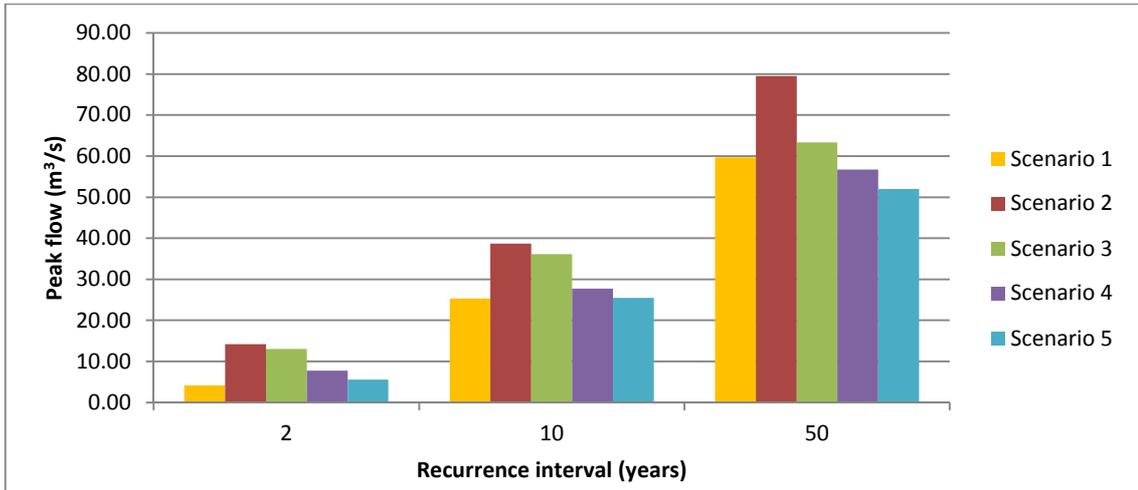


Figure 3.26: Comparison of peak flows at Location M-2

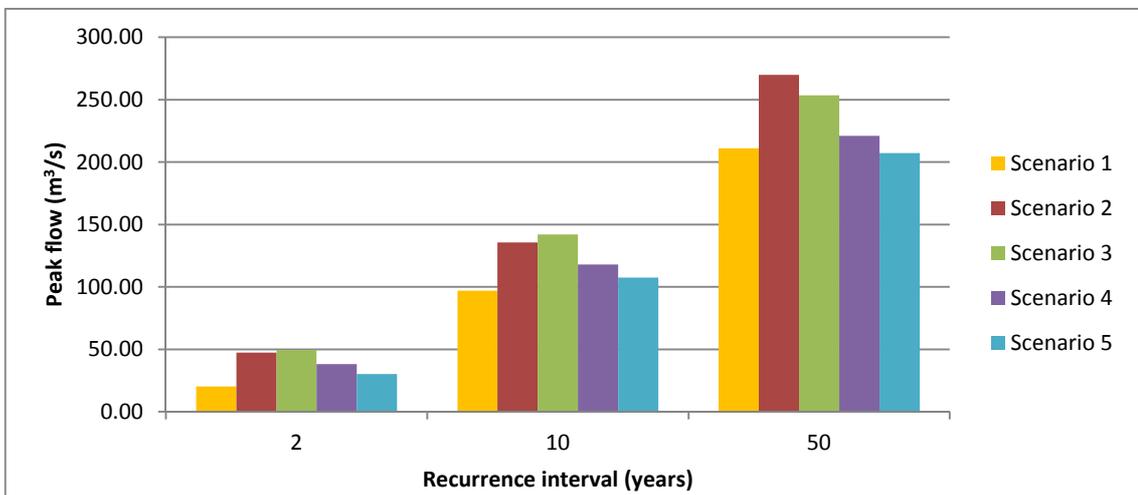


Figure 3.27: Comparison of peak flows at Location M-3

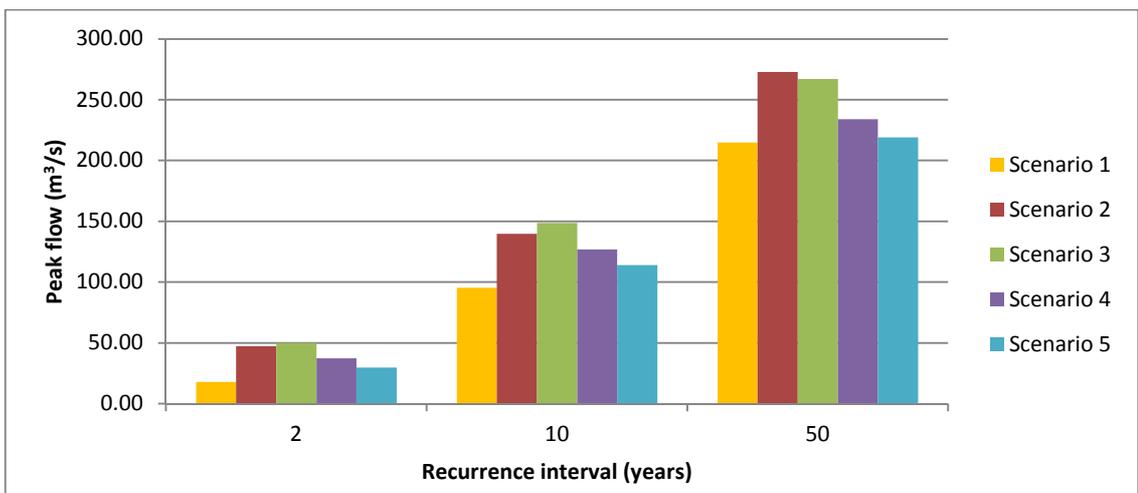
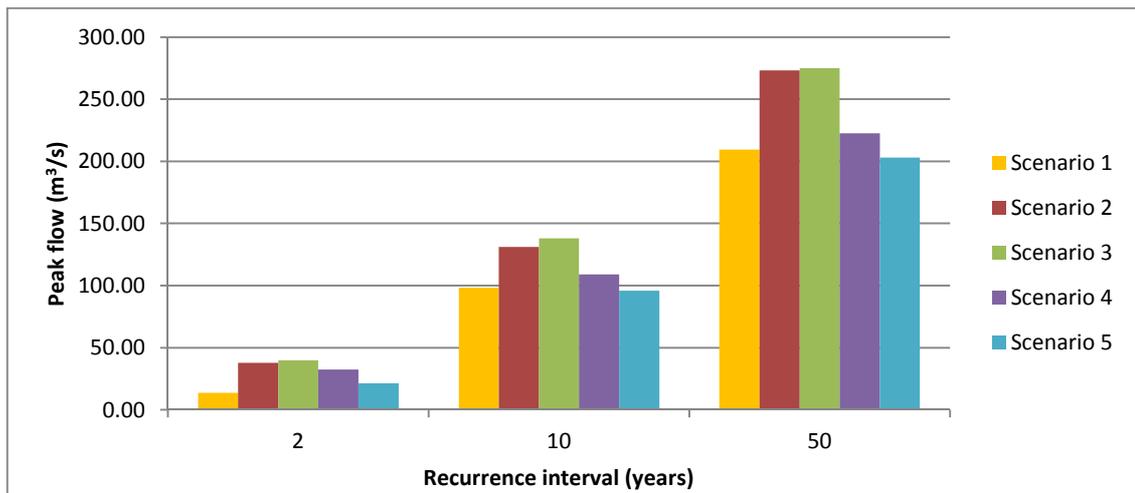


Figure 3.28: Comparison of peak flows at Location M-4



**Figure 3.29: Comparison of peak flows at Location M-5**

From **Table 3.26** and **Figures 3.25** to **3.29**, the following observations are noted:

- A comparison of pre-development (Scenario 1) and post-development (Scenario 2) peak flows shows that higher up in the catchment, urban development has a more significant impact on peak flow rates, whereas, lower down, the impact seems relatively less severe. This shows the effect of the natural storage and attenuation provided by the undeveloped floodplain of the Mosselbank River.
- An assessment of the Scenario 3 results shows that the approach of providing attenuation facilities which are only designed to eliminate the 50-year recurrence interval runoff is only effective for larger flood events, and only for a limited distance downstream of the facility (e.g. at M-1 and M-2). Further downstream, this approach was shown to in fact cause higher peak flows compared to the scenario without any attenuation facilities (i.e. Scenario 2) due to coinciding of more sub-catchment runoff peaks resulting from the changed catchment dynamics.
- The results of Scenario 4, in comparison to Scenario 3, show that attenuation facilities designed for multiple storm events are more effective at all locations and for all storm recurrence intervals modelled; however, the cumulative effect of the increased runoff volume starts impacting on the peak flows to a greater extent at the locations further downstream.
- Scenario 5 shows that by implementing a more sustainable approach to stormwater management by allowing more opportunity for stormwater runoff to infiltrate, peak flows downstream are reduced to close to pre-development levels.

### 3.9.4 Mosselbank River “continuous” modelling results

The Mosselbank River models representing the five attenuation scenarios listed in **Section 3.5** were also run with an extended rainfall data set recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir in 5-minute intervals from the beginning of May 2013 to the end of November 2013.

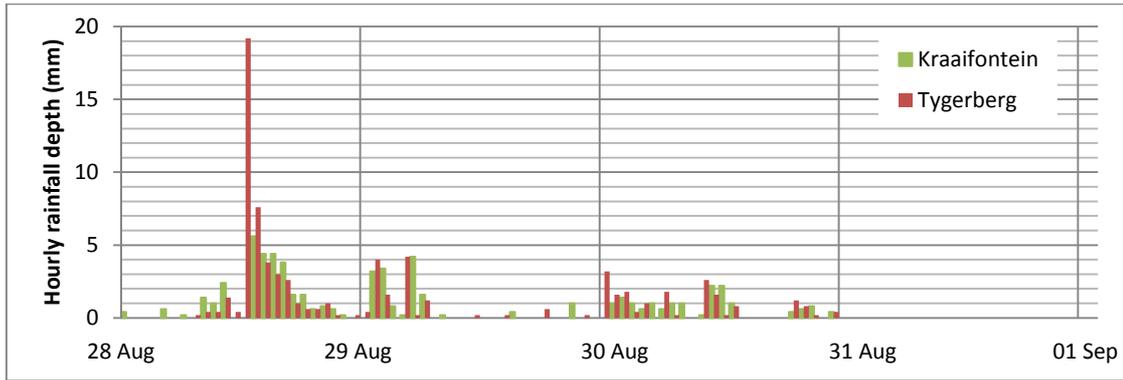
As noted in **Section 3.4.7**, the models have not been calibrated for continuous modelling in terms of simulating sub-surface base-flow continuing to feed the watercourses during dry periods. Instead, the purpose of these modelling simulations was to evaluate the various attenuation scenarios when subjected to actual rainfall events during the 7-month modelling period.

The flow hydrographs from three rainfall events at locations M-2 and M-5 (refer to **Figure 3.24**) were chosen to illustrate the effectiveness of the attenuation scenarios.

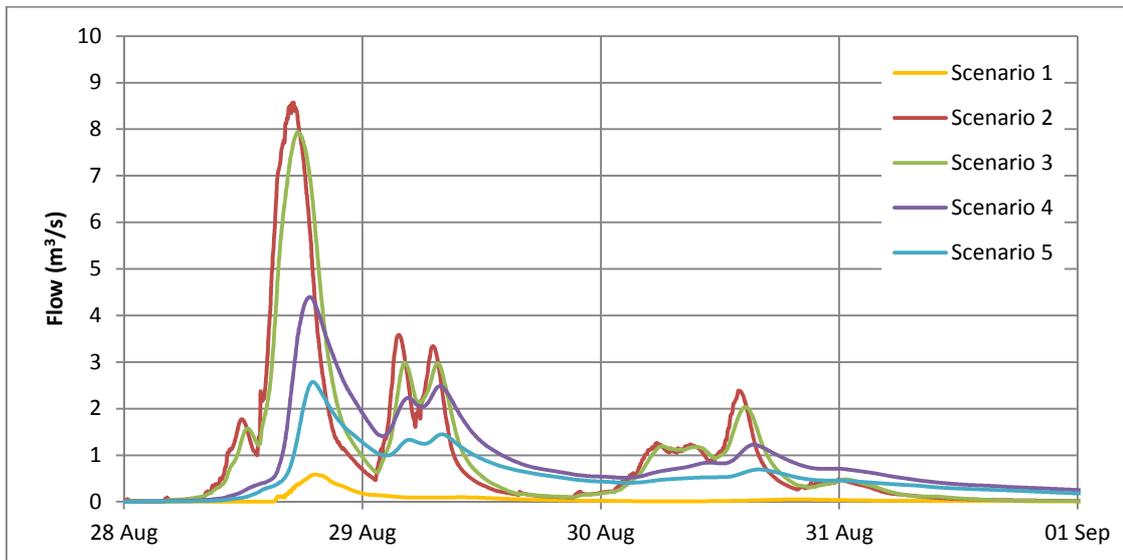
The first rainfall event, shown in **Figures 3.30 to 3.32**, was a moderate sized storm which occurred following a number of dry days. Over a 24-hour period, starting at 06h20 on 28 August 2013, a total rainfall depth of 54.2 mm was recorded at the Tygerberg Reservoir, while during the same period, 41.8 mm was recorded at the Kraaifontein Roads Depot. This corresponds approximately with a 2- to 5-year recurrence interval rainfall event in the vicinity of the Mosselbank River Catchment.

The second rainfall event, shown in **Figures 3.33 to 3.35**, consisted of two moderate sized storms occurring over three days. At the Tygerberg Reservoir, the first storm recorded a total 24-hour rainfall depth of 31.6 mm, and the second, 29.2 mm, while at the Kraaifontein Roads Depot, 42.6mm and 30.6mm was recorded over a 24 hour period for the respective storm events. Both of these storms correspond approximately with 1- to 2-year recurrence interval rainfall events.

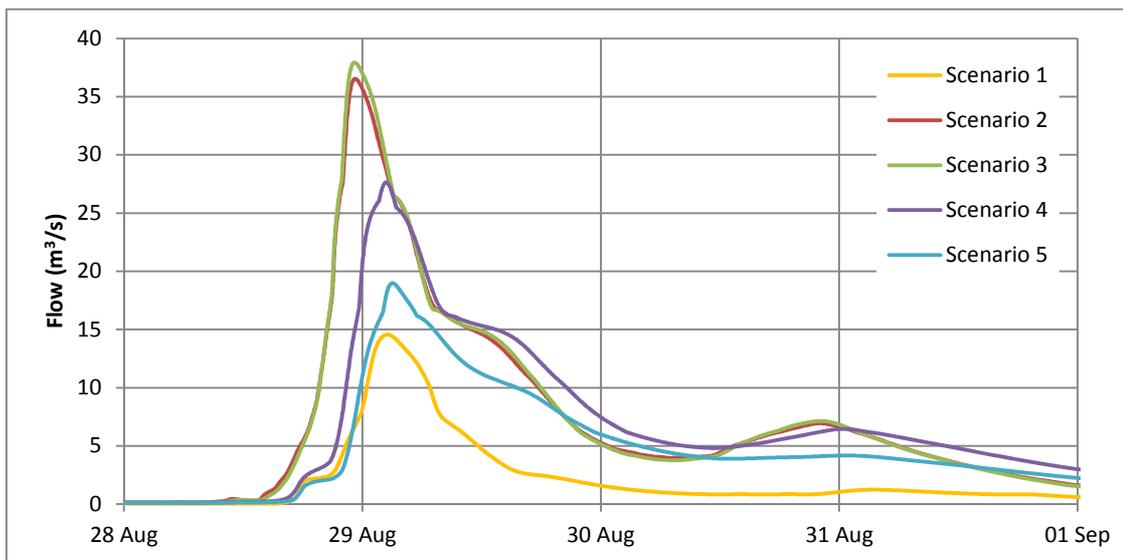
The third rainfall event, shown in **Figures 3.36 to 3.38**, represents a period of continuous light rainfall which would be typical of storm events with recurrence intervals of less than 1 year.



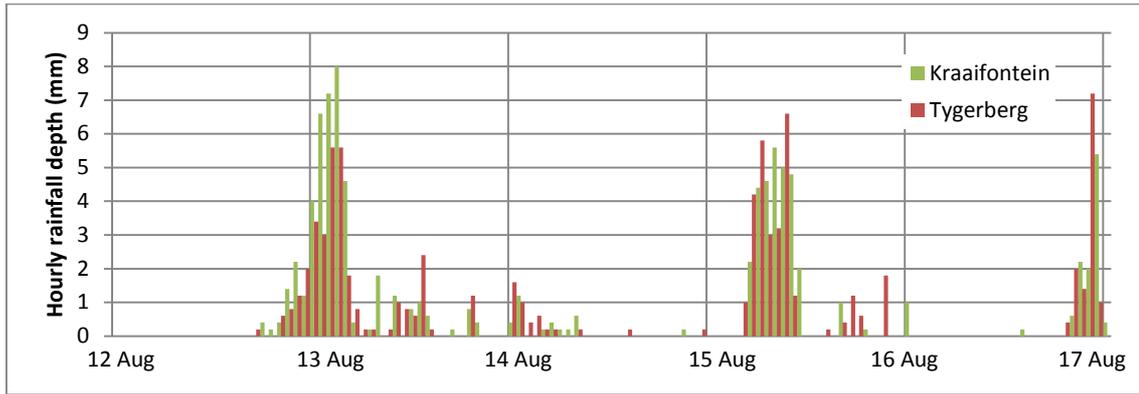
**Figure 3.30: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 28 and 31 August 2013**



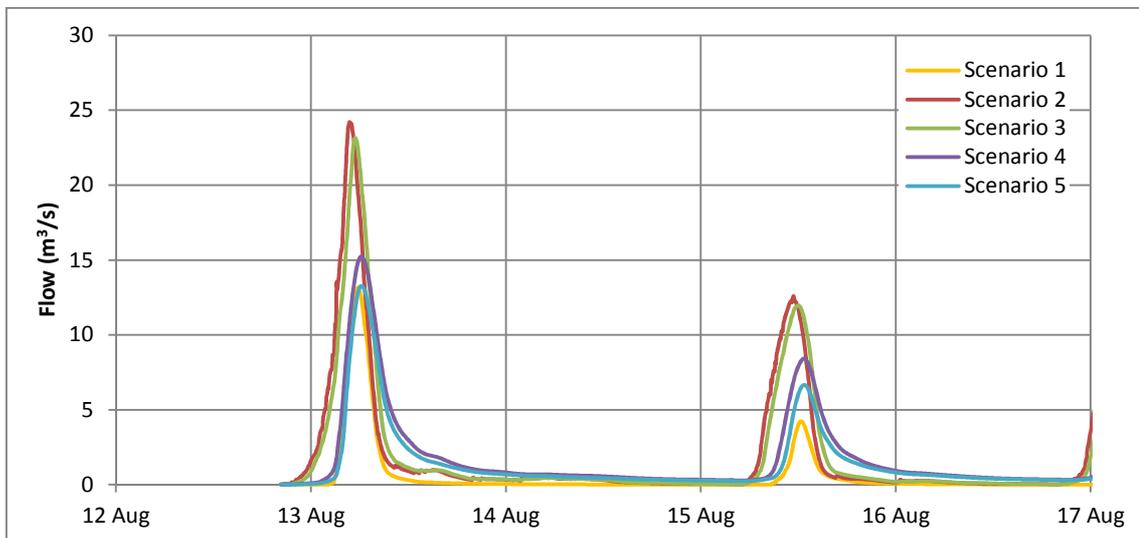
**Figure 3.31: Flow hydrograph at M-2 following heavy rainfall of 28 August 2013**



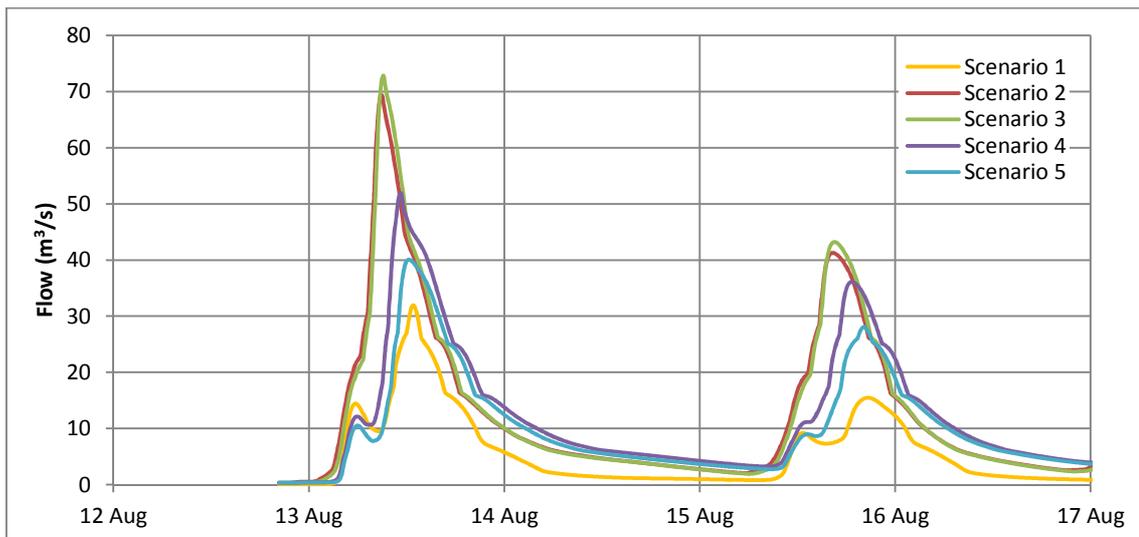
**Figure 3.32: Flow hydrograph at M-5 following heavy rainfall of 28 August 2013**



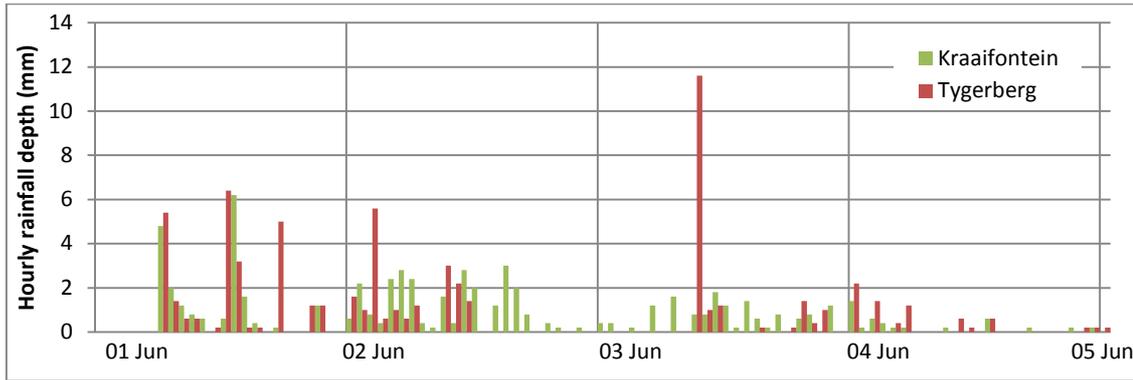
**Figure 3.33: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 12 and 16 August 2013**



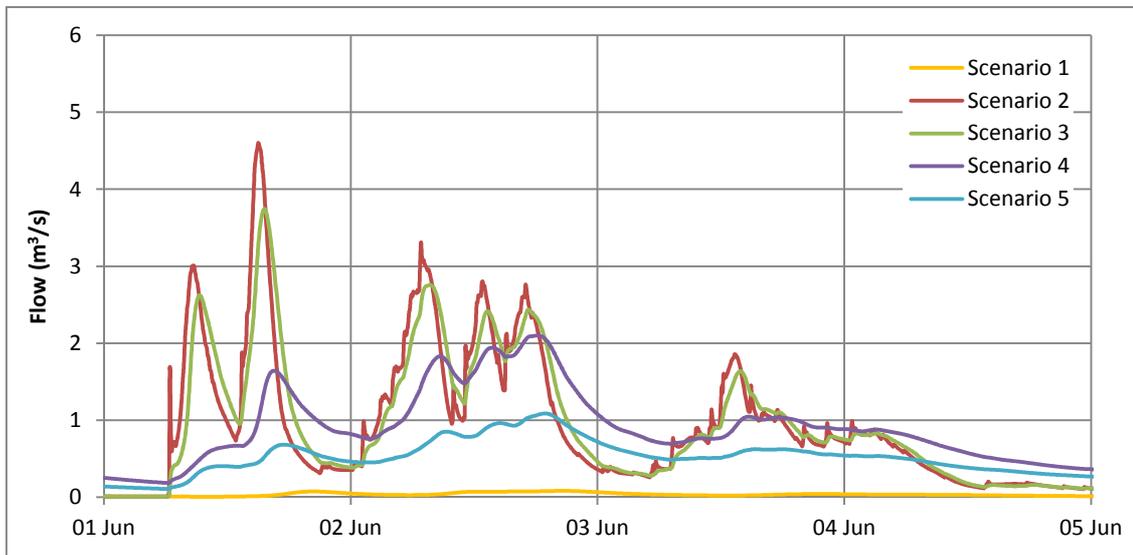
**Figure 3.34: Flow hydrograph at M-2 following rainfall between 12 and 16 August 2013**



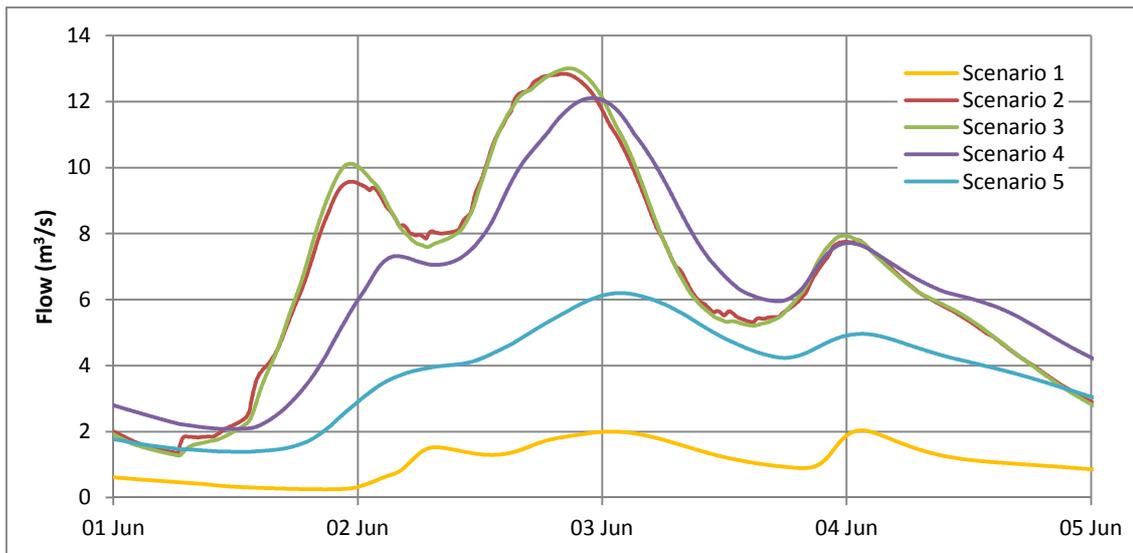
**Figure 3.35: Flow hydrograph at M-5 following rainfall between 13 and 16 August 2013**



**Figure 3.36: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 1 and 4 June 2013**



**Figure 3.37: Flow hydrograph at M-2 following rainfall between 1 and 4 June 2013**



**Figure 3.38: Flow hydrograph at M-5 following rainfall between 1 and 4 June 2013**

From **Figures 3.30 to 3.38**, the following observations are noted:

- A comparison of pre-development (Scenario 1) and post-development (Scenario 2) flows illustrates the significant impact that urban development has on natural flow patterns in receiving watercourses during small to moderate rainfall events.
- Attenuation Scenario 3 is generally ineffective for the small to moderate storm events evaluated, and in some cases results in higher peak flows compared to the post-development scenario without any attenuation. A detailed comparison of Scenarios 2 and 3 shows that in the scenario without any attenuation there is a more gradual increase in flow prior to the flood peak, whereas, with the slightly lagged flow resulting from the Scenario 3 attenuation, the runoff from upstream coincides to a greater extent with flow from catchments lower down in the catchment, resulting in slightly higher flood peaks.
- Scenario 4 shows that for the moderate storm event which occurred on 13 August 2013, the attenuation facilities upstream of location M-2 effectively attenuated flows to pre-development levels. However, at location M-5, the Scenarios 4 peak flows are approximately 60% higher than the pre-development peak flow due to the cumulative effect of the higher runoff volumes.
- Scenario 5 presents the best correlation between pre-development and attenuated post-development flows, however, this scenario still resulted in higher peak flows downstream due to the cumulative effect of the higher runoff volumes. Considering the constraints associated with reducing runoff volume through increased infiltration, it could perhaps be argued that it would not be practical to reduce runoff rates further unless attenuation storage at each sub-catchment is increased to the extent that flows downstream are reduced to less than pre-development levels.

### **3.10 Bayside Canal Model results**

#### **3.10.1 Attenuation results at sub-catchment level**

For Scenarios 3, 4 and 5, each sub-catchment within the Bayside Canal Model was drained to a hypothetical attenuation facility, modelled according to the assumptions described in **Section 3.8**. As was presented for the Mosselbank River Model, **Table 3.27** provides the average figures for the percent difference of the maximum attenuated outlet flows from the sub-catchments compared to pre-development runoff peaks.

**Table 3.27: Comparison of sub-catchment post-development and pre-development peak runoff**

Recurrence interval (years)	Average % difference of maximum post-development outlet discharge from individual sub-catchments in comparison to pre-development peak runoff			
	Scenario 2	Scenario 3	Scenario 4	Scenario 5
2	*	*	*	*
10	2183%	289%	96%	-17%
50	619%	-2%	-10%	-53%

\*Could not be determined because pre-development runoff = 0 m<sup>3</sup>/s

From **Table 3.27**, the following is noted:

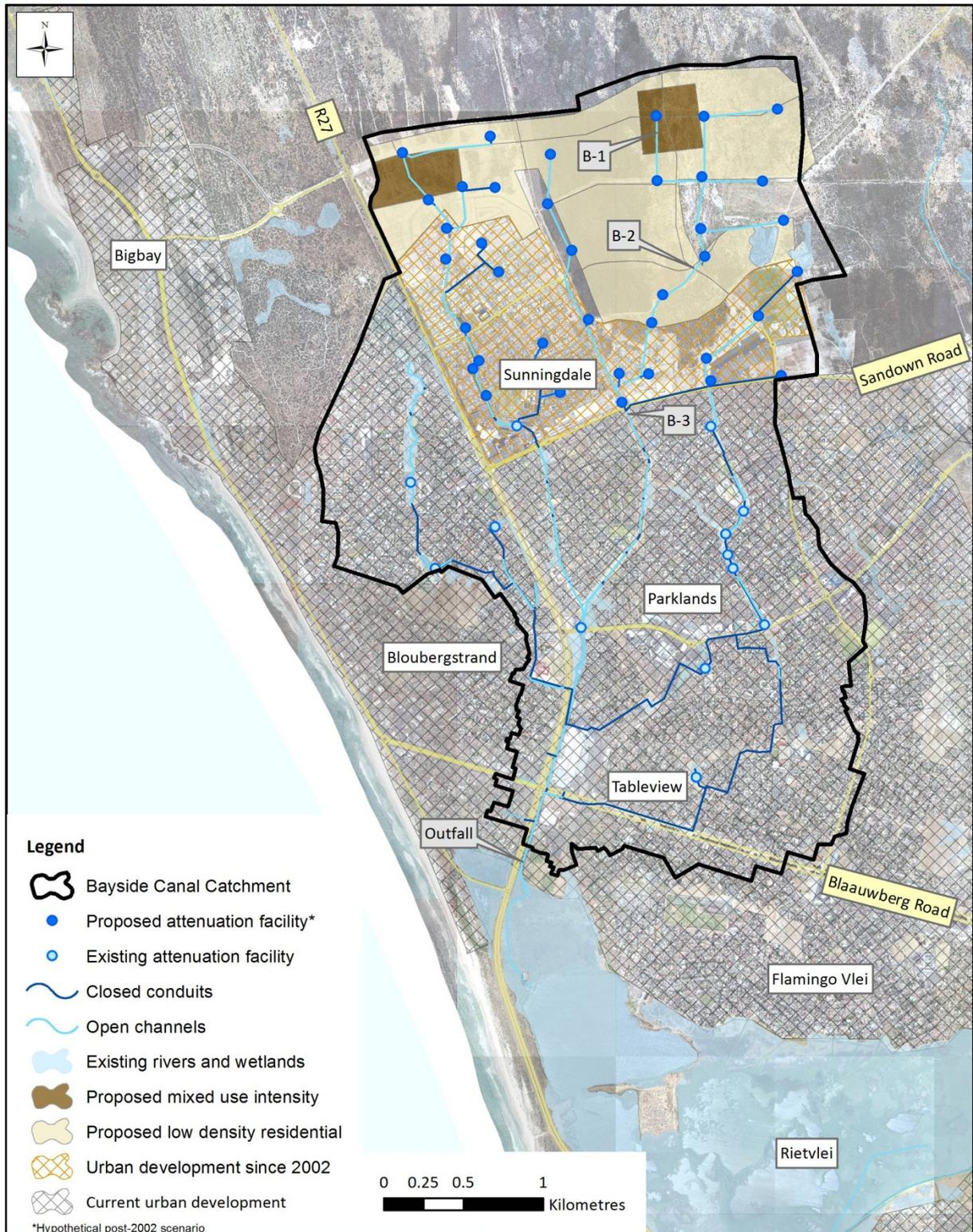
- Since the pre-development 2-year storm event effectively produced no runoff, it was impractical to reduce post-development runoff completely to pre-development levels.
- The implementation of attenuation facilities with single-stage outlet structures (Scenario 3) only tends to reduce the 50-year post-development runoff peak to less than the equivalent pre-development runoff peak at each sub-catchment. The outlet flows for the 10-year and 2-year recurrence interval events are significantly higher than the pre-development runoff peaks, although less than the scenario without any attenuation (Scenario 2).
- The multi-stage outlet structures simulated in Scenario 4 effectively reduced the 50-year runoff peaks by an average of 10% compared to the pre-development runoff peaks, however, due to practical limitations in terms of outlet sizes, and the low 10-year runoff rates, it was not possible to implement attenuation at the Bayside Canal Model sub-catchments which would completely reduce the 10-year runoff to pre-development levels.
- The attenuation implemented in conjunction with additional BMPs (Scenario 5) effectively reduced the post-development runoff peak to less than pre-development levels for the 50- and 10-year recurrence interval events.

### 3.10.2 Bayside Canal Model points of interest

Four points of interest, shown in **Figure 3.39**, were identified in the Bayside Canal Model for the purpose of comparison and assessment. The most pertinent characteristics of the catchments draining to these points of interest are provided in **Table 3.28**.

**Table 3.28: Bayside Canal Model points of interest**

Location	Total catchment size (km <sup>2</sup> )	Post-development % Imperviousness	Average catchment slope (%)
B-1	0.18	52	1
B-2	1.31	48	1
B-3	3.33	48	1
Outfall	12.57	47	1



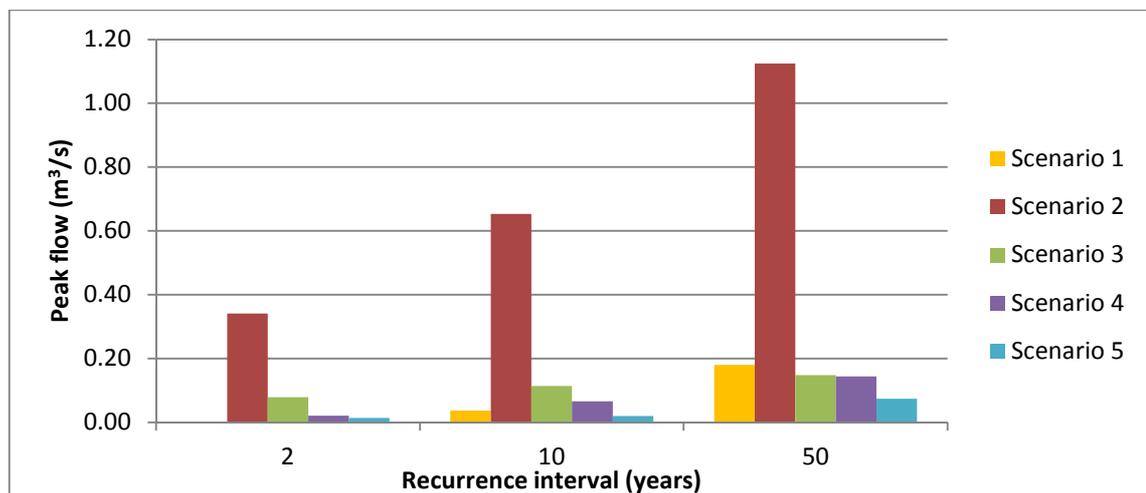
**Figure 3.39: Bayside Canal Model**

### 3.10.3 Bayside Canal design storm modelling results

The results of the five modelling scenarios listed in **Section 3.8** are provided in **Table 3.29** and shown graphically in **Figures 3.40** to **3.43** for the 2-year, 10-year and 50-year recurrence interval storm events. A layout plan of the Bayside Canal Model is provided in **Appendix B (Figure B.2)** and a full list of results from the Bayside Canal Model is provided in **Appendix D**.

**Table 3.29: Summary of Bayside Canal design storm modelling results**

Location	Recurrence interval (years)	Peak flow (m <sup>3</sup> /s)					Total flow volume passing point of interest (1000 m <sup>3</sup> )		
		Scenario					Scenario 1	Scenario 2, 3, 4	Scenario 5
		1	2	3	4	5			
B-1	2	0.00	0.34	0.08	0.02	0.01	0.00	2.72	1.05
	10	0.04	0.65	0.11	0.07	0.02	0.13	5.06	2.15
	50	0.18	1.12	0.15	0.14	0.07	0.97	8.24	3.91
B-2	2	0.00	1.17	0.32	0.11	0.07	0.00	13.29	4.67
	10	0.08	2.40	0.45	0.29	0.10	0.54	6.47	9.87
	50	0.71	4.25	0.59	0.65	0.34	4.45	40.69	18.46
B-3	2	0.00	2.49	0.85	0.29	0.18	0.00	32.51	13.51
	10	0.13	4.90	1.25	0.71	0.30	1.27	62.61	26.85
	50	1.61	6.31	1.63	1.65	0.87	10.85	101.60	50.43
Outfall	2	5.43	6.60	5.93	5.70	5.56	67.31	117.02	86.27
	10	9.27	11.58	10.03	9.61	9.44	122.95	217.20	159.51
	50	13.67	14.95	14.27	13.95	13.83	214.34	349.67	269.82



**Figure 3.40: Comparison of peak flows at Location B-1**

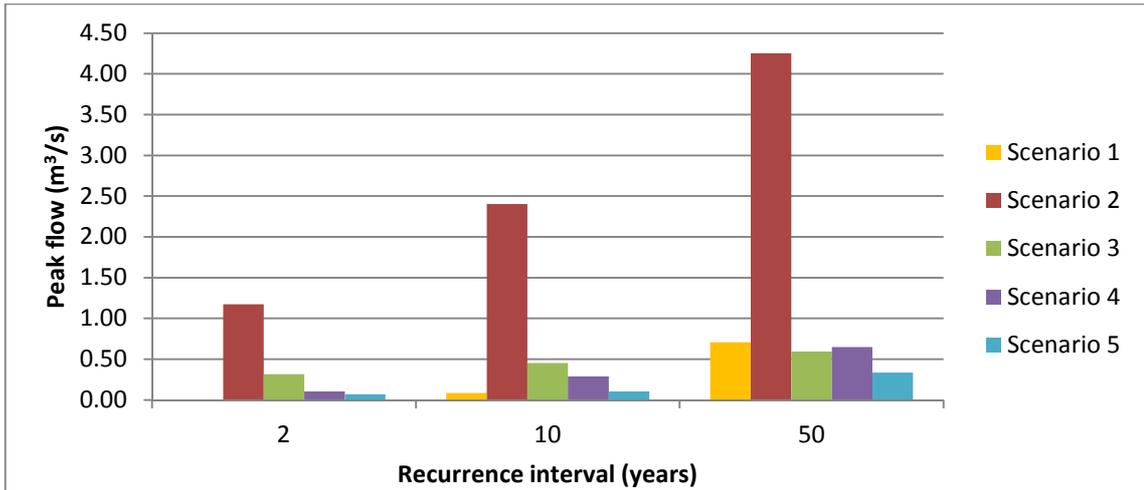


Figure 3.41: Comparison of peak flows at Location B-2

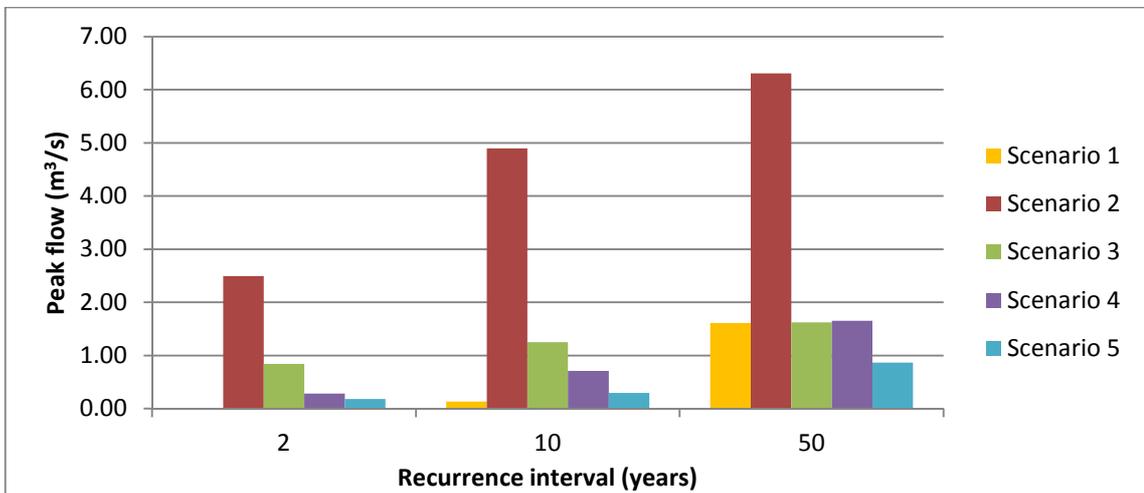


Figure 3.42: Comparison of peak flows at Location B-3

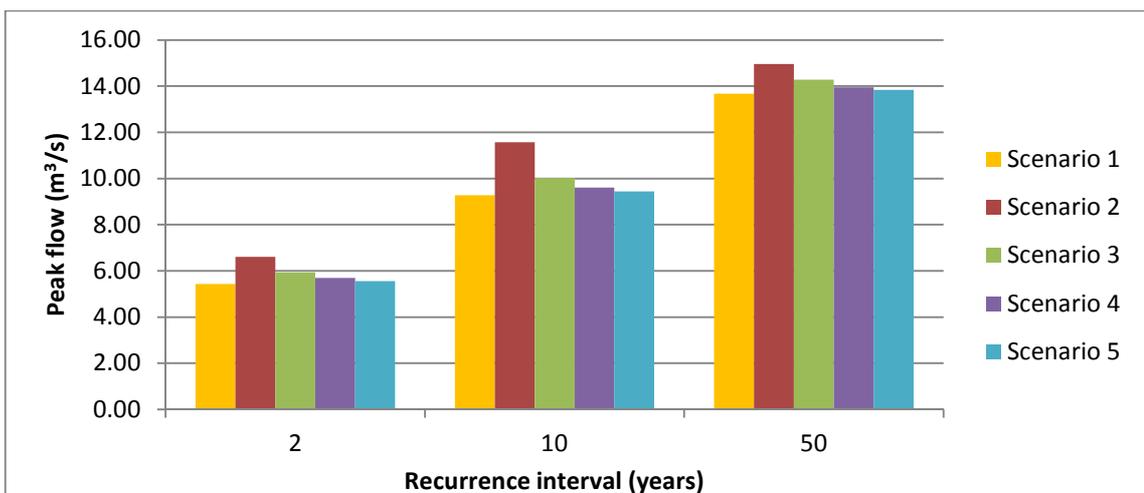


Figure 3.43: Comparison of peak flows at the Bayside Canal outfall

From **Table 3.29** and **Figures 3.40** to **3.43**, the following observations are noted:

- Due to the nature of the natural topography and ground conditions in the Bayside Canal catchment area, pre-development runoff was very low and very few defined watercourses existed in the area, as discussed in **Section 3.2.2**. This low runoff is shown in the modelling results. In the case of the 2-year recurrence interval storm, the model shows that there is no runoff from the storm event.
- In contrast to the very low pre-development runoff, the post-development runoff is many orders of magnitude greater, both in terms of runoff rates and volumes.
- The results show that Scenarios 3, 4 and 5 are all effective in reducing the 50-year post-development runoff to pre-development levels. Because, the Bayside Canal catchment area is significantly smaller than the Mosselbank River Catchment, the cumulative effect of increased runoff volumes does not have a significant effect.
- Scenarios 3 and 4 are not as effective in reducing the 10- and 2-year post-development runoff to pre-development levels, however, Scenario 5 is significantly more effective. This indicates the value of introducing BMPs within a catchment which has a high natural infiltration rate.

#### 3.10.4 Bayside Canal “continuous” modelling results

The Bayside Canal models representing the five attenuation scenarios listed in **Section 3.8** were also run with an extended rainfall data set recorded at the Potsdam Wastewater Treatment Works in 5-minute intervals from the beginning of May 2013 to the end of November 2013.

As noted in **Section 3.4.7**, the models have not been calibrated for continuous modelling in terms of simulating sub-surface baseflow continuing to feed the watercourses during dry periods. Instead, the purpose of these modelling simulations was to evaluate the various attenuation scenarios when subjected to actual rainfall events.

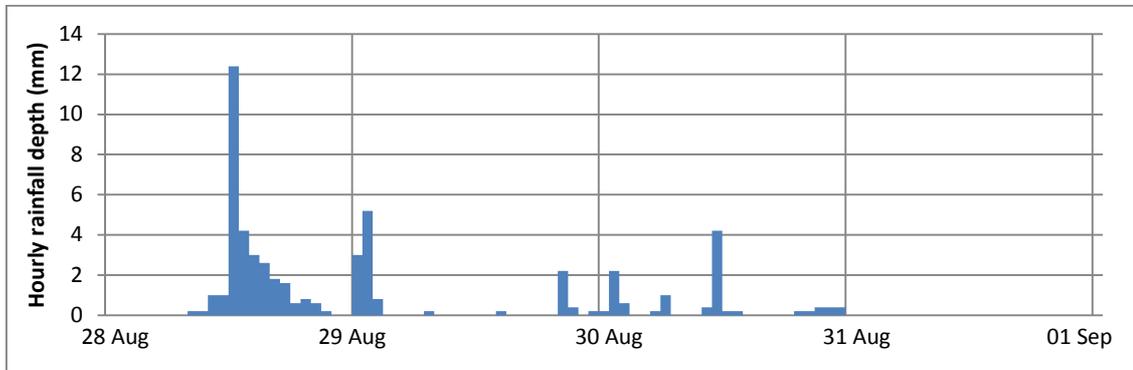
The flow hydrographs from three rainfall events at locations B-2 and B-3 (refer to **Figure 3.20**) were chosen to illustrate the effectiveness of the attenuation scenarios.

The first rainfall event, shown in **Figures 3.44** to **3.46**, was the largest storm experienced at the Potsdam Wastewater Treatment Works during the year of 2013. Over a 24-hour period, starting at 07h40 on 28 August 2013, a total rainfall depth of 39.4 mm was recorded. This corresponds approximately with a 2- to 5-year recurrence interval storm event in the vicinity of the Bayside Canal Catchment.

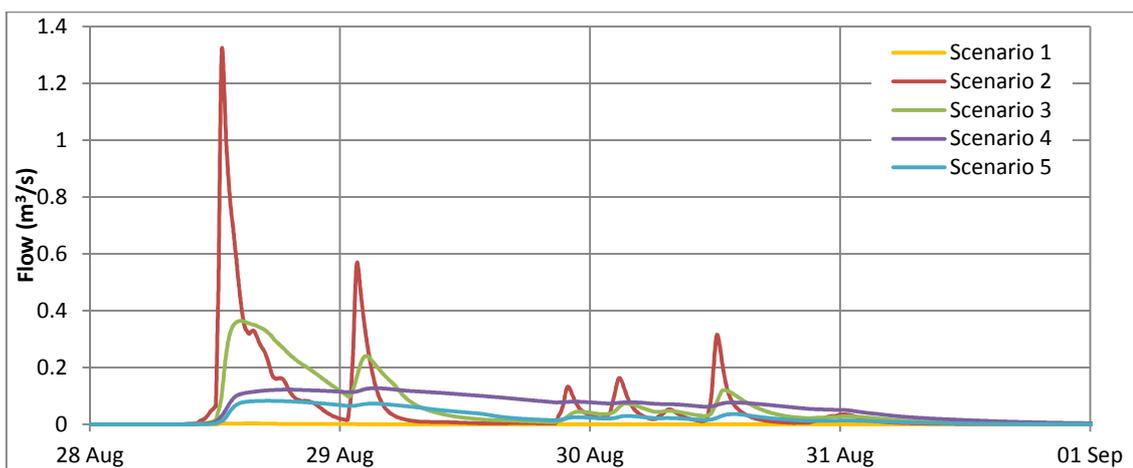
The second rainfall event, shown in **Figures 3.47** to **3.49**, consisted of two moderate sized storms occurring over three days. The first storm recorded a total 24-hour rainfall depth of

20.0 mm, and the second, 19.0 mm. Both of these storms correspond approximately with 0.5- to 1-year recurrence interval storm events.

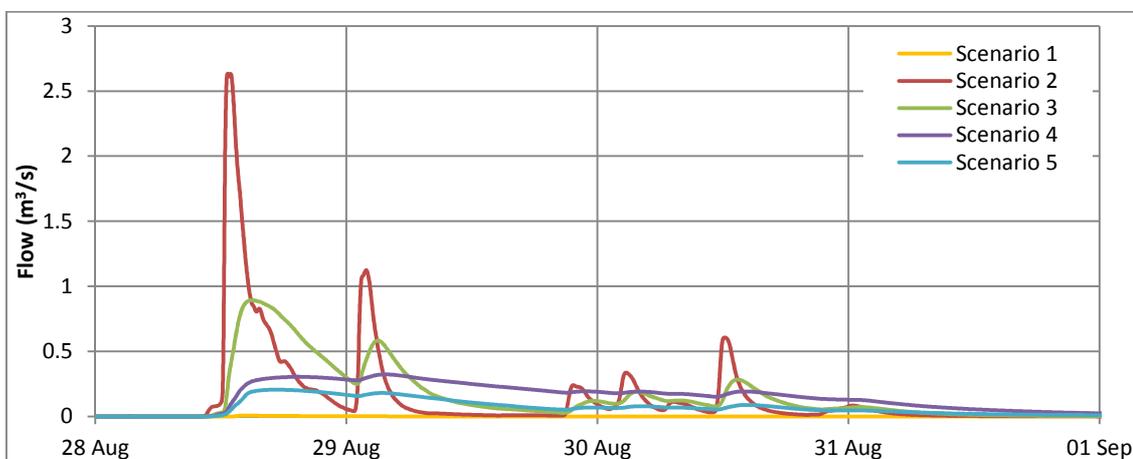
The third rainfall event, shown in **Figures 3.50 to 3.51**, represents a period of continuous light rainfall which would be typical of storm events with recurrence intervals of less than 1 year.



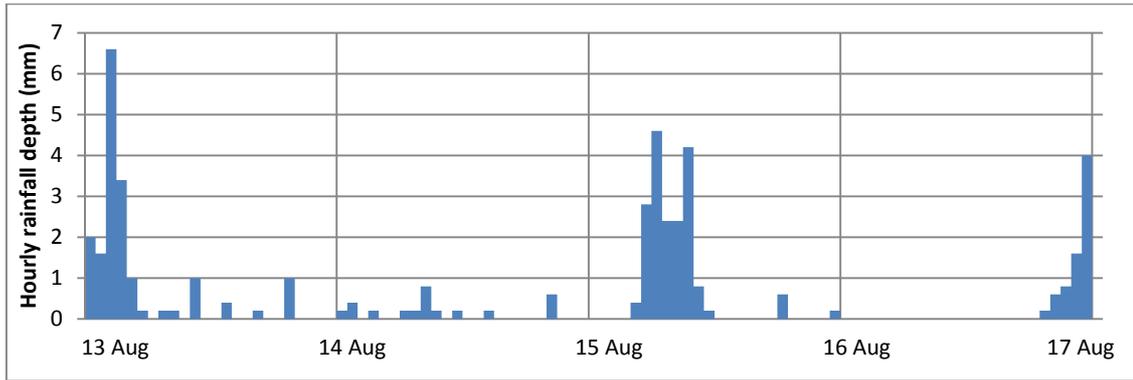
**Figure 3.44: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 28 and 31 August 2013**



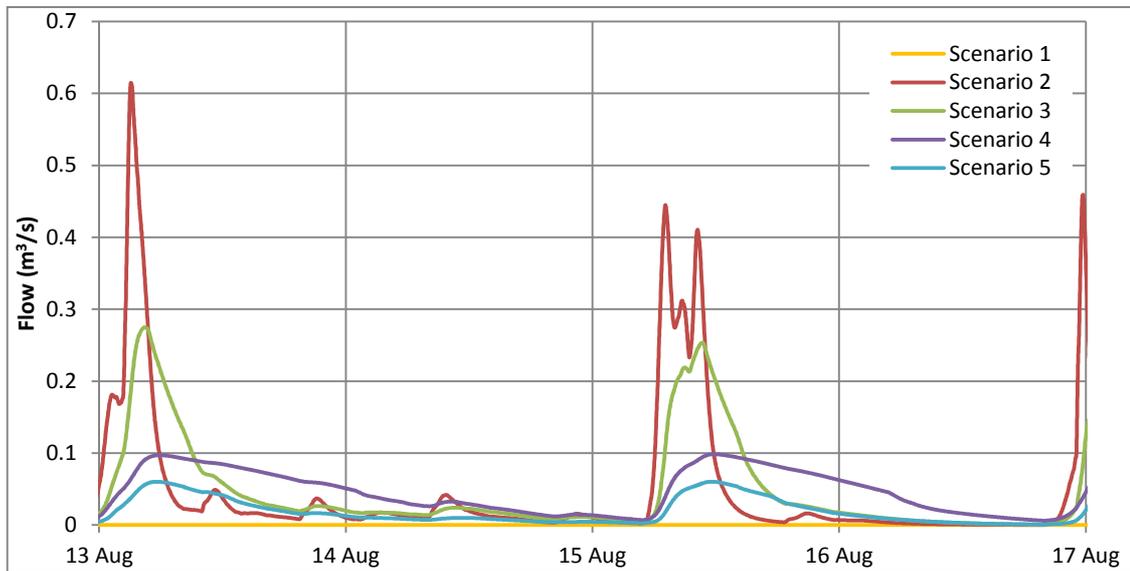
**Figure 3.45: Flow hydrograph at B-2 following heavy rainfall of 28 August 2013**



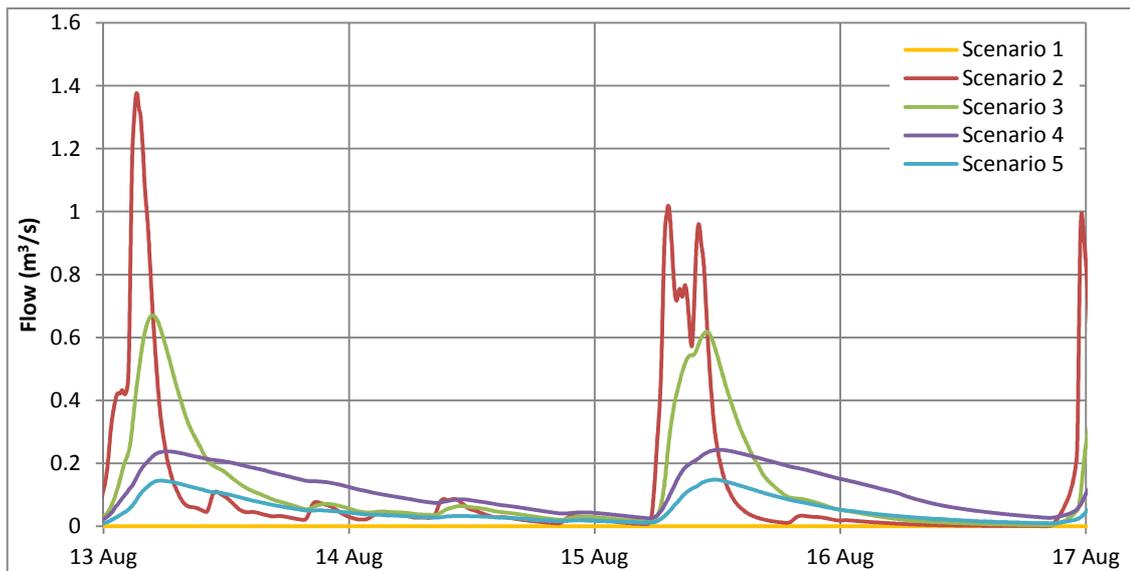
**Figure 3.46: Flow hydrograph at B-3 following heavy rainfall of 28 August 2013**



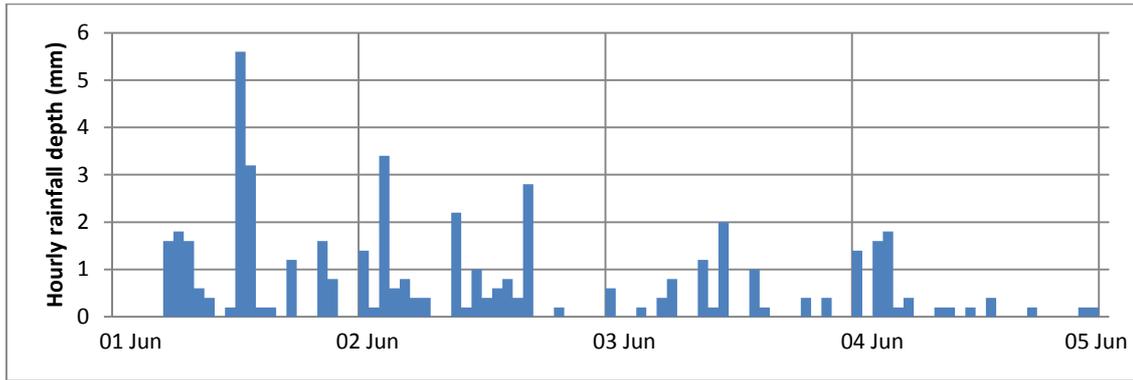
**Figure 3.47: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 13 and 16 August 2013**



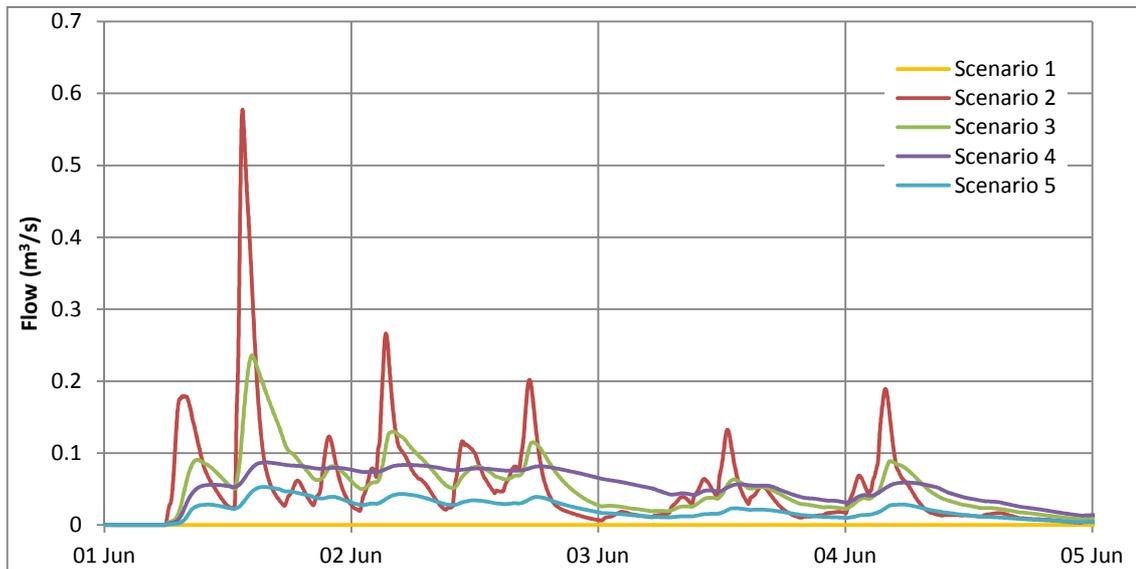
**Figure 3.48: Flow hydrograph at B-2 following rainfall between 13 and 16 August 2013**



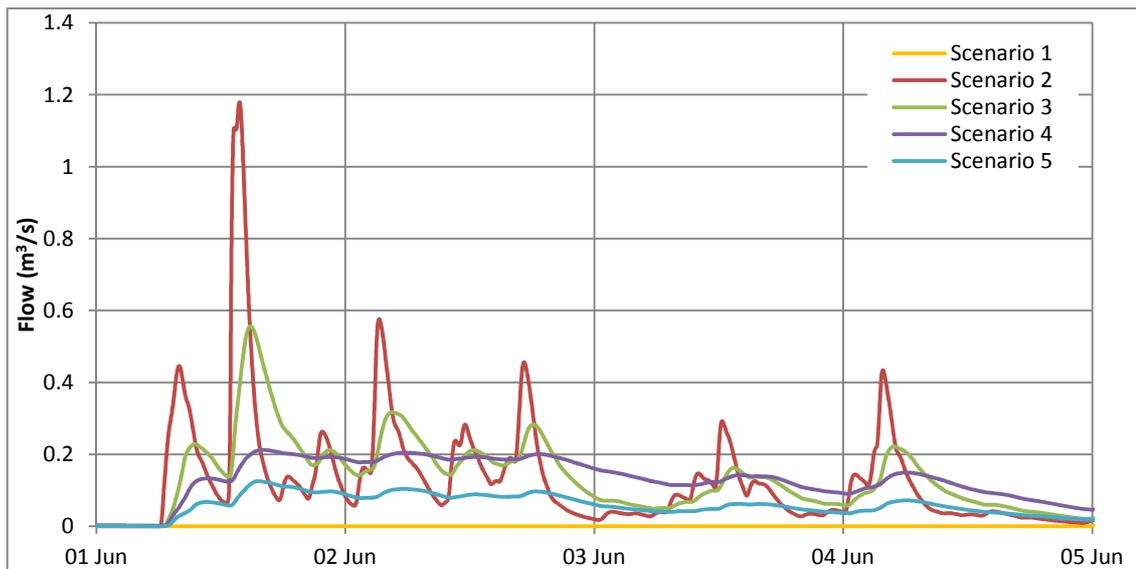
**Figure 3.49: Flow hydrograph at B-3 following rainfall between 13 and 16 August 2013**



**Figure 3.50: Hourly rainfall depths recorded at the Potsdam Wastewater Treatment Works between 1 and 4 June 2013**



**Figure 3.51: Flow hydrograph at B-2 following rainfall between 1 and 4 June 2013**



**Figure 3.52: Flow hydrograph at B-3 following rainfall between 1 and 4 June 2013**

From **Figures 3.44 to 3.52**, the following observations are noted:

- The pre-development modelling scenario (Scenario 1) shows almost no flow during any of the rainfall events. This is expected due to the very low runoff potential of the natural catchment ground surface, as described previously in **Section 3.2.2**.
- The single outlet attenuation option included in modelling Scenario 3 is shown to reduce the post-development hydrograph peaks to some degree and is more effective for larger rainfall events and higher up in the catchment (i.e. at Location B-2).
- The results from modelling scenarios 4 and 5 show significant reductions in peak flows compared to the post-development scenario without any attenuation. Although these attenuation options are unable to mimic pre-development conditions, they provide adequate control during storms so that the constructed “natural” watercourse which receives the runoff from the urban development will not flood or erode during storm events.

### 3.11 Upper Kuils River Model results

#### 3.11.1 Attenuation results at sub-catchment level

Due to the complex nature of the existing stormwater system in the Upper Kuils River Catchment, and the fact that many of the natural sub-catchments and streams discharging into the Upper Kuils River have been altered from their pre-development state, it was not possible to directly compare the attenuation facility outflows with equivalent pre-development flows.

#### 3.11.2 Upper Kuils River Model points of interest

Two points of interest, shown in **Figure 3.53**, were identified in the Upper Kuils River Model for the purpose of comparison and assessment. The most pertinent characteristics of the catchments draining to these points of interest are provided in **Table 3.30**:

**Table 3.30: Upper Kuils River Model points of interest**

Location	Total catchment size (km <sup>2</sup> )	Post-development % Imperviousness	Average catchment slope (%)
De Villiers Drive	5.99	40	4.56
N1	22.24	32	5.92

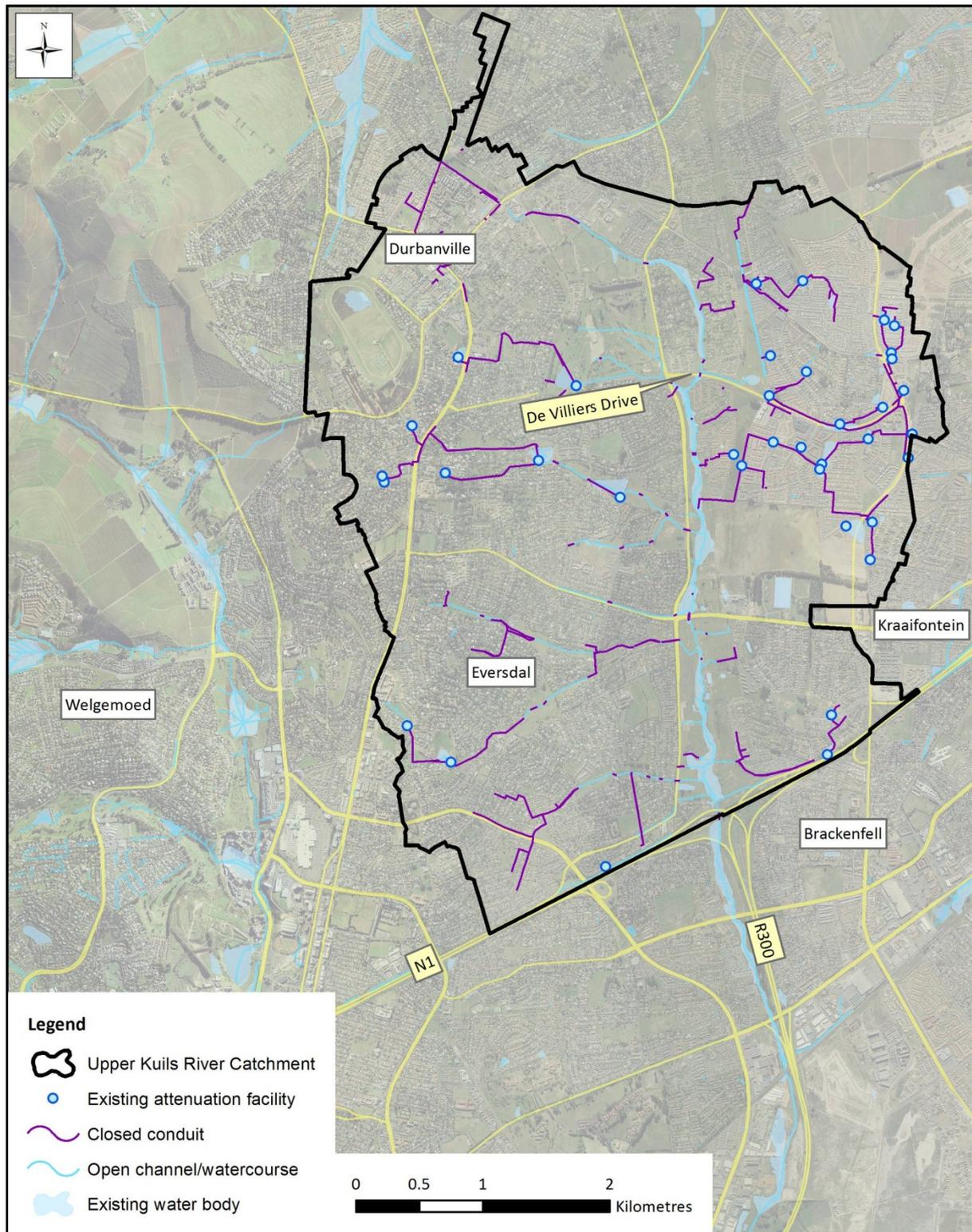


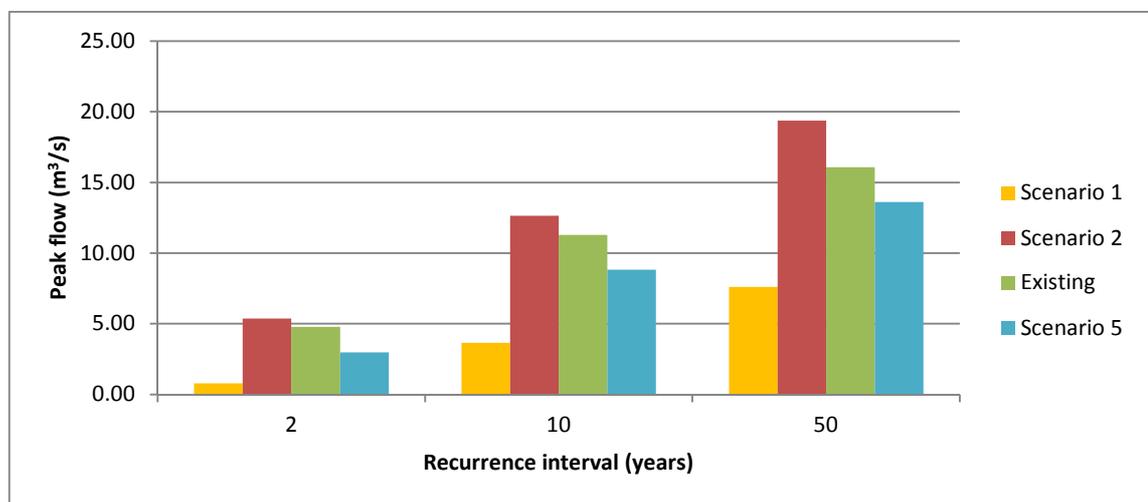
Figure 3.53: Upper Kuils River bulk stormwater system

### 3.11.3 Upper Kuils River design storm modelling results

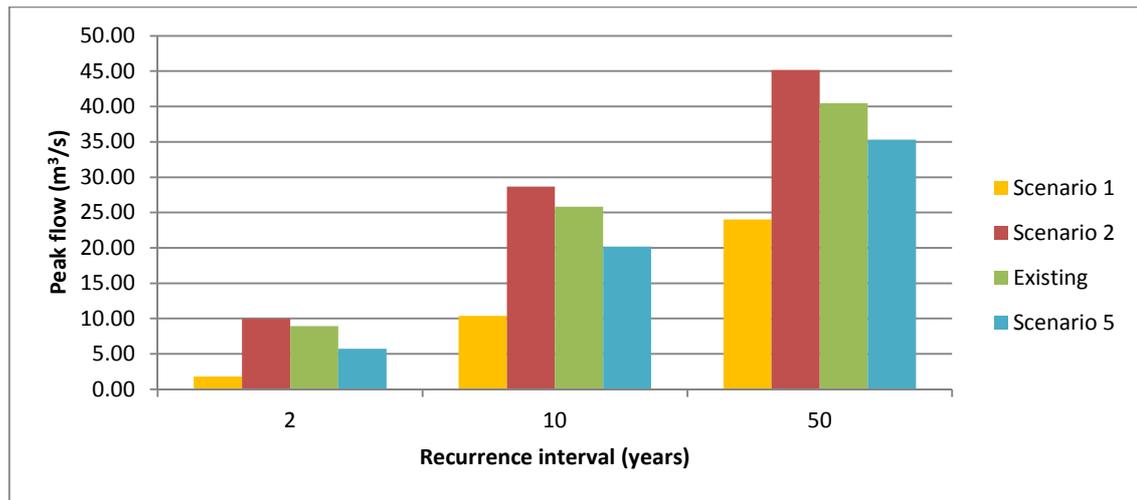
The results of the modelling scenarios listed in **Section 3.8** are provided in **Table 3.31** and shown graphically in **Figures 3.54** and **3.55** for the 2-year, 10-year and 50-year recurrence interval storm events. A layout plan of the Upper Kuils River Model is provided in **Appendix B (Figure B.3)** and a full list of results from the Upper Kuils River Model is provided in **Appendix E**.

**Table 3.31: Summary of Upper Kuils River design storm modelling results**

Location	Recurrence interval (years)	Peak flow (m <sup>3</sup> /s)				Total flow volume passing point of interest (1000 m <sup>3</sup> )		
		Scenario				Scenario 1	Scenario 2, Existing attenuation	Scenario 5
		1	2	Existing attenuation	5			
De Villiers Drive	2	0.79	5.37	4.79	2.97	14.74	86.28	64.52
	10	3.67	12.65	11.28	8.82	53.08	168.21	135.39
	50	7.61	19.37	16.07	13.62	125.33	278.10	235.97
N1	2	1.79	9.98	8.92	5.74	30.09	215.41	148.64
	10	10.40	28.68	25.84	20.17	182.14	504.80	399.80
	50	24.03	45.16	40.45	35.33	457.42	879.71	752.77



**Figure 3.54: Comparison of peak flows in the Upper Kuils River at De Villiers Drive**



**Figure 3.55: Comparison of peak flows in the Upper Kuils River at the N1**

From **Table 3.31** and **Figures 3.54** and **3.55**, the following observations are noted:

- A comparison of pre-development (Scenario 1) and post-development (Scenario 2) peak flows shows quite a significant increase in flow for all recurrence interval storm events.
- Current attenuation measures which have been implemented within the catchment (“Existing” Scenario) are not very effective along the Upper Kuils River.
- The results of Scenario 5, which includes the introduction of retrospective BMPs, show a significant reduction in the post-development peak flows.

#### 3.11.4 Upper Kuils River “continuous” modelling results

The Upper Kuils River models representing the various attenuation scenarios listed in **Section 3.8** were also run with an extended rainfall data set recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir in 5-minute intervals from the beginning of May 2013 to the end of November 2013.

As noted in **Section 3.4.4**, the models have not been calibrated for continuous modelling in terms of simulating sub-surface baseflow continuing to feed the watercourses during dry periods. Instead, the purpose of these modelling simulations was to evaluate the various attenuation scenarios when subjected to actual rainfall events.

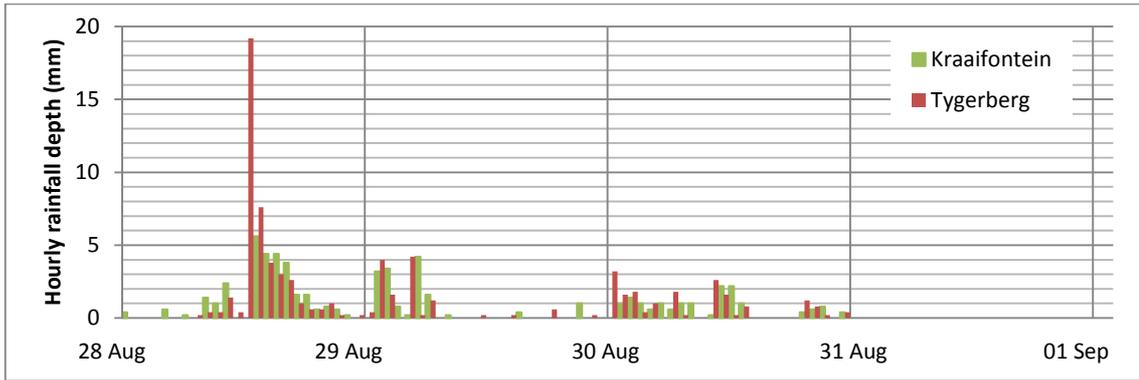
The flow hydrographs from three rainfall events at De Villiers Drive and at the N1 (refer to **Figure 3.53**) were chosen to illustrate the effectiveness of the attenuation scenarios.

As described previously in **Section 3.9.4** for the Mosselbank River Model, the first rainfall event, shown in **Figures 3.56** to **3.58**, was a moderate sized storm which occurred following a number of dry days. Over a 24-hour period, starting at 06h20 on 28 August 2013, a total

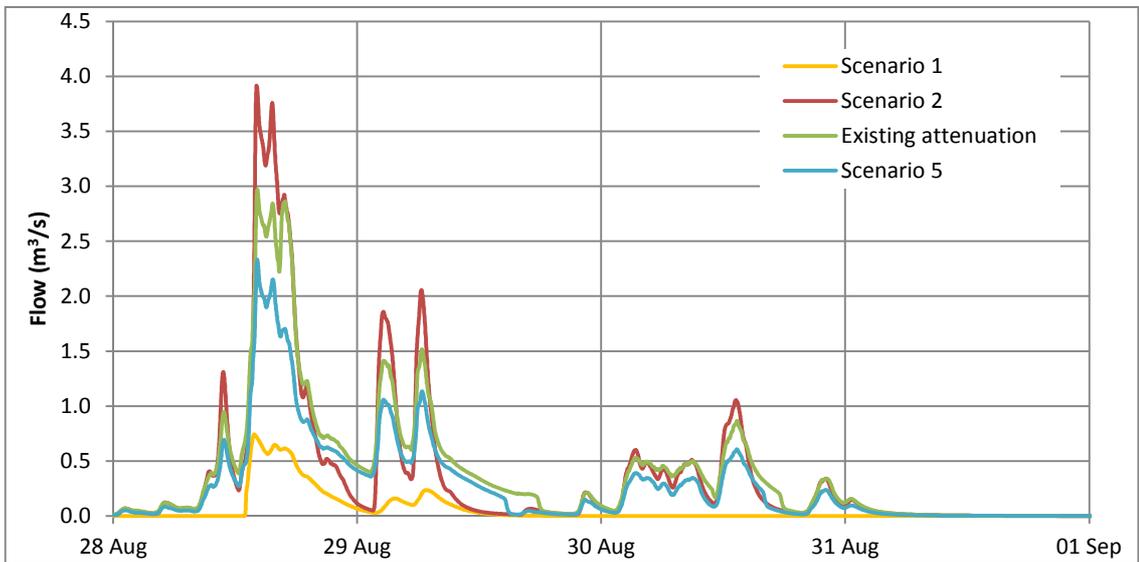
rainfall depth of 54.2 mm was recorded at the Tygerberg Reservoir, while during the same period, 41.8 mm was recorded at the Kraaifontein Roads Depot. This corresponds approximately with a 2- to 5-year recurrence interval rainfall event in the vicinity of the Mosselbank River Catchment.

The second rainfall event, shown in **Figures 3.59 to 3.61**, consisted of two moderate sized storms occurring over three days. At the Tygerberg Reservoir, the first storm recorded a total 24-hour rainfall depth of 31.6 mm, and the second, 29.2 mm, while at the Kraaifontein Roads Depot, 42.6mm and 30.6mm was recorded over a 24 hour period for the respective storm events. Both of these storms correspond approximately with 1- to 2-year recurrence interval rainfall events.

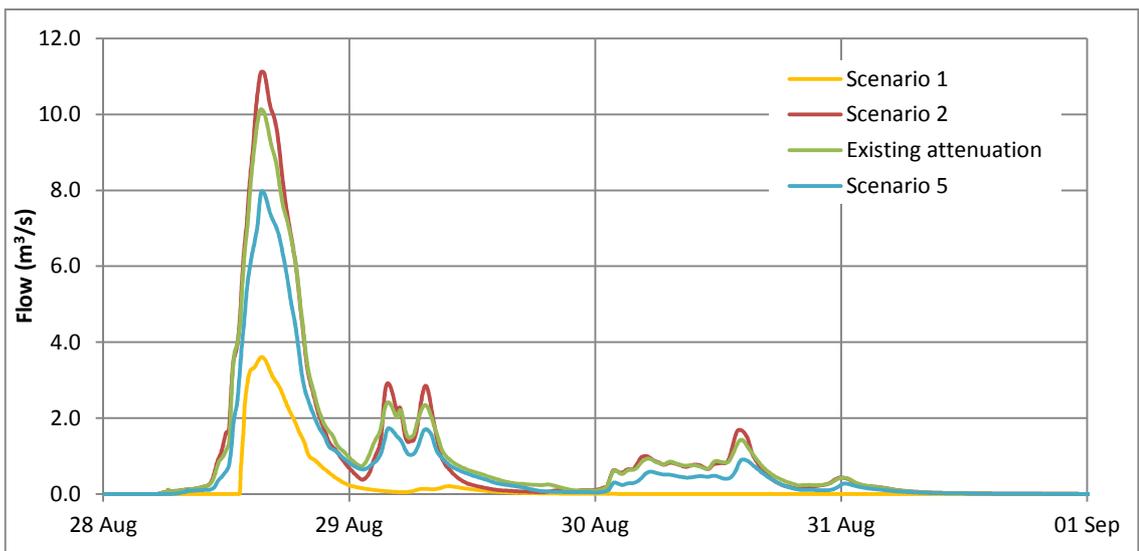
The third rainfall event, shown in **Figures 3.62 to 3.64**, represents a period of continuous light rainfall which would be typical of storm events with recurrence intervals of less than 1 year.



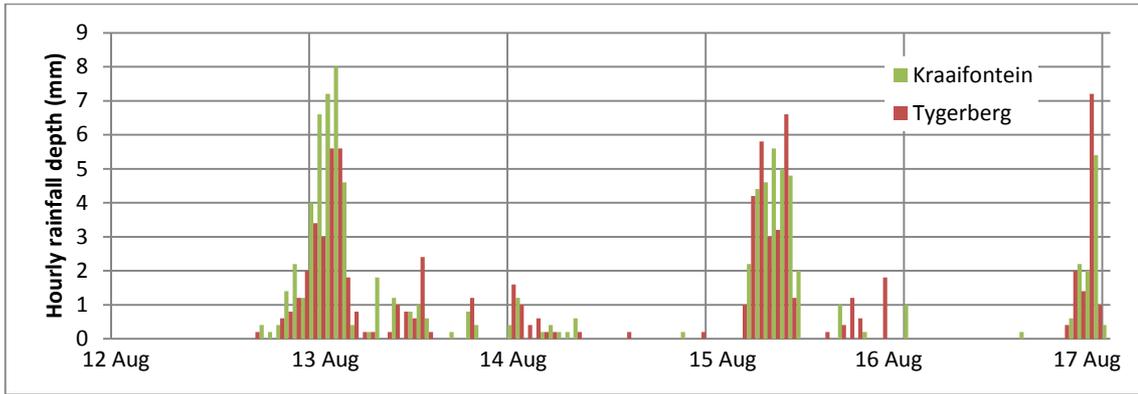
**Figure 3.56: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 28 and 31 August 2013**



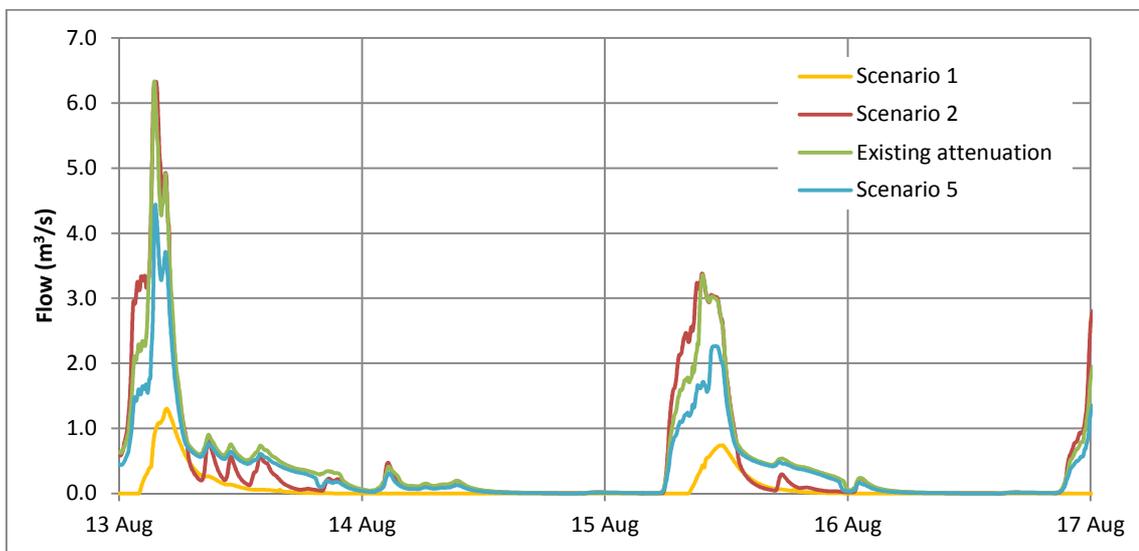
**Figure 3.57: Flow hydrograph at De Villiers Drive following heavy rainfall of 28 August 2013**



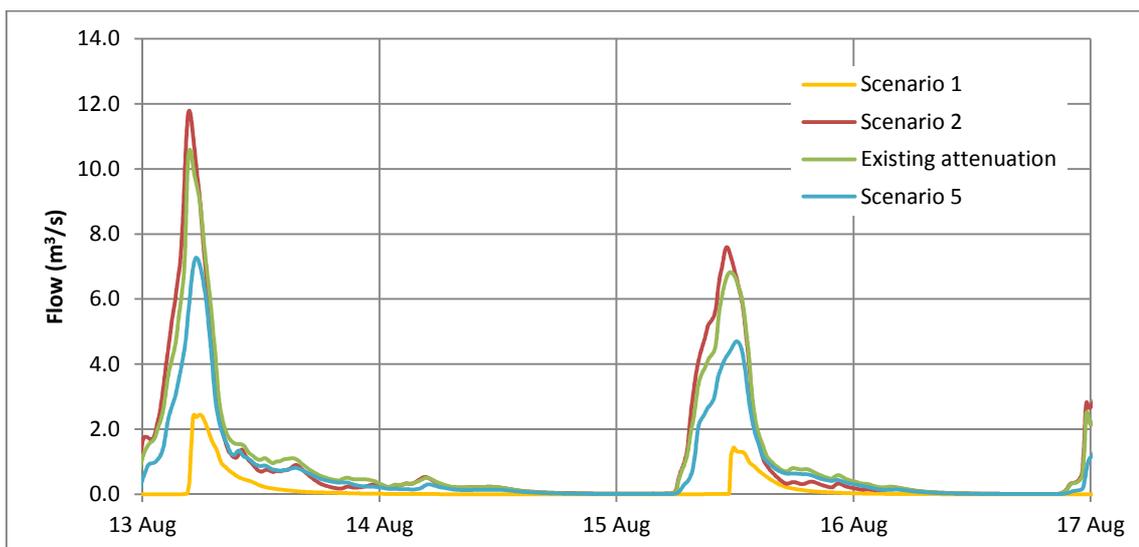
**Figure 3.58: Flow hydrograph at B-3 following heavy rainfall of 28 August 2013**



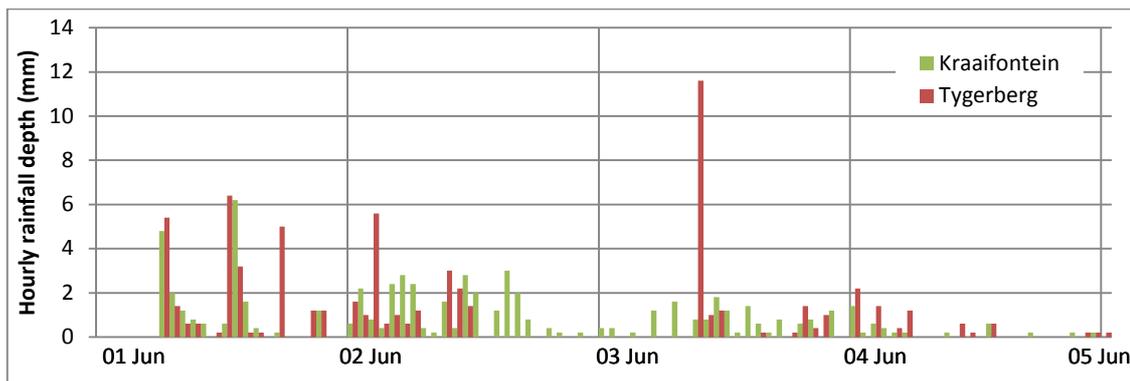
**Figure 3.59: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 12 and 16 August 2013**



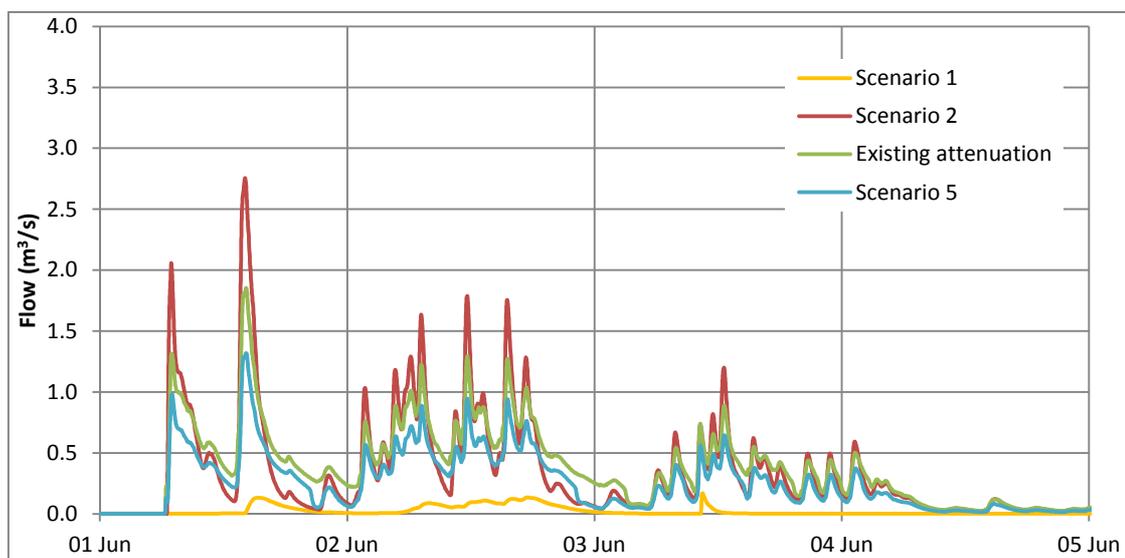
**Figure 3.60: Flow hydrograph at De Villiers Drive following rainfall between 13 and 16 August 2013**



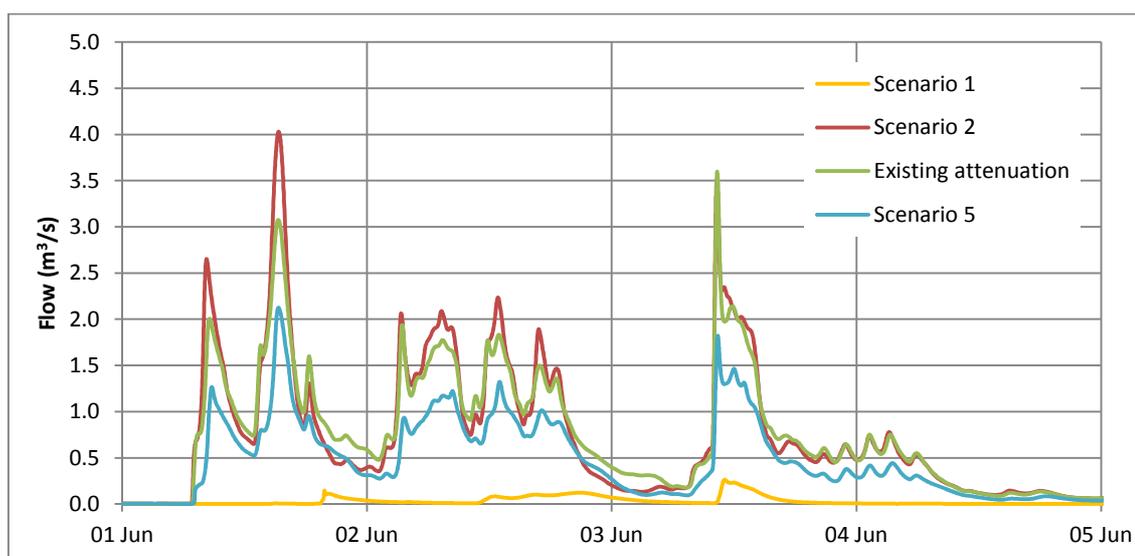
**Figure 3.61: Flow hydrograph at the N1 following rainfall between 13 and 16 August 2013**



**Figure 3.62: Hourly rainfall depths recorded at the Kraaifontein Roads Depot and Tygerberg Reservoir between 1 and 4 June 2013**



**Figure 3.63: Flow hydrograph at De Villiers Drive following rainfall between 1 and 4 June 2013**



**Figure 3.64: Flow hydrograph at the N1 following rainfall between 1 and 4 June 2013**

From **Figures 3.56 to 3.64**, the following observations are noted:

- The pre-development runoff results from the Upper Kuils River Catchment show that runoff is relatively low due to the high infiltration rate of a large portion of the catchment's ground cover.
- The Scenario 2 and "existing attenuation" modelling results show that the current urban development within the catchment has drastically changed the runoff rates and volumes resulting in significantly increased peak flows in the Upper Kuils River compared to the pre-development runoff rates.
- With the absence of attenuation facilities in many of the Upper Kuils River sub-catchments, providing additional BMPs within the Upper Kuils River Catchment could assist to some degree in reducing peak flows downstream.

### **3.12 Summary of the modelling scenario results**

#### **3.12.1 Scenario 1**

Scenario 1 modelled the catchment in natural or rural conditions prior to development. In this scenario runoff is dependent on the hydrological runoff potential of the natural land in terms of soil conditions, vegetation, and catchment slope.

In the case of the Bayside Canal Catchment, there was very little overland runoff due to the naturally low runoff potential of the catchment and large localised depressions. The models showed that small storm events will result in no overland runoff in these areas.

The results of pre-development runoff in the Mosselbank River and Upper Kuils River were higher per unit catchment area compared to the Bayside Canal Model due to the higher rainfall and higher runoff potential of the ground surface.

#### **3.12.2 Scenario 2**

Scenario 2 modelled catchment runoff from a future post-development land use scenario, however, it assumed that no attenuation measures were implemented as part of the stormwater network. This scenario took into account the significant increase in the areas now covered with impervious surfaces, such as roads, paving, and roofs.

The modelling showed a significant increase in runoff rates and volumes, especially in areas of high percent imperviousness. An assessment of the percentage increase in runoff peaks and runoff rates shows that there is a higher percent increase during small storm events and a lower percent increase during larger storms, as previously found by Konrad (2003) and Du et al (2012).

In the case of the Mosselbank River Model, it was found that the wide floodplains in the lower reaches of the Mosselbank River provided natural storage and attenuation during the higher flood peaks which limited the impact of post-development runoff to some degree.

### 3.12.3 Scenario 3

Scenario 3 modelled catchment runoff from a future post-development land use scenario. It assumed that attenuation facilities are implemented at sub-catchment (on-site or local) level and that these facilities are designed to attenuate the post-development peak flow of the 50-year recurrence interval design storm with a single culvert type outlet.

Many detention ponds in our cities today consist of a single culvert type outlet designed to reduce the 50-year post-development runoff peak down to the estimated 50-year pre-development peak. The volume of the pond is determined by calculating the difference between the flow volume coming into the pond over time, and the outlet flow volume over time. Since the ponds in this scenario are only designed to control the 50-year flood, they are much less effective, or even completely ineffective, during small to medium floods where the outlet culvert does not provide significant restriction of flow.

Even during large storm events, the modelling showed that this design approach is only effective for a limited distance downstream, before the cumulative effect of the increased runoff volume becomes evident. It was found in the case of the Mosselbank River Model that this approach, if applied widely over a large catchment area, may in fact increase peak flows higher than the post-development scenario with no attenuation due to timing effects. For example, when the runoff peak from downstream sub-catchments is lagged by the attenuation, it will in some situations coincide with the peak flow from the larger upstream catchments, resulting in higher peak flows downstream.

### 3.12.4 Scenario 4

Scenario 4 modelled catchment runoff from a future post-development land use scenario and assumed the implementation of attenuation facilities at sub-catchment level designed to attenuate multiple storm events (1-year, 10-year and 50-year) as per the CCT's *Management of Urban Stormwater Impacts Policy* (City of Cape Town, 2009b).

Many current stormwater manuals and policies recommend or specify that a range of design storm events, including large and small storms be controlled in such a way that post-development peak flow from a development site does not exceed the peak flow that would have resulted from the same site in pre-development conditions when subjected to the same design storm event. A typical detention pond which has been designed to attenuate multiple design storms would have a multi-stage outlet structure which would release flow from the pond at various rates depending on the volume of runoff and corresponding water

depth in the pond. This would therefore require a greater pond volume compared to a pond designed for a single design storm, as described in Scenario 3.

The results show that this approach achieves its objectives of attenuating a range of storm events at a sub-catchment level. However, if applied widely over a larger catchment area, the higher flow volumes converge resulting in flood peaks downstream which exceed pre-development levels.

### **3.12.5 Scenario 5**

Scenario 5 modelled catchment runoff from a future post-development land use scenario and assumed the implementation of attenuation facilities at sub-catchment level designed to attenuate multiple storm events, as in Scenario 4, however, it also assumed an increased use of BMPs and SUDs controls within the sub-catchments. The modelling results showed that by reducing the percent imperviousness of sub-catchments and providing more opportunities for runoff to infiltrate, there is quite a large reduction in runoff rates and volumes leading to reduced flow peaks in the main watercourses downstream.

## **4. Discussion of findings**

### **4.1 Findings from the Mosselbank River case study**

#### **4.1.1 Ineffective attenuation with single-stage outlet structures**

Attenuation facilities that make use of a single-stage outlet structure designed to control the 50-year recurrence interval storm event are not able to reduce smaller flood events to pre-development levels, although they do tend to assist in reducing flows to less than unattenuated post-development runoff levels. It is noted that these attenuation facilities do not fulfil internationally accepted design standards when it comes to sustainable management of drainage systems as they do not mitigate the majority of the impacts of post-development runoff.

Furthermore, in a stormwater system with multiple sub-catchments being controlled by attenuation facilities with single-stage outlet structures, the effect of the 50-year attenuation at sub-catchment level only extends for a limited distance downstream, as discussed in **Section 4.1.2**.

#### **4.1.2 Cumulative effect of increase in runoff volume**

The modelling results from the Mosselbank River case study showed the significant effects that extensive development within a catchment can have on stream flow regimes downstream even if attenuation is implemented on all sub-catchments throughout the total catchment area. The higher peak flows and resulting flooding along the main Mosselbank River channel was shown to be caused by the higher runoff volume from the catchment. Although the attenuation of runoff was effective in maintaining pre-development peak flows in the immediate vicinity of the attenuation facility and for a limited distance downstream, due to the cumulative effect of the increased runoff volumes, sub-catchment attenuation became less effective as the contributing catchment became larger. This confirms what was found in other literature sources referred to in **Section 2.8.5**.

#### **4.1.3 Changes to catchment dynamics caused by attenuation**

The modelling results from the Mosselbank River case study also illustrated the effect of changed catchment dynamics. The 2-year, 10-year and 50-year design storm analyses of Scenario 3 illustrated that widespread implementation of attenuation facilities with single-stage outlets may in fact result in higher peak flows along the lower reaches of the Mosselbank River compared to the scenario without any attenuation. This is considered to be due to the attenuated flow from the sub-catchments lower down in the catchment coinciding to a greater extent with the bulk of the flow from the upstream areas.

#### 4.1.4 Natural attenuation in the river floodplain

Although not explicitly modelled as part of this research, an observation made from the Mosselbank River case study was the contribution of the river floodplain to cause high flows to naturally pond in some areas which resulted in reduced flow peaks downstream. Where rivers and their natural floodplains are modified to allow for development, this natural attenuation would be removed from the system, resulting in changed flow regimes downstream.

#### 4.1.5 Poor results for small storm events

According to the CCT's *Management of Stormwater Impacts Policy* (City of Cape Town, 2009b), the 1-year recurrence interval 24-hour storm event should be detained and released over a 24-hour period (refer to **Section 2.6.3**). Although the Mosselbank River Model was based on this criterion, this approach was not shown to closely mimic natural pre-development flow patterns for the 2-year recurrence interval event. It may therefore be more appropriate to implement other on-site or local stormwater controls which can assist with reducing runoff volumes and attenuate flow to a greater extent during more frequent runoff events.

### 4.2 Findings from the Bayside Canal case study

#### 4.2.1 Value of constructed "natural" systems

The Bayside Canal case study showed that there are some areas in which it is completely impractical to maintain natural drainage patterns. As previously discussed in **Section 3.2.2**, the natural catchment area was not drained by defined watercourses. Instead, almost all rainfall would be drained to local depressions and infiltrate, even during large storm events. Therefore, the CCT and the developers of the upper catchment area chose to implement a constructed "natural" system of earth channels, ponds and wetlands which would most effectively suit the built environment (City of Cape Town, 2006).

The sections of the catchment which have been developed since 2002 clearly illustrate the amenity and biodiversity value of natural drainage systems implemented in an urban environment. In addition, the use of attenuation facilities in the form of ponds and wetlands throughout the system is shown to protect the downstream stormwater systems from flooding and erosion. It also ensures good water quality in these systems so that they are healthy for humans and animals using the green corridors for recreational purposes.

#### 4.2.2 Limited effect of increased runoff volumes

As discussed in **Section 4.1.1**, the modelling results from the Mosselbank River Catchment showed significant increases to the peak flows in the main watercourses downstream caused by the increase in runoff volume, despite runoff rates being attenuated to flows less than

pre-development levels at each individual sub-catchment. This effect was not found to occur in the Bayside Canal Catchment primarily because of scale differences. In the Mosselbank River Catchment, the effect of increased runoff volumes was only found to occur in watercourses once the catchment reached a size of over 30 km<sup>2</sup>. Since the catchments assessed with the Bayside Canal Model were significantly smaller than this, the increased runoff volume was not seen to impact on downstream flow peaks.

### **4.3 Findings from the Upper Kuils River case study**

#### **4.3.1 Limited effectiveness of current attenuation facilities**

The majority of the Upper Kuils River stormwater system has been implemented with the conventional approach which has generally made use of concrete pipes, box culverts and lined channels. Attenuation facilities are not very widely used across the catchment area, tending to be concentrated in those areas in the east of the catchment which have been developed more recently following the introduction of on-site attenuation requirements by the CCT.

The modelling results of the Upper Kuils River case study show that although the developments which have implemented on-site attenuation have assisted in reducing peak flow rates in the main Kuils River channel downstream, as a whole, the flow regime in the Upper Kuils River is still significantly changed from pre-development conditions. In addition to the modelling results, this change in flow patterns is evident from the erosion seen along many of the unlined sections of the Upper Kuils River.

#### **4.3.2 Addressing flooding and erosion with additional BMPs and SuDS controls**

The implementation of retrofit BMPs and SuDS controls within the Upper Kuils River would have a significant improvement on the flow regimes in the Kuils River downstream which would assist in reducing flooding and erosion along the watercourse. It is noted, however, that the implementation of BMPs and SuDS controls in existing built up areas may prove impractical in many areas due to space limitations. In addition, the cost of implementation needs to be weighed up against the benefits, considering that flooding along the Kuils River is not currently considered a significant risk to the CCT (City of Cape Town, 2013).

## 5. Conclusions

The purpose of this study was to critically evaluate the use of attenuation controls in South African urban areas and to investigate various technical concepts which should be taken into account when implementing attenuation as part of an urban stormwater network. This was achieved by reviewing both local and international literature as it relates to the implementation of attenuation controls, and by conducting three case studies which were investigated using the stormwater modelling software PCSWMM.

Attenuation controls have been in use in South Africa for many years. In the past they have typically only been implemented in an ad-hoc manner in order to address downstream flooding during large storm events, however, in the last 20 years, South African local authorities have begun enforcing regulations which require all new developments above a certain size to implement on-site attenuation which addresses multiple design storm events. The primary objectives of this approach are to mitigate the increasing flooding and erosion problems being experienced adjacent to urban watercourses, as well as to address stormwater quality concerns to some degree. The literature review, however, found that there is little practical guidance in South Africa on the appropriate design procedure to follow when implementing these attenuation facilities and often the operational performance of these facilities, in terms of mimicking natural flow regimes, is unverified.

Furthermore, previous studies have shown that there are secondary impacts resulting from the implementation of multiple attenuation facilities within a larger catchment area. This is because attenuation does not address the increased volume of urban runoff, but merely reduces runoff peaks. The lagged runoff peaks from multiple sub-catchments coincide downstream resulting in higher peak flows. In addition, attenuation can play a role in worsening flooding downstream by changing the timing of peak flow conveyance from a number of sub-catchments.

Internationally, there is a lot of relevant literature available, including useful design information found in various design manuals, policies and regulations, which addresses most of the technical aspects regarding the correct use of attenuation controls within urban areas. Some of the typical aspects of attenuation design highlighted in the literature review include the sizing and placement of attenuation facilities, the design of multi-stage outlet structures for the control of multiple storm events, and the assessment of catchment-wide impacts of attenuation practices within an urban catchment.

Three case studies were undertaken as part of this study in order to evaluate some of the above-mentioned technical concepts highlighted in the literature review. The study areas included:

- The Mosselbank River Catchment – north of Durbanville/Kraaifontein
- The Bayside Canal Catchment – Blaauwberg area
- The Upper Kuils River Catchment – Durbanville/Kraaifontein

The stormwater modelling software PCSWMM, developed by CHI, was used to model the above-mentioned case studies for the following scenarios:

- Scenario 1: Pre-development.
- Scenario 2: Post-development flow if no attenuation is implemented.
- Scenario 3: Attenuation implemented at sub-catchment level designed to attenuate the post-development peak flow of the 50-year design storm with a single culvert type outlet.
- Scenario 4: Attenuation implemented at sub-catchment level designed to attenuate multiple storm events (1-year, 10-year and 50-year) as per the CCT's *Management of Urban Stormwater Impacts Policy* (City of Cape Town, 2009b) (refer to **Section 2.6.3**).
- Scenario 5: Attenuation implemented as in Scenario 4 in conjunction with increased use of stormwater BMPs and SuDS controls within the sub-catchments.

Besides the pre-development scenario (Scenario 1), all of the other scenarios assumed full development of the modelled catchments based on the future development planning information contained within the CCT's study entitled *Investigation into the Medium to Long Term Growth Options for Cape Town*. For comparative purposes, the catchment and watercourse modelling parameters of all of the post-development scenarios (i.e. Scenarios 2 to 5) were kept consistent for the respective simulations and only the attenuation control parameters were varied. One exception to this statement is that the percent imperviousness and routing to pervious areas was adjusted for Scenario 5 to account for the increased use of BMPs and SuDS controls.

The key findings of the case studies are summarised as follows:

- Attenuation controls which make use of a single-stage outlet structure are only effective in controlling the recurrence interval storm for which it was designed. The modelling results showed that attenuation facilities designed for the 50-year recurrence interval storm event are not effective in attenuating lower order storm events to pre-development levels. It is noted that these attenuation facilities do not fulfil internationally accepted design standards when it comes to sustainable management of drainage systems as they do not mitigate the impacts of post-development runoff for the majority of rainfall events.

- Attenuation facilities with multi-stage outlet structures are much more effective at mimicking pre-development flow during a range of storm events compared to those with single-stage outlet structures. However, there is often a limit to how effective these facilities are for storms with a recurrence interval of less than two years.
- It was also found that because attenuation does not reduce post-development runoff volumes to pre-development levels, but merely reduces peak flow rates, the cumulative runoff from multiple attenuation controls across a large (>30 km<sup>2</sup>) urban catchment resulted in higher runoff peaks in downstream watercourses.
- The lagged hydrograph peak of attenuated flow from a sub-catchment can result in unexpected peak flows downstream due to the changed timing effects within the wider catchment area.
- The natural attenuation of an undeveloped floodplain can assist in reducing peak flows downstream.
- The results showed that many of the secondary impacts resulting from the use of multiple attenuation facilities are due to higher post-development runoff volumes. This can be addressed to some extent by implementing stormwater BMPs and SuDS controls which assist in providing more opportunity for runoff to infiltrate.

In addition to the above key findings, the case studies illustrated the value of detailed catchment-wide stormwater modelling to understand the catchment dynamics holistically. The need for short time-step rainfall and streamflow data was also highlighted as a key input in accurately simulating real rainfall-runoff events and evaluating stormwater management practices.

## **6. Recommendations for further research**

### **6.1 Case studies in other regions of South Africa**

The case studies presented in this thesis specifically evaluated various attenuation scenarios in the vicinity of Cape Town, which typically receives low intensity frontal rainfall during South Africa's winter season. It is therefore recommended that further case studies be undertaken in urban catchments located in other regions of South Africa where rainfall conditions are different to those experienced in Cape Town. For example, Johannesburg and the surrounding Highveld Region typically experience short duration, high intensity rainfall during the summer months; the north-eastern regions of South Africa are exposed to tropical cyclones; and further south, KwaZulu-Natal and the Eastern Cape have experienced wide-spread flooding due to cut-off low pressure systems. Varying climatic conditions will have an impact on the typical timing of runoff and the associated runoff peaks and volumes, and therefore may provide differing results from those reported on in this study.

### **6.2 Modelling parameters and calibration data**

The compilation of the Mosselbank River, Bayside Canal and Upper Kuils River models revealed a lack of standardised procedures for modelling and a lack of verified data which can be used as input in the models. There is therefore a lot of research required in terms of determining appropriate modelling parameters applicable to specific areas based on specialist studies.

Furthermore, stormwater practitioners and authorities should realise the value, and research potential, associated with rainfall and stream flow logging data. Not only can this data be used for model calibration purposes, but it can shed light on the continuously changing nature of the catchment as urban development expands or becomes denser. Furthermore, there is uncertainty regarding the effect that climate change may have on rainfall distribution, and hence, on catchment flow regimes, which can be monitored with logging data. This is especially important in areas identified for future development, such as the Mosselbank River Catchment, so that stormwater attenuation controls, which are implemented with the development, can be assessed in terms of their effectiveness, based on real flow data from before and after development.

### **6.3 Research of other SuDS techniques**

This study focussed specifically on one SuDS technique, attenuation of post-development storm runoff. Therefore, following from this, there is potential research to be undertaken in assessing the broader catchment impacts of other SuDS techniques, such as runoff volume reduction within urban catchments. Currently, there is still significant scope for more detailed

research in terms of the implementation of SuDS as part of urban stormwater management in South Africa. *The South African Guidelines for Sustainable Drainage Systems* (Armitage et al., 2013a) provides a useful overview of the SuDS approach and many of the SuDS controls, however, it is recommended that further research and case studies be undertaken which clearly indicate the advantages and disadvantages of each of these various SuDS controls, and where they are most applicable for implementation within an urban catchment.

#### **6.4 Urban drainage guidelines for South Africa**

It is noted that many of the findings of this study which address broad catchment management issues are not addressed in any readily available South African literature. Therefore, there is scope for the development of more detailed urban drainage guidelines, or a manual, which can better inform stormwater management policies in South Africa. The stormwater design guidelines provided in the *Red Book* (CSIR, 2000) are still used by many design engineers and authorities in South Africa, however, it is argued that these guidelines are now outdated in many aspects. In addition, The recently published *South African Guidelines for Sustainable Drainage Systems* (Armitage et al., 2013a) focuses primarily on individual SuDS techniques and does not address some of the broader catchment management issues which have been highlighted in this study. It is therefore recommended that further research be undertaken for the compilation of revised stormwater design guidelines for South Africa which attempt to address all aspect of stormwater management in South Africa.

#### **6.5 Economic implications of various flood attenuation controls**

An economic assessment of the various flood attenuation options evaluated in this thesis was beyond the scope of this study, and it is therefore recommended that further research be undertaken to evaluate the cost implications associated with more wide-spread use of attenuation facilities which are designed to attenuate multiple storm events.

## References

- ANZECC and ARMCANZ, 2000. National Water Quality Management Strategy - Australian Guidelines for Urban Stormwater. Commissioned by the Australian and New Zealand Environment and Conservation Council (ANZECC) and the Agriculture and Resource Management Council of Australia and New Zealand (ARMCANZ).
- Armitage, N., Vice, M., Fisher-Jeffes, L., Winter, K., Spiegel, A., 2013a. The South African Guidelines for Sustainable Drainage Systems. Water Research Commission. Report No. TT 558/12.
- Armitage, N., Vice, M., Fisher-jeffes, L., Winter, K., Spiegel, A., Dunstan, J., 2013b. Alternative Technology for Stormwater Management - Sustainable Drainage Systems – Report and South African Case Studies. Water Research Commission. Report No. 1826/1/13.
- Booth, D., Jackson, C., 1997. Urbanization of Aquatic Systems: Degradation Thresholds, Stormwater Detection, and the Limits of Mitigation. *Journal of the American Water Resources Association* 33, 1077–1090.
- Botha, N., 2005. Stormwater Management and Related Urban Environmental Issues along the Fourways Spruit.
- Brown, C., Magoba, R., 2009. Rivers and Wetlands of Cape Town - Caring for our rich aquatic heritage. Water Research Commission. Report No. TT376/08.
- Buyts, F., Aldous, M., 2009. The perceived economic impact of the City of Johannesburg's storm water attenuation policy on private property developers. *Acta Structilia* 16, 18–45.
- City of Cape Town, 2001. Mosselbank & Klipmuts Rivers: Determination of Floodlines. Compiled by Africon Consulting Engineers.
- City of Cape Town, 2002. Catchment, Stormwater and River Management Strategy 2002 - 2007.
- City of Cape Town, 2005. By-Law Relating to Stormwater Management.
- City of Cape Town, 2006. Stormwater Master Plan for Blaauwberg Development Area. Compiled by CIVtech Consultants cc.
- City of Cape Town, 2009a. Floodplain and River Corridor Management Policy. Cape Town.
- City of Cape Town, 2009b. Management of Urban Stormwater Impacts Policy. Cape Town.
- City of Cape Town, 2009c. Durbanville North Stormwater Masterplan and Floodline Determination. Compiled by Bergstan Consulting Engineers.
- City of Cape Town, 2011. Report to Executive Mayor MC 33/05/11 - Application for amendment of the interim urban edge.

- City of Cape Town, 2012a. Investigation into the Medium to Long Term Growth Options for Cape Town - Bulk Infrastructure Review. Compiled by BKS (Pty) Ltd.
- City of Cape Town, 2012b. Stormwater and River Management Stormwater Asset Management Plan (Phase 2B): High Level Master Planning for the Eastern Catchments - Aquifer Infiltration Potential and Contamination Risk Assessment. Compiled by Umvoto (Pty) Ltd on behalf of Arcus Gibb (Pty) Ltd for the City of Cape Town; Directorate: Roads and Stormwater; Branch: Catchment, Stormwater and River Management.
- City of Cape Town, 2012c. Stormwater Infrastructure Asset Management Plan (Phase 2A) Rainfall Analysis and High Level Masterplanning: Salt River Catchment: Final Report: High Level Masterplanning. Report compiled by SRK Consulting for the City of Cape Town.
- City of Cape Town, 2013. High Level Master Planning for the Eastern Catchments Volume B : Kuils / Eerste River High Level Stormwater Masterplan. Compiled by GIBB Engineering & Science as part of the Stormwater Asset Management Plan (Phase 2B), in association with BKS (Pty) Ltd, Jeffares & Green, and Giscoe.
- City of Johannesburg, 2003. State of the Environment Report. Department of Development Planning, Transportation and Environment, Environmental Planning and Management Unit.
- City of Johannesburg, 2009. Draft By-Laws Relating to Stormwater Management.
- Council of Australian Governments, 2005. Intergovernmental Agreement on a National Water Initiative 1–39.
- CSIR, 2000. Guidelines for Human Settlement Planning and Design (The Red Book). Council for Scientific and Industrial Research, Pretoria.
- DEFRA, 2010. Surface Water Management Plan Technical Guidance. Department for Environment, Food and Rural Affairs (DEFRA), London.
- DEFRA, 2011a. National Standards for sustainable drainage systems - Designing, constructing, operating and maintaining drainage for surface runoff. Department for Environment, Food and Rural Affairs (DEFRA), London.
- DEFRA, 2011b. Guidance for risk management authorities on sustainable development in relation to their flood and coastal erosion risk management functions. Department for Environment, Food and Rural Affairs (DEFRA), London.
- DEFRA; Environment Agency, 2011. Understanding the risks, empowering communities, building resilience: the national flood and coastal erosion risk management strategy for England. Presented to Parliament pursuant to Section 7 of the Flood and Water Management Act 2010.
- Du, J., Qian, L., Rui, H., Zuo, T., Zheng, D., Xu, Y., Xu, C.-Y., 2012. Assessing the effects of urbanization on annual runoff and flood events using an integrated hydrological modeling system for Qinhuai River basin, China. *Journal of Hydrology* Volumes 46, Pages 127 – 139.

- Ellis, J., D'Arcy, B., Chatfield, P., 2002. Sustainable Urban- Drainage Systems and Catchment Planning. *Water and Environment Journal* 16.
- Environment Agency, 2008. Sediment transport & alluvial resistance in rivers. Joint Defra / Environment Agency Flood and Coastal Erosion Risk Management R&D Programme.
- eThekweni Municipality, 2006. State of the Rivers Report. Ground Truth Biomonitoring Services and Environmental Consultants.
- eThekweni Municipality, 2008. Design Manual: Guidelines and Policy for the Design of Stormwater Drainage and Stormwater Management Systems. Engineering Unit - Coastal Stormwater and Catchment Management Department, Durban.
- European Commission, 2011. Towards Better Environmental Options for Flood Risk Management. European Commission, Directorate-General Environment, Brussels.
- European Parliament, 2007. Directive 2007/60/EC of the European Parliament and of the Council. *Official Journal of the European Union* 27–34.
- Faulkner, B., 1999. The control of surface water runoff from new development—UK national “policy” in need of review? *Urban Water* 1, 207–215.
- Fennessey, L., Hamlett, J., Aron, G., LaSota, D., 2001. Changes in runoff due to stormwater management pond regulations. *Journal of Hydrologic Engineering* 6, 317–327.
- Gericke, O., du Plessis, J., 2013. Development of a customised design flood estimation tool to estimate floods in gauged and ungauged catchments. *Water SA* 39, 67–94.
- Giricke, O., du Plessis, J., 2012. Catchment parameter analysis in flood hydrology using GIS applications. *Journal of the South African Institution of Civil Engineering* 54, 15–26.
- Goff, K., Gentry, R., 2006. The influence of watershed and development characteristics on the cumulative impacts of stormwater detention ponds. *Water resources management* 20, 829–860.
- Government of Western Australia, 2004. Stormwater Management Manual for Western Australia. Department of Water, Government of Western Australia, Perth.
- Government of Western Australia, 2009. Decision process for stormwater management in WA. Department of Water, Government of Western Australia, Perth.
- Hardt, R., Burges, S., 1976. Some consequences of area wide runoff control strategies in urban watersheds - *Water Resources Series Technical Report No. 48*. University of Washington, Department of Civil and Environmental Engineering, Seattle.
- Haskins, C.A., 2012. Cape Town’s sustainable approach to urban stormwater management. Catchment, Stormwater and River Management Branch, City of Cape Town.

- Haubner, S., Reese, A., Brown, T., Claytor, R., Debo, T., 2001. Georgia Stormwater Management Manual. Compiled by AMEC Earth and Environmental, Center for Watershed Protection, Debo and Associates, Jordan Jones and Goulding, and the Atlanta Regional Commission.
- Horton, R., 1940. An Approach Towards a Physical Interpretation of Infiltration Capacity. *Soil Science of America* 5, 399–417.
- James, W., Rossman, L., James, W., 2010. User's Guide to SWMM5 13th Edition. CHI, Guelph, Ontario, Canada.
- Knox County, 2008. Knox County, Tennessee - Stormwater Management Manual. Compiled by AMEC Earth & Environmental Inc, Knoxville.
- Konrad, C.P., 2003. Effects of Urban Development on Floods. US Geological Surveys FS-076-03, 1–4.
- Lee, J.G., Selvakumar, A., Alvi, K., Riverson, J., Zhen, J.X., Shoemaker, L., Lai, F., 2012. A watershed-scale design optimization model for stormwater best management practices. *Environmental Modelling & Software* 37, 6–18.
- Lockie, T., 2006. Catchment Modelling Using SWMM. Hydraulic Analysis Limited.
- Matsuda, I., 2004. River Morphology and Channel Processes. In: *Fresh Surface Water. Encyclopedia of Life Support Systems (EOLSS)*, Developed under the Auspices of the UNESCO, Eolss Publishers, Oxford ,UK.
- Melbourne Water, 2012. Draft Stormwater Strategy. Melbourne.
- Monk, E., Chalmers, L., 2006. Mimic natural drainage processes - A practical approach for stormwater management in Western Australia. In: 1st National Hydropolis Conference 2006 Burswood Convention Centre Perth, Western Australia. pp. 1–16.
- Monk, E., Till, B., 2006. Quantity is the road to quality. In: 2006 Public Works Engineering State Conference (8-10 March 2006).
- Pattison, I., Lane, S.N., Hardy, R.J., Reaney, S., 2008. Sub-catchment peak flow magnitude and timing effects on downstream flood risk. BHS 10th National Hydrology Symposium 282–288.
- Petrucci, G., Deroubaix, J., Tassin, B., 2011. Stormwater source control regulation : a hydrological comparison of alternative policies. In: 12th International Conference on Urban Drainage, Porto Alegre, Brazil. pp. 1–8.
- Queensland Government, 2007. Queensland Urban Drainage Manual.
- Republic of South Africa, 1996. Constitution of the Republic of South Africa. Act No. 108 of 1996 230.
- Republic of South Africa, 1998a. National Water Act. Act No. 36 of 1998.
- Republic of South Africa, 1998b. National Environmental Management Act. Act No. 97 of 1998.

River Health Programme, 2005. State of Rivers Report - Greater Cape Town's Rivers. Department of Water Affairs and Forestry, Pretoria.

SANRAL, 2013. Drainage Manual (6th Edition). South African National Roads Agency Limited.

Schulze, R., Knoesen, D., Kunz, R., van Niekerk, L., 2010. Impacts of Projected Climate Change on Design Rainfall and Streamflows in the Cape Town Metro Area. ACRUcons Report 63. August 2010. Report compiled for SRK Consulting as part of the Stormwater Infrastructure Asset Management Plan (Phase 2A) project commissioned by the City of Cape Town.

Tarlock, A.D., 2012. United States Flood Control Policy : The Incomplete Transition from the Illusion of Total Protection to Risk Management Adaptation. Duke Environmental Law & Policy Forum 23, 151–183.

United Kingdom, 2010. Flood and Water Management Act 2010.

United States Environmental Protection Agency, 1999. Preliminary Data Summary of Urban Storm Water Best Management Practices. United States Environmental Protection Agency.

United States Environmental Protection Agency, 2009a. Technical Guidance on Implementing the Stormwater Runoff Requirements for Federal Projects under Section 438 of the Energy Independence and Security Act.

United States Environmental Protection Agency, 2009b. STORM WATER MANAGEMENT MODEL USER ' S MANUAL.

Urbonas, B., Glidden, M., 1983. Potential effectiveness of detention policies. In: Southwest Storm Drainage Symposium, Texas A&M. pp. 1–18.

Urbonas, B., Wre, D., 2007. Stormwater Runoff Modeling; Is it as Accurate as We Think? In: Engineering Conferences International Conference on Urban Runoff Modeling: Intelligent Modeling to Improve Stormwater Management, Humboldt State Univ., Arcata, CA, July. 2007. pp. 1–12.

USACE, 2010. HEC-RAS River Analysis System – Hydraulic Reference Manual (Version 4.1). United States Army Corps of Engineers.

Van der Spuy, D., Rademeyer, P., 2014. Flood frequency estimation methods as applied in the Department of Water Affairs. Proceedings from the 2014 Flood Hydrology Course at Stellenbosch University.

Wang, X., Jin, J., 2001. Assessing the Impact of Urban Growth on Flooding with an Integrated Curve Number-Flow Accumulation Approach. Water International 26, 215–222.

Water Research Commission, 2005. Water Resources of South Africa 2005.

Whitlow, R., Brooker, C., 1995. The historical context of urban hydrology in Johannesburg. SAICE Journal 37 (4).

Wong, T.H.F., 2007. Water Sensitive Urban Design – the Journey Thus Far. Environment Design Guide.

Woods-Ballard, B., Kellagher, R., Martin, P., Jefferies, C., Bray, R., Shaffer, P., 2007. The SUDS Manual. CIRIA, London.

Aerial photographs were sourced from the City of Cape Town unless otherwise stated on the figure.