Influence of Catchment Development on Peak Urban Runoff

Report to the
Water Research Commission

by

Prof. Stefanus Johannes van Vuuren
Department of Civil Engineering
University of Pretoria

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Water Research Commission
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Executive Summary

The research reviewed catchment response due to urban development on the basis of comparative assessment. This required the identification of similar rainfall in the catchment during different development stages for which gauged flow rates were recorded. The hypothesis which was reviewed here relates to the statement that urban development which creates more impervious areas on the one hand also generated longer times of concentration due to the changes in the length of the flow path as well as more temporal storage capacity which could result in a higher groundwater recharge.

Three different catchments were evaluated:

> Catchment 1 (Willowspruit) in the Tshwane Metropolitan Council Area where a flow gauging structure was installed and autographic rainfall stations were placed to obtain a representative distribution and intensity of rainfall. Flow and rainfall was recorded since 1992 when the catchment was fairly undeveloped. During this research the flow gauging was undertaken for the current developed levels in the catchment.
>
> Catchment 2 (Robert's Place) is a highly developed urban security complex. Flow gauging weirs for two defined areas of the catchment as well as an autographic rainfall recorder was installed on this property. The recorded data was compared with the results obtained from a detailed modelling of the catchments.
>
> Catchments 3 (Rietvlei Dam, Daspoort gauging structure and Kameeldrift catchment) were selected to conduct a comparative discharge evaluation for similar rainfall seasons for different levels of catchment development.

Catchment 1 (Willowspruit) was fairly undeveloped for the period for which the discharge was recorded (October 1993 to May 1995) and it was believed when the research stated that autographic rainfall data were also available at three autographic rainfall stations. It was later established that this valuable data set was apparently lost. Rainfall data from two rainfall stations, Irene WO and Pretoria Eendracht, which had hourly recorded rainfall data, was then used to calculate the rainfall intensities. The rainfall intensity was also calculated by reviewing the time periods in which the discharge rate through the gauging station increased.

Catchment 2 was a densely developed smaller catchment. An autographic rainfall recorder as well as a flow gauging structure was installed for both zones in Robert's Place. A comparison was conducted between the modelled discharge, from a detailed EPA SWMM that was set up for this catchment, and the recorded gauged discharge.

The assessment of the Willowspruit catchment revealed that the discharge flow rates are less than the calculated values, reflecting the lack in uniform rainfall distribution, storm duration and influence of retaining structures (culverts that are fully or partially blocked). Although "similar storms" were selected for comparisons between the discharges during predevelopment versus post development it was realised that even for small catchments the variation in storm events are significant and it was impossible from the limited data to discard the hypotheses. It was concluded that both the temporal rainfall distribution and the influence of antecedent conditions are important when discharge calculating techniques are applied.

The assessment of Robert's Place reflected the calculated discharge was at times larger and at times smaller than the gauged discharge. Based on the comparison of calculated discharge (Peak flow rate and volume of discharge) with the recorded data it was impossible to derive any conclusive findings, but to indicate that the modelled results tend to be higher than the recorded runoff data.

In the case of the large catchments, a comparison was set between the cumulative rainfall and the cumulative runoff produced by similar rainfall events for different development levels in the catchment. This analysis compared years of similar volumetric rainfall, antecedent conditions and temporal distributions. There was a general trend indicating an increase in the percentage runoff produced as urban development increased, but certain anomalies were observed.

The hypothesis that the influence of urban catchment development will decrease the peak runoff has neither been proved nor disproved.

Consideration should be given to conduct further research in this field. It is therefore recommended that further investigation be done for both developed and undeveloped catchments to quantify a full understanding of the influence of drainage structures and different types of catchment development on the stormwater response from urban catchments.

Acknowledgements

The research presented in this report emanated from a study funded by the Water Research Commission (WRC) and conducted by the University of Pretoria and supported by the Tshwane Metropolitan Council.

The Reference group made important contributions and provides direction and support to the Project team for this project.

The patience, support and guidance of the Chairman and Manager of this study, Mr Wandile Nomquphu, as well as the supporting staff of the WRC and Tshwane Metropolitan Council is greatly appreciated.

The Reference Group and Project Team responsible for this study consisted of the following persons:

Mr Wandile Nomquphu	:	Water Research Commission – Chairman
Prof S J van Vuuren	:	University of Pretoria – Main researcher
Mr P Odendaal	:	Tshwane Metropolitan Council
Mr T Coleman	:	Consultant
Mr V Jonker	:	Consultant
Mr B Wood	:	Consultant
D Mahlangu / alt M Braune	:	Consultant
Prof N Armitage	:	University of Cape Town
Mr C Langhout	:	Consultant
Mr G Tooley	:	eThekwini Metropolitan Council

The contribution by a number of students from the University of Pretoria, who assisted in the compiling of research findings, is appreciated. They are: Mr H Peens, Mr T Nakana, Mr G Christie, Mr N Dlamini, Mr M Moolman and Ms L Rorke.

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1 Introduction

1.1 Background

Development of urban areas is limited by regulations pertaining potential flooding. (National Water Act – Act 36 of 1998). The 1:100 year flood line is used to define the extent of the development. Furthermore it is required by the Authorities that all developers should create temporal storage if the development contributes to the increase of the flood peaks. The rational is that new development should not affect the downstream ground users; neither should it compromise the hydraulic capacity of existing water conveying structures.

In a preliminary investigation that was conducted of three similar rainstorm events in the incremental catchments of the Crocodile River (A2H51), for different development stages reflected that:

- The flow peak reduces,
- The time of concentration enlarged and
- The total volume of runoff increased.

If the focus is to control the increase in the flow rate, it is essential that the influence of development on the catchments response should be reviewed.

The layout and features of South African urban development in the high and middle income groups, is influenced by the objective to prevent crime leading to the erection of solid boundary walls encircle most developments, leading to increased temporal storage, reducing the peak discharge rates.

The objective of this research was to conduct comparative analyses of “similar storm events for different development levels” with the aim to obtain a relationship between peak discharge and level of development.

1.2 Research methodology

A literature study was undertaken to identify the parameters normally used to determine the peak discharge from a catchment. Hydrological data was evaluated to identify "similar/compatible" storm events from which the influence of catchment development on the response from the catchment could be determined.

Urban areas with development history and recorded hydrological data (rainfall and runoff information) were identified for the review. Historical and current development levels of the catchments were determined.

The following catchments were reviewed:

> Catchment 1 (Willowspruit) in the Tshwane Metropolitan Council Area where a flow gauging structure was installed and autographic rainfall stations were placed to obtain a representative distribution and intensity of rainfall. Flow and rainfall was recorded since 1992 when the catchment was fairly undeveloped. During this research the flow gauging was undertaken for the current developed levels in the catchment.
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> Catchment 2 (Robert's Place) is a highly developed urban security complex. Flow gauging weirs for two defined areas of the catchment as well as an autographic rainfall recorder was installed on this property. The recorded data was compared with the results obtained from a detailed modelling of the catchments.
>
> Catchments 3 (Rietvlei Dam, Daspoort gauging structure and Kameeldrift catchment) were selected to conduct a comparative discharge evaluation for similar rainfall seasons for different levels of catchment development.

1.3 Research objectives

The objective of this study was to review historical hydrological data, identifying similar storm events and runoff response from the catchments for different development levels.

Table 1-1 reflects details of the deliverables which were identified for this project.

Table 1-1: Details of the deliverables which were identified for this project

No.	Deliverable Title	Description
1	Literature review	Investigation into current methods and available systems (writing of draft literature report)
2	Catchment identification and characterization	Identification of development area(catchments) and characterization
3	Rainfall/runoff modelling and verification	Verification of available rainfall and runoff data
4	Data collection methodology	Review of data acquisition systems for rainfall and runoff
5 & 6	Setting-up of measuring stations	Installation of measuring stations
7	Detailed description of development influences	Identification of development layouts that influences the catchment response
8	Rainfall/runoff modelling and verification	Verification of available rainfall and runoff data (and comparisons with others)
9	Comparison of design criteria	Comparison\Verification of the design criteria
10-12	Report on Hydrological years	First hydrological year
		Second hydrological year
		Part of third hydrological year
13	Draft report	Compilation of draft report
14	Final report	Completed final report

1.4 Layout of the Report

In this section (**Section 1**) the background, research methodology and research objectives were discussed. The following sections will focus attention on the aspects listed below:

- Review of applicable literature (**Section 2**);
- Field work conducted to determine the change in peak discharge due to catchment development for similar storm events (**Section 3**);
- Review of Willowspruit catchment peak runoff (**Section 4**);
- Review of Robert's Place peak runoff (**Section 5**);
- Assessment of the hydrological records to establish the influence of development on the runoff response (**Section 6**);
- Conclusions (**Section 7**);
- Recommendation for further research (**Section 8**); and
- References (**Section 9**).

The following Appendixes are included on the CD at the back of the report.

List of Appendices on the enclosed CD

Appendix A contains additional literature of studies that have been conducted on the effect of urban development on runoff response

Appendix B provides details of hydraulic structures in the Willowspruit catchment.

Appendix C contains the computer coding used in **Section 3**.

Appendix D provides rainfall data at Ramkie oor Willows, Irene WO and Pretoria Eendracht.

Appendix E provides rainfall data at Robert's Place.

Appendix F reflects the years that were selected in **Section 6** for the analysis of cumulative hydrological data, based on regional rainfall.

Appendix G gives an indication of which rainfall stations were used to determine the weighted average monthly rainfall on the contributing catchments upstream from the flow gauging stations at Rietvlei Dam, Daspoort and Kameeldrift.

Appendix H provides the return flow data for Rietvlei and Daspoort catchments.

2 Literature review

2.1 Introduction

In the literature review the methods that are normally used are firstly discussed with the objective to reflect all the parameters that could influence the peak discharge. This is followed by some relevant studies that were conducted to determine the influence of catchment development on peak discharge. Reference is also made to the modelling of stream events.

2.2 Calculation methods to evaluate the peak flow rate

There are different hydrological calculation methods in use (Sanral, 2007). The proven and most used methods in Southern Africa are:

- Statistical methods
- Rational method
- Alternative Rational method
- Unit Hydrograph method
- Standard Design Flood (SDF) method
- Empirical method

It is good practice in the determination of design floods to use more than one of the above methods, and if historical run-off data is available it should be analysed as well. It is always required to use the SDF method and compare with the other applicable methods. Where large discrepancies occur, an assessment should be conducted to motivate the selected design flood.

These methods have been developed by various institutions, and are either based on measured data (statistical); or on a deterministic basis (Rational, Unit Hydrograph and SDF methods); or are empirical relationships. Except for the statistical method, the methods were "calibrated" for certain regions and flood events, and are limited in terms of the size of the catchment areas on which they could be applied. **Table 2.1** lists the methods, input data requirements, maximum recommended catchment area for which each procedure can be used and references related to the procedures.

Table 2-1: Applications and limitation of flood calculation methods

<table>
<tr><th>Method</th><th>Input data</th><th>Recommended maximum area (km²)</th><th>Return period of floods that could be determined (years)</th><th>Reference paragraph</th></tr>
<tr><td>Statistical method</td><td>Historical flood peak records</td><td>No limitation (larger areas)</td><td>2-200 (depending on the record length)</td><td>3.4</td></tr>
<tr><td>Rational method</td><td rowspan="2">Catchment area, watercourse length, average slope, catchment characteristics, rainfall intensity</td><td>< 15</td><td>2-100, PMF</td><td>3.5.1</td></tr>
<tr><td>Alternative Rational method</td><td>No limitation</td><td>2-200, PMF</td><td>3.5.2</td></tr>
<tr><td>Synthetic Hydrograph method</td><td>Catchment area, watercourse length, length to catchment centroid (centre), mean annual rainfall, veld type and synthetic regional unit hydrographs</td><td>15 to 5000</td><td>2-100</td><td>3.5.3</td></tr>
<tr><td>Standard Design Flood method</td><td>Catchment area, slope and SDF basin number</td><td>No limitation</td><td>2-200</td><td>3.5.4</td></tr>
<tr><td>Empirical methods</td><td>Catchment area, watercourse length, distance to catchment centroid, mean annual rainfall</td><td>No limitation (larger areas)</td><td>10-100, RMF</td><td>3.6</td></tr>
</table>

Methods that are not discussed are the SCS and Run hydrograph methods (Smithers, 2000)

The procedures on which these methods are based are briefly reflected here and are then followed by a detailed discussion in separate sections.

Statistical methods involve the use of historical data to determine the flood for a given return period. Their use is thus limited to catchments for which suitable flood records are available, or for which records from adjacent catchments are comparable and may be used. Where accurate records covering a long period are available, statistical methods are very useful to determine flood peaks for long return periods. The method lends itself to extrapolation of data to determine flood magnitudes for longer return periods.

The **Rational method** is based on a simplified representation of the law of conservation of mass. Rainfall intensity is an important input in the calculations. Because uniform spatial and temporal distributions of rainfall have to be assumed, the method is normally only recommended for catchments smaller than about 15 km^2. Only flood peaks and empirical hydrographs can be determined by means of the rational method. Judgement and experience on the part of the user with regard to the run-off coefficient selection is important in this method, but thanks to improved methods, subjective judgement is becoming less important.

The **Alternative Rational method** is an adaptation of the standard rational method. Where the rational method uses the depth-duration-return period diagram to determine the point precipitation, the alternative method uses the modified recalibrated Hershfield equation as proposed by Alexander for storm durations up to 6 hours, and the Department of Water Affairs' technical report TR102 for durations from 1 to 7 days (Alexander, 2001).

The **Unit Hydrograph method** is suitable for the determination of flood peaks as well as hydrographs for medium-sized rural catchments (15 to 5 000 km^2). The method is based mainly on regional analyses of historical data, and is independent of personal judgement. The results are reliable, although some natural variability in the hydrological occurrences is lost through the broad regional divisions and the averaged form of the hydrographs. This is especially true in the case of catchments smaller than say 100 km^2 in size.

The **Standard Design Flood (SDF) method** was developed by Alexander to provide a uniform approach to flood calculations. The method is based on a calibrated discharge coefficient for a recurrence period of 2 and 100 years. Calibrated discharge parameters are based on historical data and were determined for 29 homogeneous basins in South Africa (Alexander 2002).

Empirical methods require a combination of experience, historical data and/or the results of other methods. Empirical methods are more suited to check the order of magnitude of the results obtained by means of the other methods.

2.3 Factors affecting run-off

In flood hydrology it is essential to be familiar with and to understand the influence of the various factors affecting run-off before an attempt is made to undertake hydrological calculations. Such factors may be broadly classified as:

- topographical factors;
- developmental influences; and
- climatological variables.

These factors are mutually dependent. Only the most important factors in the above-mentioned classes are discussed hereafter.

2.3.1 Topographical factors

2.3.1.1 Size of catchment

The size of a catchment has an important influence on the rainfall/run-off relationship, and consequently on the suitability of calculation methods. In small catchments for example, the relationship between rainfall intensity and infiltration rate of the soil is very important, whereas in large catchments the quantity of rainfall relative to the water storage capacity of the ground is more important. The peak discharges of small streams within the same geographical area are approximately proportionate to the sizes of the catchments (catchment area < 10 km^2). As the catchment becomes large, the peak run-off becomes proportionate to $\sqrt{A}$. The effect of other factors, however, often decreases the influence of catchment size alone.

Topographical maps (1: 50 000) are usually used to determine the area of a catchment. However, for small catchments the accuracy and contour intervals on these maps are not acceptable and topographic detail on a smaller scale, say 1: 10 000, should be obtained. Ortho-photographs should be used, if available. It is considered essential for the designer to visit a catchment personally to obtain an impression of developments and other important characteristics of the catchment.

2.3.1.2 Catchment shape

Even if all other characteristics are the same, fan-shaped catchments (Storm 1) will give rise to higher peak flows than long, narrow catchments (Storm 2). The slope of the main watercourse and other factors may, however, neutralise this influence. When reduction factors are used to adjust rainfall intensities, the movement and intensity of a storm passing over the catchment should be considered, since using only the size of the catchment could be misleading as is illustrated in **Figure 2-1**.

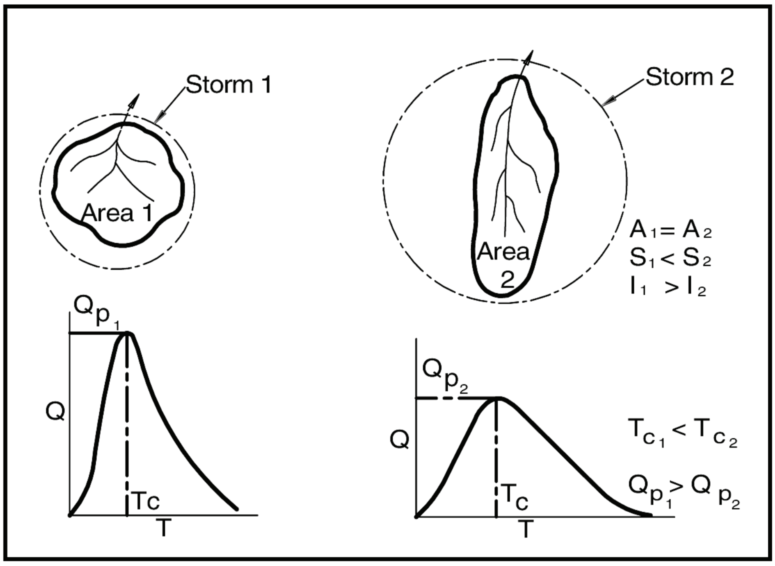


Figure 2-1: Catchments of the same size, but producing different peak discharges

2.3.1.3 Catchment slope

The slope of a catchment is a very important characteristic in the determination of flood peaks. Steep slopes cause water to flow faster and to shorten the critical duration of a flood-causing storm, thus leading to the use of higher rainfall intensities in the run-off formulae. On steep slopes the vegetation is generally less dense, soil layers are shallower, and there are fewer depressions, which cause water to run off more rapidly. The result is that infiltration is reduced and flood peaks are consequently even higher.

Generally there is a good correlation between the slopes of the main watercourse, tributaries and the surrounding landscape. The slope of the main watercourse is usually determined from topographical maps.

2.3.1.4 Stream patterns

Well-drained catchments have shorter times of concentration and consequently give rise to larger peak flows. The hydraulic effectiveness of a watercourse, whether natural or man-made, affects the flow rates and, therefore, has to be taken into account.

Some streams have numerous tributaries, and others may have only one main watercourse, which receives run-off from overland flow. The meandering of watercourses, marshes and flows outside of river banks affect the flood's progress and increase attenuation of flood peaks.

2.3.1.5 Infiltration

Infiltration is the movement of water through the ground surface into the soil. Usually the infiltration rate is considerably higher at the start of precipitation than a few hours later. Soil moisture tension in the upper layers beneath the surface initially reinforces the effect of gravity to draw water into the soil. In time, however, the soil becomes increasingly saturated so that the tension decreases, capillary spaces become filled with water and infiltration takes place more slowly. Once the soil has become saturated, the surface infiltration rate becomes equal to the deeper infiltration rate to ground water (with interflow and evapotranspiration not considered). In **Paragraph 2.5** references are made to research that has been conducted.

2.3.1.6 Soil type and geology

Soil type has an important influence on the run-off, mainly because of the effect of the infiltration rate. The effect of the soil type also often depends on the volume, duration and intensity of rainfall. The condition of the soil at the onset of a storm will affect run-off. Freshly ploughed or unsaturated soil, for example, will produce a smaller run-off volume and peak discharge than compacted or saturated soil of the same type.

Underlying rock formations and other geological factors, such as riverine deposits, may have significant effects on run-off. Medium to large catchments with underlying dolomite, for instance, result in considerably reduced run-offs.

2.3.1.7 Seasonal effects of vegetation

Seasonal vegetation and falling leaves retard the flow of water and increase infiltration. Normally no provision is made in flood calculations for such seasonal effects.

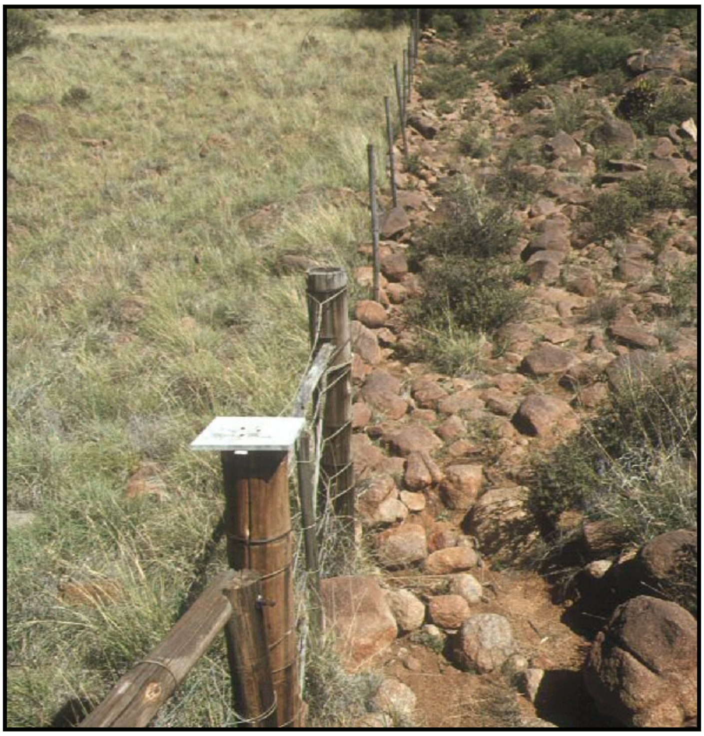


Figure 2-2: Variation in catchment cover

2.3.2 Developmental influences

2.3.2.1 Land use

Since human activities may well have a considerable effect on the run-off characteristics of a catchment, present and future conditions should be properly taken into account, particularly with regard to urbanisation. **Figure 2-2** reflects the influence of over grazing in a catchment. The effect of urbanisation depends on the percentage of the surface area that is made impermeable and on changes in the drainage pattern caused by stormwater systems. Urbanisation usually increases the size of flood peaks by 20 to 50 per cent of those under natural conditions. Where there is industrial or other high-density building development, this figure may rise to 100 per cent or more. Examples of the influence of urbanisation on the peak discharge from a catchment as a function of impermeable surface area, return period and percentage area with stormwater drainage are given in **Table 2-2** and **Table 2-3** (Todd, 1974).

Figure 2-3 provides an indication of urban development. The type of urban development in South Africa, where wall boundaries are common could, however, reduce the peak discharge rate while it increases the flow volume. Preliminary results support this conclusion. This is however dependent on the topographical characteristics and is site specific.

Table 2-2: Possible influence of % of impermeable surface area on peak flow, for different return periods, expressed as multiples (Todd, 1974)

Return period (years)	Percentage area consisting of man-made impermeable surfaces				
	1	**10**	**25**	**50**	**80**
2	1,0	1,8	2,2	2,6	3,0
5	1,0	1,6	2,0	2,4	2,6
10	1,0	1,6	1,9	2,2	2,4
25	1,0	1,5	1,8	2,0	2,2
50	1,0	1,4	1,7	1,9	2,0
100	1,0	1,4	1,6	1,7	1,8

Table 2-3: Possible influences on peak flow of % of impermeable surface area and % area with stormwater drainage, expressed as multiples (Todd, 1974)

Percentage area with stormwater drainage	Percentage of impermeable surface area					
	0	**20**	**40**	**60**	**80**	**100**
0	1,0	1,3	1,5	1,8	2,0	2,4
20	1,3	1,5	2,1	2,5	2,9	3,7
40	1,4	2,1	2,5	2,9	3,7	4,7
60	1,5	2,2	2,8	3,6	4,5	5,5
80	1,6	2,3	3,0	4,2	5,0	6,2
100	1,7	2,4	3,2	4,4	5,6	6,8

Figure 2-3 : Developmental influences (Loftus/University of Pretoria area)

2.3.2.2 Storage

Storage in a catchment occurs as detention storage (the filling up of small depressions in the ground surface), storage in overland (sheet) and river flows, as well as in pans, lakes, vleis and marshes. Storage could have a considerable effect on the attenuation and translation of flood peaks.

2.3.2.3 Reservoirs

Reservoirs may intercept large volumes of run-off and thus considerably reduce peak flows. Generally it is realistic to assume that reservoirs would be reasonably full when conditions that favour large floods (large catchments) occur. The effect of interception by reservoirs can be investigated by assuming that the reservoir is full and conducting routing calculations. Uncertainty regarding the operation of sluice gates during a flood, or prior to a flood event, to create storage volume for flood attenuation, complicates the assessment. The operational guidelines and policy for the release strategy should be investigated. A general assumption, however, is that the maximum controlled release from a dam should not be higher that the inflow peak.

2.3.3 Climatological variables

2.3.3.1 Climate

Climate has an important influence on many of the factors that affect run-off. Vegetation growth and soil formation, for example, are strongly affected by rainfall and temperature. There is a clear relationship between rainfall intensity and mean annual precipitation in different regions of South Africa. The wetter parts of the country generally experience higher rainfall intensities.

Areas with high rainfall generally also have wetter antecedent soil moisture conditions with correspondingly higher run-off from rainfall.

2.3.3.2 Rainfall as a flood parameter

In South Africa, rainfall is the most important form of precipitation, and together with hail, is mainly responsible for flood run-off. Snow does not contribute significantly to floods in SA, but hydrologically speaking contributes to low flow in certain regions. In large catchments the quantity, intensity and distribution of rainfall are important factors, but in the determination of flood run-off for small catchments, rainfall intensity remains the dominant factor.

The relationship between rainfall and run-off depends on many factors, which will be discussed later, and consequently cannot be simplified. Although the correlation between the rainfall return period and the resulting flood peak is poor, it has been found that when the peak run-off and rainfall are considered separately, the relationship between peak run-off for a given period and the rainfall intensity for the same return period remains reasonably constant for different return periods (Schaake, 1967). Rainfall could thus be used to determine design floods, although a rainstorm of a given return period very seldom results in a flood peak with the same return period.

2.3.3.3 Time and area distribution of rainstorms

The run-off from a catchment depends not only on the intensity and quantity of rainfall, but is also affected by the duration, size, uniformity, velocity and direction of any storm passing over the catchment. Rain rarely falls evenly over a catchment, with the result that the rainfall inputs and flood run-offs vary across such an area. The point-to-point differences in the area and time distribution of rainfall depend, in turn, on the type of rain, for example, convection, orographic, frontal or cyclonic rain.

Convection rain occurs in the form of thunderstorms and tends to be extremely uneven and unpredictable; orographic rain also shows significant point-to-point differences but the distribution is more predictable; frontal rain is fairly evenly distributed along the longitudinal direction of the front, but there are marked differences in the direction of movement.

In contrast, cyclonic rains show fairly even distributions with the heaviest precipitation and intensity at the centre. The type of rain that would cause floods depends largely on the location and size of the catchment.

In most of the methods of calculation used in road drainage, it is assumed that the flood-causing storm has a precipitation duration just long enough to allow run-off from all parts of the catchment to contribute simultaneously to the flood peak, hence the relationship between the critical duration of a storm and the so-called time of concentration (T_C) as well as other methods used to measure catchment response time. In large catchments heavy rainfall over only a part of the area may also cause flooding, but the design floods for large catchments are mainly obtained via statistical analyses of measured discharges.

Storms that move over a catchment in the downstream direction often cause larger flood peaks than stationary or other storms, since in effect they shorten the time of concentration. Where the prevailing direction of storms is usually downstream, particularly within a long catchment, an indication of the possible effect of such storms could theoretically be obtained by shortening the time of concentration by the time the storm takes to move across the catchment. Whilst storms may move at speeds of up to 50 km/h, it is difficult to determine a design speed. Storms also rarely move in a straight line. Such adjustments are generally not made in practice. **Figure 2-4** and **Figure 2-5** reflect different rain storm types.

Figure 2-4: Rainfall over catchment (frontal)

Figure 2-5: Convection rain

2.4 Deterministic methods

2.4.1 Rational method

2.4.1.1 Background and principles

The method is widely used and hence it is described in detail in this section.

This method was first proposed in 1851 by the Irish engineer, Mulvaney. Since then it has become one of the best-known and most widely used methods for determining peak flows from small catchments. The basis of the relationship is the law of the conservation of mass and the hypothesis that the flow rate is directly proportional to the size of the contributing area and the rainfall intensity, with the latter a function of the return period. The peak flow is obtained from the following relationship:

$$\boldsymbol{Q = \frac{CIA}{3.6}}$$

Equation 2-1

Where:

Q	=	peak flow (m^3/s)
C	=	run-off coefficient (dimensionless)
I	=	average rainfall intensity over catchment (mm/hour)
A	=	effective area of catchment (km^2)
3,6	=	conversion factor

The rational formula represents outflow in the hydraulic continuity equation, and its application is based on the following assumptions:

- The rainfall has a uniform area distribution across the total contributing catchment.
- The rainfall has a uniform time distribution for at least a duration equal to the time of concentration (T_C).
- The peak discharge occurs when the total catchment contributes to the flow, occurring at the end of the critical storm duration, or time of concentration (T_C).
- The run-off coefficient, C remains constant throughout the duration of the storm.
- The return period of the peak flow, T, is the same as that of the rainfall intensity.

Despite this method's shortcomings and widespread criticism, it provides realistic results if it is used circumspectly, and it has generally provide good results in studies when compared with other methods. Although it is generally recommended that the method should only be applied to catchments smaller than 15 km², it can in some cases be used by experienced engineers for larger catchments (HRU, 1978)

Many of the assumptions listed above are to a greater or lesser degree also applicable to other methods of flood calculation.

2.4.1.2 Run-off coefficient (C)

2.4.1.2.1 Recommended C values

The run-off coefficient in the rational method is an integrated value representing the most significant factors influencing the rainfall-run-off relationship. It reflects the part of the storm rainfall contributing to the peak flood run-off at the outlet of the catchment. There is no objective theoretical method for determining C, and as a result the subjective elements of experience and engineering judgement play a very important part in the successful application of this method.

Table 2.4 (DWA) provides a description of recommended values of C.

Table 2-4: Recommended values of run-off factor C for use in the rational method

Rural (C1)					Urban (C2)	
Component	**Classification**	**Mean annual rainfall (mm)**			**Use**	**Factor**
		< 600	**600-900**	**> 900**		
Surface slope (Cs)	Vleis and pans (<3%)	0,01	0,03	0,05	Lawns	
	Flat areas (3 to 10%)	0,06	0,08	0,11	- Sandy, flat (<2%)	0,05-0,10
	Hilly (10 to 30%)	0,12	0,16	0,20	- Sandy, steep (>7%)	0,15-0,20
	Steep areas (>30%)	0,22	0,26	0,30	- Heavy soil, flat (<2%)	0,13-0,17
					- Heavy soil, steep (>7%)	0,25-0,35
Permeability (Cp)	Very permeable	0,03	0,04	0,05	Residential areas	
	Permeable	0,06	0,08	0,10	- Houses	0,30-0,50
	Semi-permeable	0,12	0,16	0,20	- Flats	0,50-0,70
	Impermeable	0,21	0,26	0,30		
					Industry	
					- Light industry	0,50-0,80
Vegetation (Cv)	Thick bush and plantation	0,03	0,04	0,05	- Heavy industry	0,60-0,90
	Light bush and farm lands	0,07	0,11	0,15	Business	
					- City centre	0,70-0,95
	Grasslands	0,17	0,21	0,25	- Suburban	0,50-0,70
	No vegetation	0,26	0,28	0,30	- Streets	0,70-0,95
					- Maximum flood	1,00

2.4.1.2.2 Rural areas

In rural areas there are five main factors affecting the value of C, namely catchment slope, permeability of the soil, vegetation, mean annual rainfall and return period.

Accurate and time-consuming calculation of the slope is not necessary. It may be determined with sufficient accuracy by selecting a representative part of the catchment on a contour map and determining a slope according to these contours. Steeper slopes give rise to higher run-off percentages.

The following classification could be used as a qualitative guide to the permeability of the soil:

- Very permeable - gravel, coarse sand
- Permeable - sandy, sandy loam
- Semi-permeable - silt, loam, clayey sand
- Impermeable - clay, peat rock

The classification could be made from a visual inspection of the terrain and/or by using the soil maps, available from the Government Printers. Where dolomite occurs, the following reduction factors are recommended for the dolomitic parts of a catchment to be applied to C_s in **Table 2-4** on a pro rata basis:

- Steep areas (slopes > 30%) - 0,50
- Hilly (10 to 30%) - 0,35
- Flat areas (3 to 10%) - 0,20
- Vleis and pans (slopes < 3%) - 0,10

Vegetation could be classified as follows:

- Forestry plantations
- Dense bush or bushveld
- Light bush and cultivated lands
- Grasslands
- No vegetation

The vegetation should be determined by inspections in loco, although the publication by Acocks, "Veld types of South Africa" may also be useful (Acocks, 1953).

Where the periodic felling of trees in forestry plantations could have a considerable influence on the run-off from a specific catchment, the C value should be increased by taking into account the proportionate part that would be left without effective plant cover. Where this proportionate part is greater than about 30 per cent of the catchment area covered with trees, the return period should be re-determined.

The mean annual rainfall also affects the run-off, as discussed previously. Recommended values of the catchment's response influenced by the slope C_s, permeability C_p and vegetation C_v are given in **Table 2-4** for different classes of mean annual rainfall.

The return period has an important effect on the run-off percentage. The relationship between rainfall and run-off is not linear and a catchment is often more saturated at the start of a storm with a long return period than is the case with storms of shorter return periods. It is thus recommended that the C value $C_1 = C_s + C_p + C_v$ be multiplied by the appropriate factor (F_t) from **Table 2-5.**

The influence of initial saturation is, however, also dependent on the catchment characteristics. The influence of the return period will thus be smaller for steep and impermeable catchments than for flat permeable catchments. For these cases the factors are given in **Table 2-5**.

Table 2-5: Adjustment factors for value of C1

Return period (years)	2	5	10	20	50	100
Factor (Ft) for steep and impermeable catchments	0,75	0,80	0,85	0,90	0,95	1,00
Factor (Ft) for flat and permeable catchments	0,50	0,55	0,60	0,67	0,83	1,00

For the probable maximum flood (PMF), $C_1 = C_s + C_{pmax} + C_{vmax}$; $C_2 = 1$; and $F_t = 1$. Where C_{pmax} and C_{vmax} are in this case the maximum values from **Table 2-4**.

2.4.1.2.3 Urban areas

Recommended values of C for urban areas are given in **Table 2-4**. Because of the fairly large percentages of impermeable surface area in urban areas, it is normally not necessary to adjust the value of C according to the return period. Adjustment is, however, possible in accordance with **Table 2-5.**

2.4.1.3 Rainfall intensity (I)

The intensity of a design storm increases as the return period becomes longer and as the duration of the storm decreases. To obtain the largest possible peak discharge for a given return period using the rational method, the storm rainfall should have a duration equal to the time required for the whole catchment to contribute to run-off, defined as the time of concentration, T_C. If the storm has a shorter duration, it will not be possible for all the parts of the catchment to contribute simultaneously to run-off at the point of measurement. Consequently, the effective catchment area would be smaller than the actual area of the catchment.

Apart from the duration and return period, the intensity of rainfall is also related to the mean annual rainfall and to the rainfall region. The "depth-duration-return period" relationship depicted in **Figure 2-6** (HRU, 1979) may be used to determine point rainfall, which is then converted to intensity by dividing the point rainfall by the time of concentration ($P_{iT} = P_T / T_C$). In road drainage the volume of water that runs off as a result of a storm of less than 15 minutes duration is usually not large, and much of this run-off is absorbed in filling of the watercourses. Times of concentration of less than 15 minutes are thus generally not significant, and the maximum intensity is assumed to occur at approximately this time. It is difficult to calculate the rainfall intensity for storms less than 15 minutes (i.e. time of concentration of less than 15 minutes) and thus the intensity is based on the assumption that the storm duration is 15 minutes. The mean annual rainfall could be obtained from the simplified **Figure 2-7** or from the Weather Service (Weather Bureau) as well as other alternative sources (Van Heerden, 1978) such as the Hydraulic Research Unit reports (HRU, 1978).

2.4.1.3.1 Time of concentration (T_C)

The time of concentration, T_C, is defined as the required time for a storm of uniform area and temporal distribution to contribute to the run-off from the catchment. In calculating the time of concentration, distinction is made between overland flow (sheet flow) and flow in defined watercourses.

(i) Calculation of the time of concentration for overland flow

This type of flow usually occurs in small, flat catchments or in upper reaches of catchments, where there is no clearly defined watercourse. Run-off, then, is in the form of thin layers of water flowing slowly over the fairly uneven ground surface. The Kerby formula is recommended for the calculation of T_C in this case. It is only applicable to parts where the slope is fairly even.

$$T_C = 0.604\left(\frac{rL}{S^{0.5}}\right)^{0.467}$$

Equation 2-2

Where:

T_C	=	time of concentration (hours)
r	=	roughness coefficient obtained from **Table 2-6**
L	=	hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)
S	=	slope of the catchment (m/m)
H	=	height of most remote point above outlet of catchment (m)

Table 2-6: Recommended values of r

Surface description	Recommended value of r
Paved areas	0,02
Clean compacted soil, no stones	0,1
Sparse grass over fairly rough surface	0,3
Medium grass cover	0,4
Thick grass cover	0,8

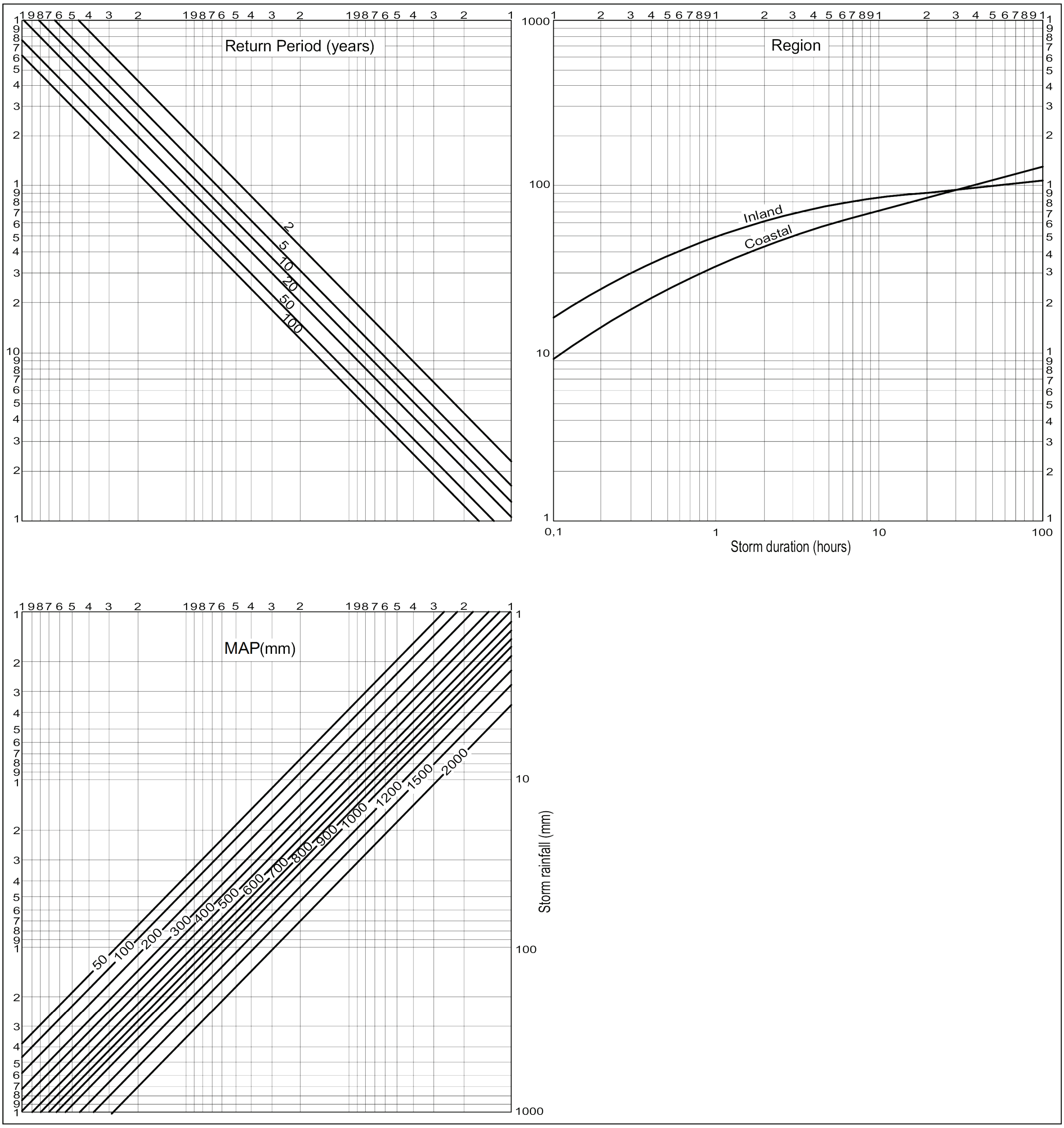


Figure 2-6: Depth-Duration-Frequency diagram for point rainfall

Figure 2-6 was obtained from HRU Report 2/78, *A Depth-Duration-Frequency Diagram for Point Rainfall in Southern Africa* (1978), which is an update from a similar diagram published in HRU Report 1/72, *Design Flood Determination in South Africa* in 1972 (HRU, 1978, 1979).

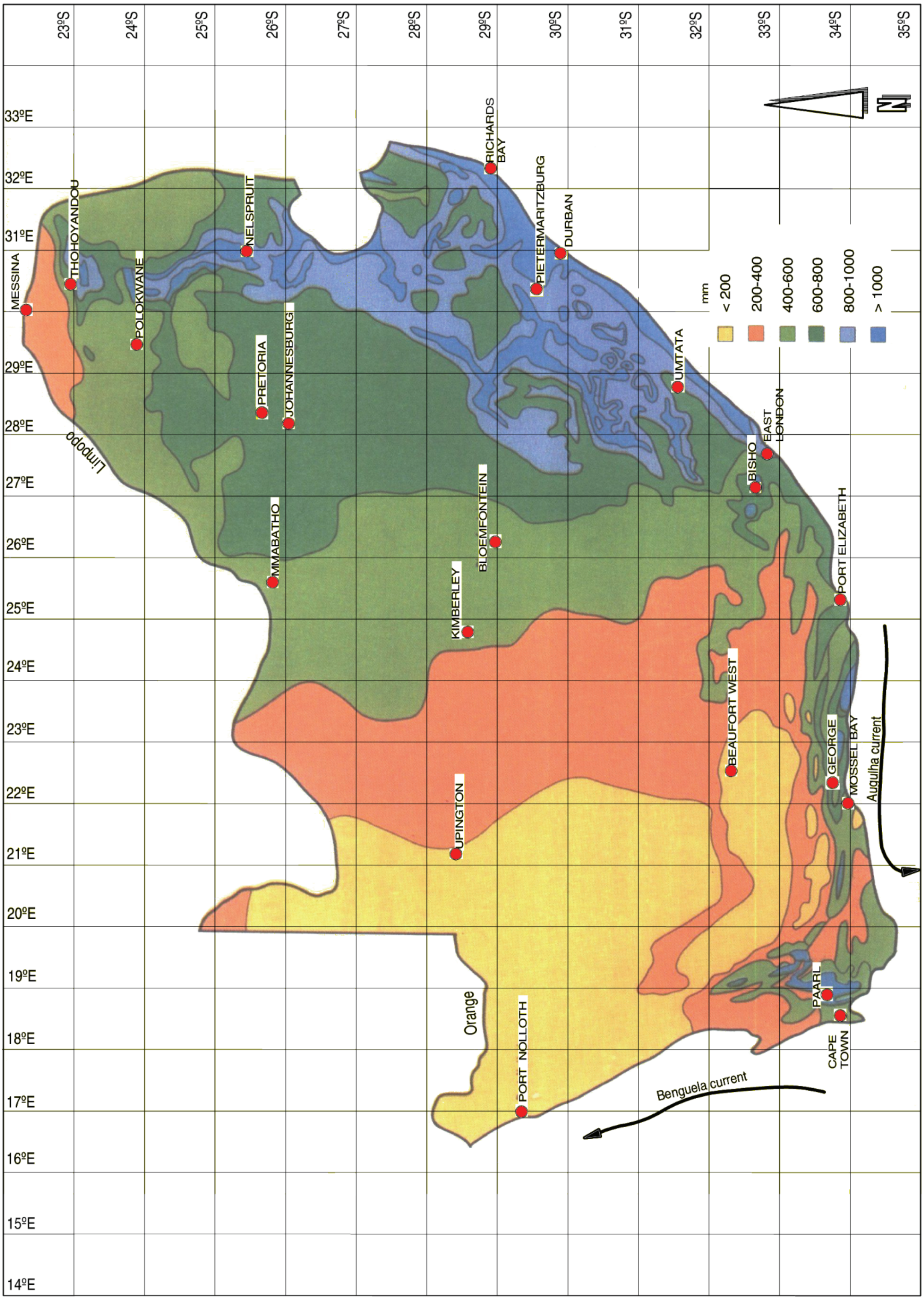


Figure 2-7: Average annual rainfall in southern Africa

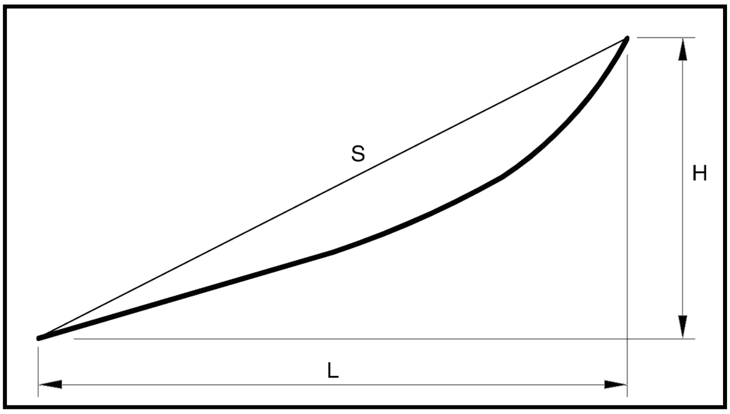


Figure 2-8: Slope definition for overland flow

(ii) Calculation of time of concentration for a defined watercourse

In a defined watercourse, channel flow occurs. The recommended empirical formula for calculating the time of concentration in natural channels was developed by the US Soil Conservation Service.

$$T_C = \left(\frac{0.87L^2}{1000S_{av}}\right)^{0.385}$$

Equation 2-3

Where:

T_C = time of concentration (hours)

L = hydraulic length of catchment, measured along flow path from the catchment boundary to the point where the flood needs to be determined (km)

S_{av} = average slope (m/m)

The average slope may be determined graphically in two ways. The first procedure is based on the balance of areas obtained by balancing the areas above and below the line of average slope, as shown in **Figure 2-9**. Alternatively the formula developed by the US Geological Survey, and referred to as the 1085-slope method could be used (**Figure 2-10)**.

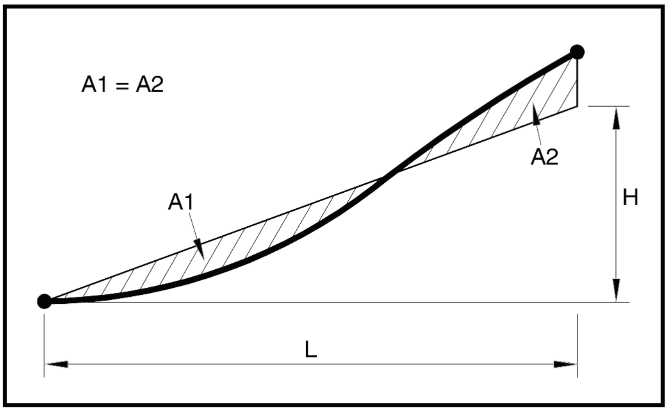


Figure 2-9: Slope according to weighted area method

In most cases the longest water path includes both overland and channel flow. In large catchments the channel flow is usually dominant, but in small catchments it may be necessary to determine T_C as the sum of the flow times, for overland and channel flow. To obtain a broad indication, it may usually be accepted that a defined watercourse exists when the average slope of the catchment is greater than 5 per cent, and the catchment itself is larger than 5 km².

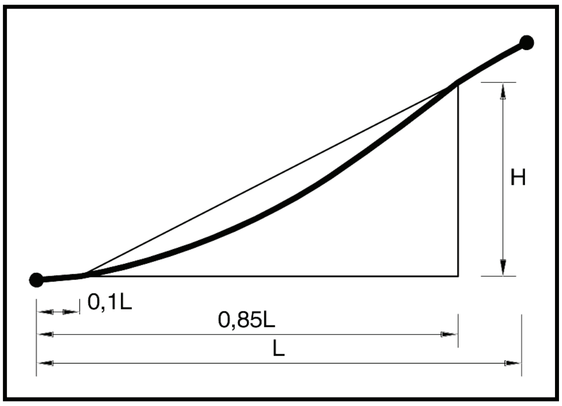


Figure 2-10: 1085-Slope according to "US Geological survey"

The formula for determining the slope according to the 1085 slope method reads:

$$S_{av} = \frac{H_{0,85L} - H_{0,10L}}{(1\,000)(0{,}75L)} \qquad \text{or} \qquad S_{av} = \frac{H}{(1\,000)(0{,}75L)}$$

Equation 2-4

Where:

S_{av}	=	average slope (m/m)
$H_{0,10L}$	=	elevation height at 10% of the length of the watercourse (m)
$H_{0,85L}$	=	elevation height at 85% of the length of the watercourse (m)
L	=	length of watercourse (km)
H	=	$H_{0,85L} - H_{0,10L}$ (m)

The height of waterfalls and high rapids are subtracted from the gross H value.

(iii) Calculation of the time of concentration for urban areas

In urban areas the time of concentration should be determined, where applicable, by means of the flow velocities according to the Chezy or Manning equation for uniform flow through representative cross-sections with representative slopes.

In road drainage the volume of water that runs off as a result of a storm of less than 15 minutes' duration is usually not large, and much of this run-off is absorbed in filling of the watercourses. Times of concentration of less than 15 minutes are thus generally not significant.

It is sound practice to calculate the average flow velocity $(v = L/T_C)$ after determining T_C in order to ensure that it falls within realistic limits. Typical values of the flow velocity range from 0,1 to 4 m/s, depending on the natural conditions.

2.4.1.4 Effective catchment area (A)

The effective area is that part of the total catchment which would contribute to the peak flow. Pans or areas that are artificially cut off should consequently be excluded.

2.4.1.5 Simplified hydrograph for the Rational method

Although the rational method is not strictly suitable for determining hydrographs, a simple triangular hydrograph could be used for low-risk application, such as flood routing through a culvert or the determination of the run-off volume. A typical triangular hydrograph is shown in **Figure 2-11**.

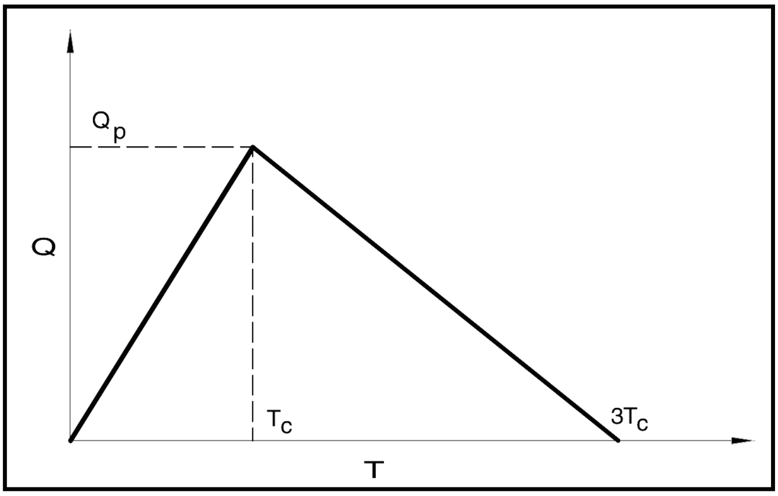


Figure 2-11: Simplified (triangular) hydrograph for the rational method

2.5 Other studies conducted on some of the factors that influence run-off peaks from urban catchments

2.5.1 Introduction

Based on the write-up above in which two references conducted on the influence of infiltration on the peak flow discharge and the influence of roughness, flow path length and infiltration on the peak discharge rate and run-off volume is discussed below. In none of these assessments the influence of temporal storage was included.

2.5.2 Infiltration and hydraulic head

Infiltration is one of the most difficult parameters to estimate due to the vast unknown parameters associated with infiltration. The Department of Soil and Water Engineering, COAE, Punjab Agricultural University, Ludhiana, India conducted a study on groundwater recharge through surface drains (Singh et al, 2005).

The objective of the study was to develop a mathematical model for the estimation of groundwater recharge for both free flow as well as detained flow conditions. The model can be utilised to estimate the groundwater recharge through unlined stormwater drains and detention facilities. Field measurements were utilised to calibrate the model. It was concluded **that the recharge rate increases with an increase in the flow depth**. The recharge volumes under detained flow conditions varied between 1,2 and 9,4 times more than what would have recharged under free flow conditions. The relationship of detained flow / free flow recharge ratios for various initial soil moisture conditions, all except one of the nine scenarios, was below a factor of three.

In a South African context where boundary walls are erected around most properties, temporal detention is created resulting in higher infiltration rates. The discharge openings through the boundary walls are often limiting the discharge rate during major storm events. In addition to the higher infiltration rate, flood routing is also introduced due to the temporal storage that further contributes to the reduction in the peak flows. This phenomenon can result in a scenario where the difference in peak flows for pre and post development for the major return periods will be much less than that of the minor events.

Current available software can be used for flood routing. It might not be feasible for authorities and developers to conduct these analyses. Furthermore there is a lack of a description of a relationship between infiltration and hydraulic head.

2.5.3 Selection of Stormwater Model Parameters

Stephenson (Stephenson, 1989) modelled an urban catchment in Sunninghill, north of Johannesburg. The objective of the study was to test the hypothesis that a fine discretization level model will increase the accuracy of the results. The WITWAT (Green, 1984) model was utilised and calibrated against an observed storm event, which occurred on 20 December 1986.

The storm had a duration of 3 000 seconds (50 minutes), with a total downpour of 26 mm and a peak intensity of 120 mm/h.

The aim was to calibrate the Manning roughness and infiltration rates for the study area. Both these parameters contain characteristics of the hydrological processes that occur within the residential stand boundary.

The researchers calibrated these two parameters for both the peak flow rate and runoff volume. It was found that a 50% variation in infiltration affected runoff volume by 30%. Furthermore it was established that a 50% variation in Manning roughness affects the peak runoff by up to 20%. The conclusion was that both the rate of infiltration as well as the roughness has to be altered extremely to obtain a reasonable estimate of the peak discharge rate.

Table 2-7 reflects the findings of the assessment of the variation of the different parameters (Mannings roughness, initial infiltration rate and the overland flow path length).

Table 2-7: Summary of the results that were obtained from the assessment

Changing the Manning "n" roughness for the pervious areas		Changing the Initial Infiltration, f mm/h		Changing the Length factor that reflects the flow path		Level of discretization that was used in the analyses
Initial infiltration = Standard value Length factor = 1.0		Manning "n" = Standard value Length factor = 1.0		Initial infiltration = Standard value Manning "n" = Standard value		
Parameter variation	Volume calculated	Parameter variation	Volume calculated	Parameter variation	Volume calculated	
0.170	0.110	45.2	64.1	3.5	1.6	Very Fine
0.186	0.061	59.7	47.9	3.0	0.9	Fine
0.260	0.048	73.0	61.6	2.1	0.7	Medium
0.185	0.027	48.4	53.0	2.0	0.65	Course

Table 2-7 reflects that:

- For all the levels of discretization two separate analyses are required, one to determine the peak runoff rate and another to determine the runoff volume. This is major problem if runoff hydrographs is required for the design of an attenuation pond or hydraulic structure.

- If the standard values for both the initial infiltration rate and the Manning roughness are used, the flow path was increased between 2 and 3,5 times the measured length from contour plans, to obtain a reasonably comparable calculated discharge rate to calibrate the peak runoff rate. This indicates that under estimation of the response time of the catchment and results in an over estimate of the peak flow rates.

- Calibration of any method used remains essential for accurate estimates and further studies need to be conducted to obtain a better understanding of the relative influence of these parameters for other cases

Additional literature of studies that were conducted on the effect of urban development on runoff response are supplied in List of Appendices on the enclosed CD

Appendix A

3 Field work conducted to review the change in peak discharge associated with the development in the urban catchments

3.1 Introduction

In order to reflect the influence of development on catchment response it is required to identify catchments for which historical data (rainfall and runoff) is available and for which the gauging infrastructure is still available.

The investigation to determine the influence of catchment development on the runoff response was conducted in a comparison manner. Firstly the evaluation was conducted for catchments which had similar storm events for different development conditions. Secondly larger catchments were evaluated where the volumetric runoff response for years with similar yearly rainfall and different development levels were compared (**Section 6)**.

The following catchments were identified, based on the available information for the evaluation of comparable storm events:

Catchment 1: Willow Ridge Catchment – area developed from low density agricultural land use to a high density residential area. The City Council of Tshwane (CCT) anticipated urban development in this catchment and has conducted flow and rainfall recordings for a period prior to development (October 1993 to May 1995).

A second catchment was identified on which a comparative assessment based on the recorded rainfall and runoff could be compared to the modelled response from the catchment. **Catchment 2** a High density household development catchment in Willows, Pretoria (Tshwane – Erf 683 and 685, Equestria 137) was selected for this purpose.

Details of each of the catchments are reflected below.

3.2 Details of Catchment 1: Willowspruit catchment – Tshwane

CCT has been recording flow data since 1993 at a gauging structure just upstream from the Willowspruit crossing of the N4. The contributing catchment area is reflected in **Figure 3-1;** which also indicates the level of development in 2007 in this catchment. The gauging weir is just downstream from the bridge in Ouklipmuur Avenue (**Figure 3-2**).

Figure 3-3 and **Figure 3-4** reflect the gauging structure and **Figure 3-5** shows the culverts at the Ouklipmuur Avenue.

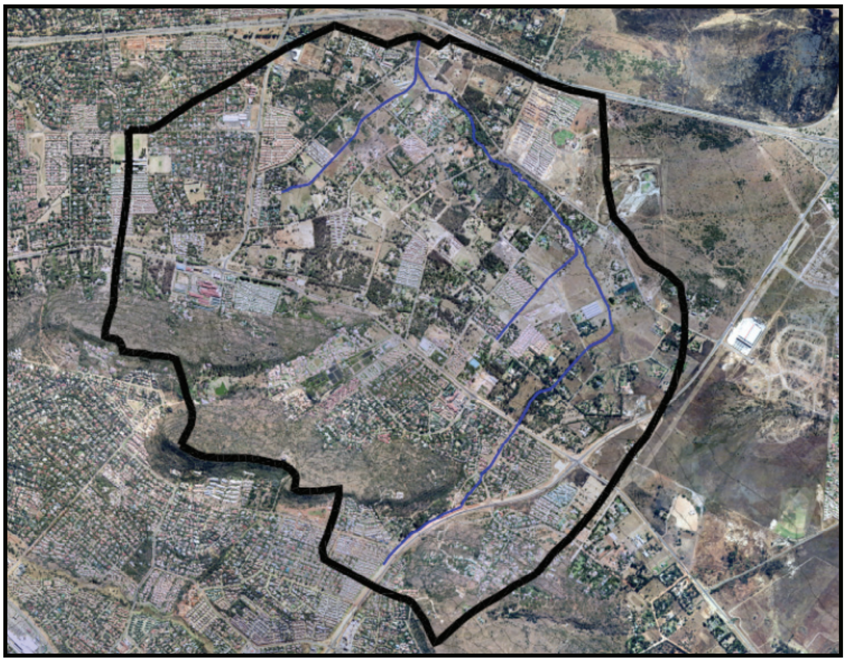


Figure 3-1: Level of development in the Willowspruit catchment

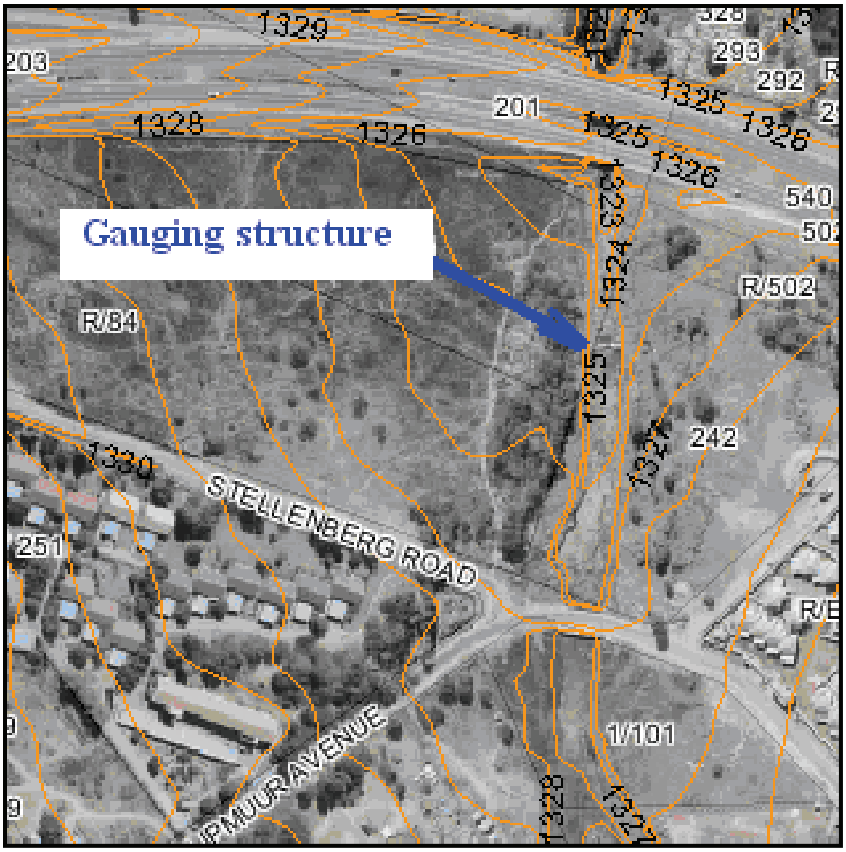


Figure 3-2: Location of the gauging structure for the runoff from Willowspruit catchment

Figure 3-3 reflects that there was some siltation that had to be removed to ensure accurate flow gauging. An instruction for the cleaning of the siltation was issued by the City Council of Tshwane after permission was granted by the Department of Environmental Affairs during the latter part of 2009. **Figure 3-4** reflects the structure which houses the water level recorded. Just upstream of the gauging structure there is a culvert that is partly blocked (**Figure 3-5**). **Table 3-1** reflects some details of the Willowspruit catchment. It must be noted that the siltation that was present before removal in 2009, was assumed not to have affected the recorded discharge data for the period 1993 to 1995.

Figure 3-3: Gauging station with siltation on the left flank

Figure 3-4: Structure that houses the water level recorder

Figure 3-5: Culvert upstream from gauging weir in the Willowspruit

Table 3-1: Catchment parameters of the Willowspruit

Parameter	Value
Catchment	Pienaars River
Drainage region	A2H
Area of catchment (km^2)	10,4
Reference of the runoff data	Strip recorded data to be transferred to digital format
River	Willowspruit
MAP (mm)	750
Q_{50} (m^3/s)	84
Time of concentration, (h)	0,9
Rainfall intensity, $I_{50,}$ (mm/h)	92,3
Adopted runoff coefficient, C	0,34
Reference period	1993-present
Current level of development (2007)	Largely developed – See **Figure 3-1**

Appendix B reflects details of the hydraulic structures in the Willowspruit catchment in 2008.

3.3 Details of Catchment 2: Robert's Place – Tshwane (Erf 683 and 685, Equestria 137)

This catchment has been developed recently and reflects a typical "security type high density residential development". **Figure 3-6** reflects the number of housing units in this development which has been divided into a **Zone A** and **Zone B**.

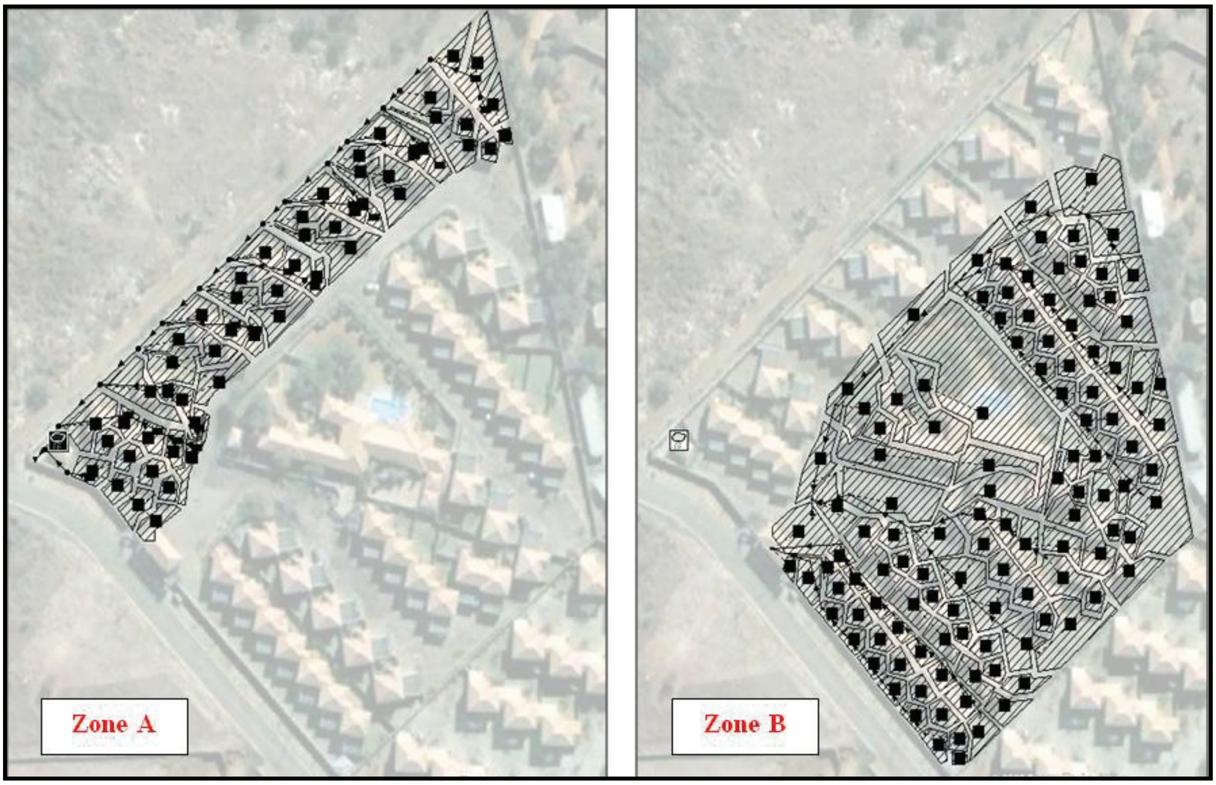


Figure 3-6: Details of the development in the catchment (Zone A and Zone B)

3.4 Available hydrological data

3.4.1 Catchment 1: Willowspruit – Tshwane

Tshwane City Council (CCT) has installed a number of autographic rain gauges. Three of these gauges were located in the Willowspruit catchment. **Table 3-2** reflects details of the rainfall stations which recorded the rainfall since 1993 but for which all the data was apparently lost.

Table 3-2: Autographic rain gauges

Autographic Rain Gauges			
ID Number (1)	7	8	9
Recording started	1993		
Description of location	Koedoesnek Reservoir	Willow Glen Channel	Equestria

The flow was recorded at the flow gauging station (**Figure 3-3**). The water level recording charts of the gauging at the Willowspruit for the period 1991 up to 1995 have been received and digitized. **Figure 3-7** reflects the typical details of the recording chart.

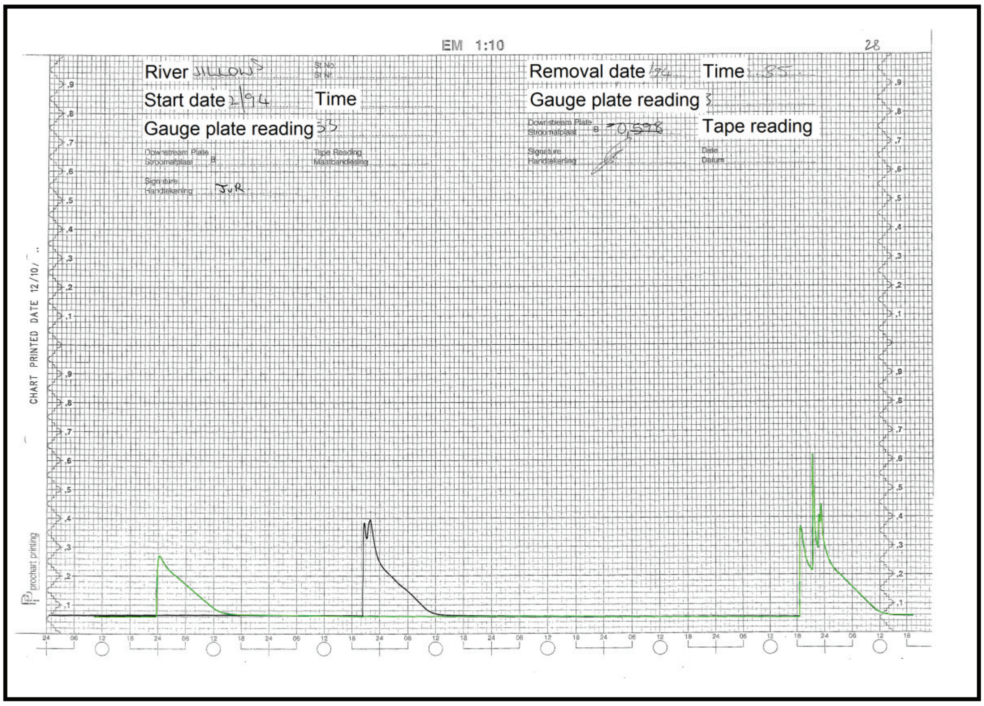


Figure 3-7: Water level variations recorded by the chart recorder

3.4.2 Catchment 2: Robert's Place – Tshwane (Erf 683 and 685, Equestria 137)

No historical hydrological data is available or required for this development since the assessment of a number of storm events at the fully developed level will be compared to the modelled discharge from the two Zones in this catchment. An autographic rainfall data recorder was installed and the discharges from the zones were recorded at two flow gauging stations.

3.4.3 Summary

Table 3-3 summarizes the location where hydrological rainfall data was acquired for each catchment.

Table 3-3: Summary of locations for data acquisition

Catchment	Stations to measure daily rainfall	Stations to measure intensities
1 – Willowspruit	Ramkie oor Willows	Pretoria Eendracht & Irene WO
2 – Robert's Place	New autographic rain gauge installed at Robert's Place	

3.5 Flow gauging infrastructure

3.5.1 Catchment 1: Willowspruit – Tshwane

A flow gauging station was constructed in 1991 in the Willowspruit canal at the outlet point of the catchment. The gauging structure is a V-Crump weir, identified by Wessel and Rooseboom (2009a) as the preferred choice when low discharges are expected to be prominent. Furthermore the structure is robust enough to handle high flow rates and sediment build-up on the measuring structure is reduced by the shape.

An upstream view of the set-up of the gauging station is illustrated in **Figure 3-8**. The gauging system has 3 main components namely:

- the weir structure (including lower level pipe outlets),
- the upstream gauge plate (located in the detention pond), and
- the manhole structure hosting the water level recorder (connected to the detention pond by means of a pipe).

Figure 3-8: V-Crump weir with an upstream measuring plate and inlet pipe to the manhole

The upstream water level (in the pond) is recorded with the equipment installed in the manhole structure, where it is assumed that atmospheric pressure conditions prevail. The water level inside the manhole structure reflects the water level upstream from the gauging weir. This setup reduces the influence of wave action. The water level is used to obtain the upstream energy level (depth of flow) which is required for discharge calculations.

A survey of the weir profile was conducted to verify its dimensions. The dimensions required for the calculation of flow over the weir structure were obtained. The cross section of the weir crest is shown in **Figure 3-9** on which the following dimensions are reflected:

- the crest cross slope, n = 9,5 m/m (the angle of the V is 2,93 rad); and
- the level of the V portion on the weir crest, with reference to the horizontal sections, D_V = 0,610 (m).

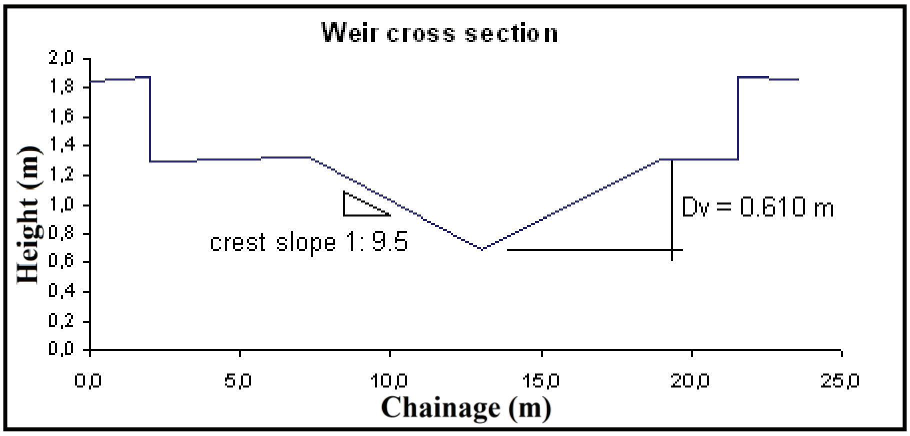


Figure 3-9: Surveyed cross section of the weir crest

The upstream and downstream slopes from the crest of the weir were also surveyed. It was found that these slopes are not uniform. The upstream slope ranges between 1:2,9 and 1:5 with the slope increasing towards the crest. The same phenomenon was observed for the downstream slope, ranging between 1:6,1 and 1:12,7. The average slopes of 1:3,5 and 1:8,3 applies to the upstream and downstream slopes respectively. A plan view of the structure is illustrated in **Figure 3-10** with the appropriate heights as surveyed denoted at several positions. The values denoted in brackets are to indicate the variance in the surveyed measurements.

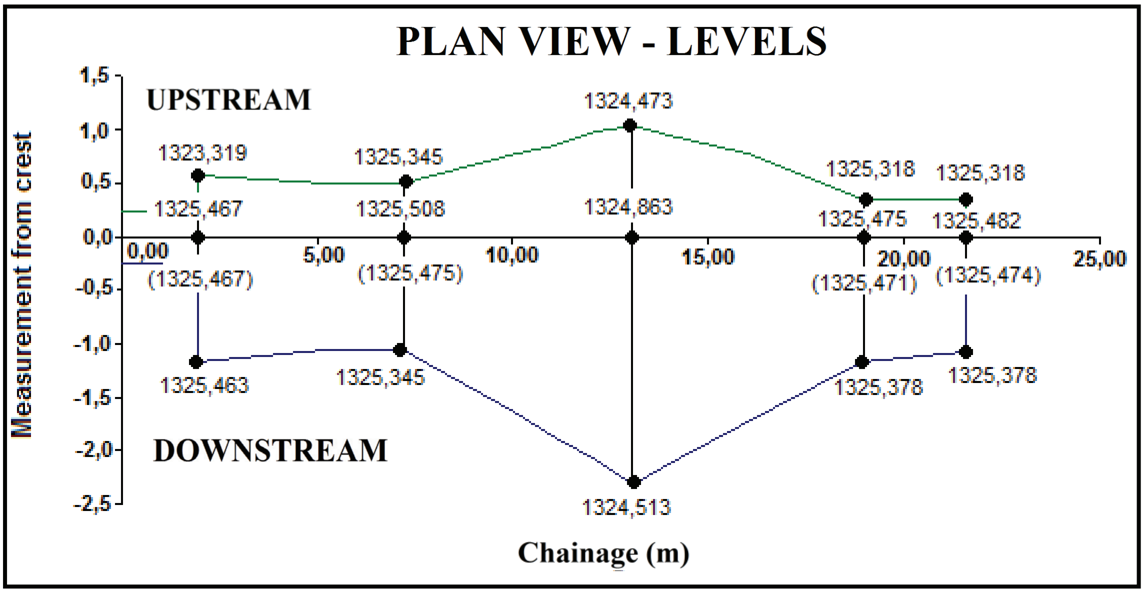


Figure 3-10: Plan view of the V-crump weir with denoted levels (Willowspruit)

The variability in the dimensions could have a significant effect on the accuracy of the flow measurements. The horizontal distances relative to the weir crest are indicated in **Figure 3-11**.

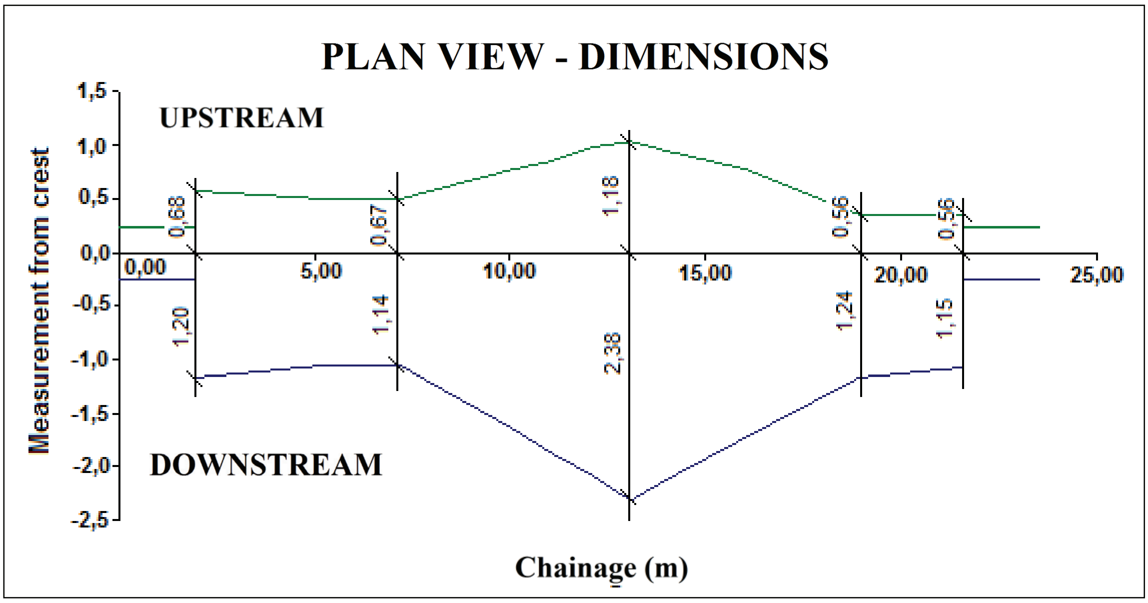


Figure 3-11: Plan view of the V-crump weir with denoted horizontal dimensions (Willowspruit)

Drain pipes were installed as part of the weir structure. When these pipes are open, the structure would serve as a compound weir. The invert level of the pipes culverts is approximately 0,5 m below the weir crest. **Figure 3-12** reflects the bottom drain pipe outlets and the connecting pipe between the detention pond and the manhole structure.

Figure 3-12: UPSTREAM view of the compound weir structure and the pipe connecting the detention pond and manhole housing the measuring device (Willowspruit)

It is apparent from surveyed data that the invert level of the connecting pipe, as it enters the manhole, is at a level 0,15 m below the weir crest. This would entail that the water level cannot be recorded in cases where the water level in the pond is lower than 0,15 m below the weir crest. This results that no flow released through the pipes can be recorded.

The DL/N (Data logger for level measurement with conductivity module) pressure probe manufactured by STS (Sensoren Transmitter System) was used in conjunction with the V-notch to accurately log the water level upstream from the V-notch. The pressure probe can measure up to 3 bars and has an accuracy of ± 0,25%. The Data logger can log up to 500 000 measurements and the measuring device runs on two 3,6 V type AA batteries. **Figure 3-13** shows the probe and data logger connected by a fairly sensitive tube type wire (Data Logger For Level Instruments User Manual).


Figure 3-13: DNL pressure probe and data logger

3.5.2 Catchment 2: Robert's Place – Tshwane (Erf 683 and 685, Equestria 137)

Figure 3-14 shows a plan view of the town house complex indicating the location of the discharge measuring positions for Zone A and B. These measuring structures were located at existing draining structures.

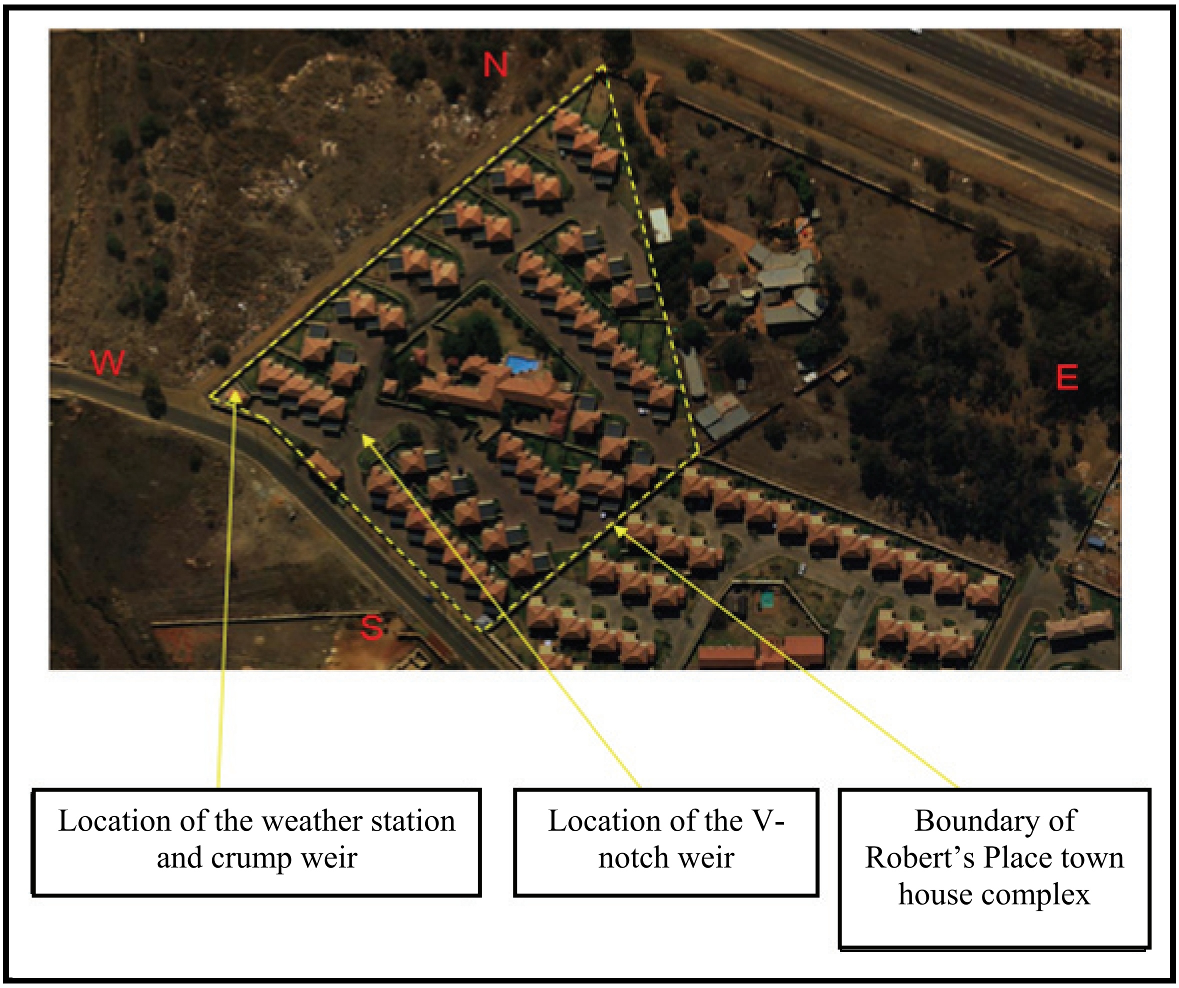


Figure 3-14: Robert's Place and the locations of all the stations

(i) *Zone A*

Figure 3-15 and **Figure 3-16** reflect the Crump weir and the flow gauging equipment that was installed for the runoff from Zone A located at the South-Western end of the Robert's Place town house complex. The water level at the Crump weir was recorded by two pressure transducers and stored on a Hobo (12 Bit) data recorder.

Figure 3-15: Crump weir to measure the flow from Zone A

Figure 3-16: Pressure transducers to measure the flow depth upstream from the Crump weir

The water level upstream from the Crump weir was measured by the two pressure recorders which were installed on a pipe that was open to the upstream side of the Crump weir.

The two pressure transducers which in turn were connected to a Hobo (data logger) and a 12V battery that logged the pressures generated by the stormwater passing over the crump weir when discharge across the Crump weir occurs. Greenline software was used to extract the recorded data from the data logger. The logger was set to an average logging interval of 10 seconds, allowing a record length of two days.

Equation 3-1 and **Equation 3-2** were used to calculate flow across the crump weir (Chadwick, Morfett and Borthwick, 2004, PP 440).

$$Q = C_d C_v \sqrt{b h_1^3}$$

Equation 3-1

$$C_v = \frac{h_1}{h_1 + P_s}$$

Equation 3-2

Where:

Q	=	Peak flow (m^3/s)
Q_{ideal}	=	Ideal flow through a hydraulic structure [Q/C_d] (m^3/s)
h_1	=	Depth of upstream water level (m)
h_p	=	Head at the second gauging point (m)
C_d	=	Discharge coefficient of 0,63(dimension less)
C_v	=	Variation coefficient (dimension less)
P_s	=	Crest height of crump weir (m)
b	=	Length of weir (m)

Figure 3-17 shows the casing for the data logger that is accessible for taking read outs and keep the probe safe from weather and traffic damage.

Figure 3-17: Casing for data logger

(ii) *Zone B*

Runoff from Zone B was measured near the entrance to the complex, were there was a rectangular stormwater inlet covered by a steel grid. This served as a good location to fit a V-notch weir where the water level could be recorded. **Figure 3-18, Figure 3-19** and

Figure 3-20 reflect the inlet to the stormwater system where the V-notch was installed and the gutter that diverts all the inflow to the grid inlet to the upstream side of the V-notch.

Figure 3-18: View of the drive way looking north of the V-notch

Figure 3-19: View to the East of the V-notch under construction

Figure 3-20: View of the entrance of Robert's Place from the inside

Figure 3-21 provides an upstream view of the V-notch weir, while **Figure 3-22** reflects the installed gutter to ensure that all the discharge passes through the V-notch structure.

Figure 3-21: The V-notch from the up-stream side

Figure 3-22: Gutter that routes stormwater upstream of the V-notch

Figure 3-23 indicates the dimensions of the V-notch. These dimensions together with the processed pressure data is ultimately used in the formula below to calculate the discharge through the V-notch. The flow that is measured in m^3/s is the runoff in the applicable area of the site. Below is the V-notch **Equation 3-3** together with **Figure 3-23** to show the dimensions of the V-notch (Chadwick, Morfett and Borthwick, 2004:440).

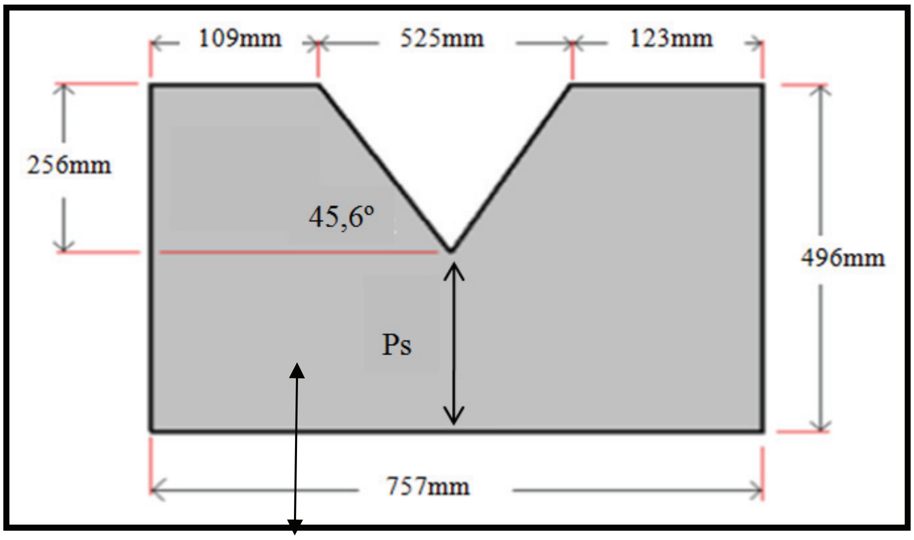


Figure 3-23: The size and upstream view of the V-notch

$$Q_{ideal} = \frac{8}{14}\sqrt{2g}tan\left(\frac{\theta}{2}\right)h_1^{\frac{5}{2}}$$

Equation 3-3

Equation 3-3 is transformed to **Equation 3-4** to take the effect of contraction into account.

$$Q = C_d\frac{8}{14}\sqrt{2g}tan\left(\frac{\theta}{2}\right)h_1^{\frac{5}{2}}$$

Equation 3-4

Where:

Q	=	Peak flow (m^3/s)
Q_{ideal}	=	Ideal flow through a hydraulic structure [Q/C_d] (m^3/s)
h_1	=	Depth of upstream water level above the lowest point of the V (m)
C_d	=	Discharge coefficient, 0,6 (dimensionless)
θ	=	Theta, opening of the V in the V-notch (degrees)

3.6 Rainfall gauging stations

3.6.1 Catchment 1: Willowspruit – Tshwane

Due to the loss of the historical autographical recorded rainfall data at the rain gauging stations installed by Tshwane Metropolitan Council in Catchment 1, it was decided that rainfall in this catchment will be obtained from rain gauges operated by the Weather Services. The rainfall data was obtained from the South African Weather Service (SAWS) for the pre- and post-development periods for which runoff data was available. The rainfall station located on the boundary of the Willowspruit catchment area is called Ramkie-oor-Willows (0513556 4). **This station only measures daily rainfall**. The nearest stations (outside of the catchment boundaries), measuring daily rainfall as well as hourly rainfall (intensity), are Pretoria Eendracht (0513314C9) and Irene WO (0513385A2). These stations are within a radius of approximately 18 km from the outlet point of the catchment. The rainfall stations' relative positions are illustrated on the map in **Figure 3-24** and their coordinates are provided in **Table 3-4**. It was assumed that similar storm event characteristics identified from this data holds true for the catchment area under consideration as no other rainfall data was available.

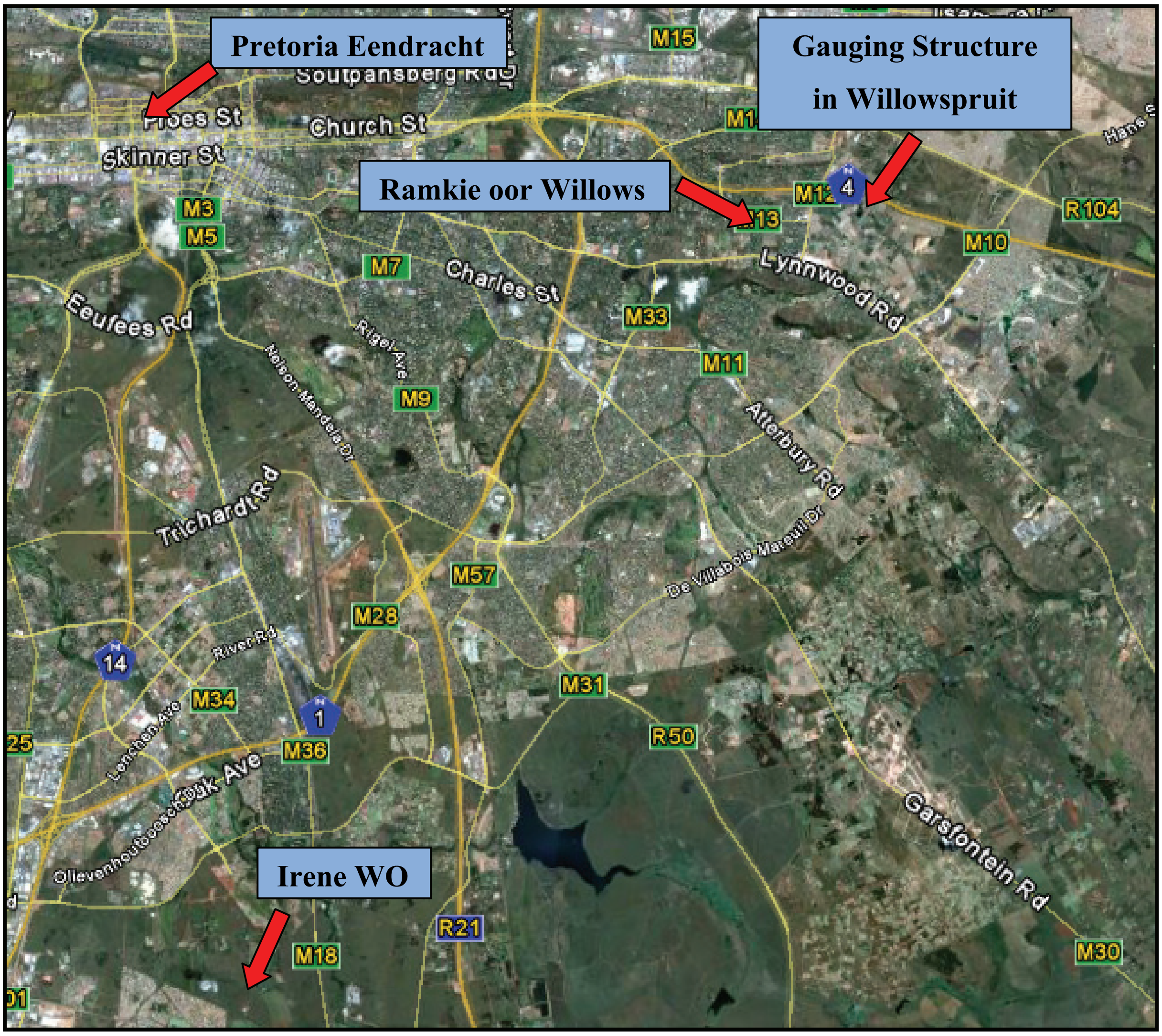


Figure 3-24: Relative positions of the SAWS rainfall stations to the weir in Willowspruit

Table 3-4: Location and capability of rainfall stations utilized in this study

Rainfall station	**Location (latitude; longitude; altitude)**	**Data measured**	
		Daily rainfall (mm)	Rainfall intensity (mm/hr)
Ramkie-oor-Willows (0513556 4)	-25,7600; 28,3130; 1378 m	•	
Pretoria Eendracht (0513314C9)	-25,7420; 28,1830; 1308 m	•	•
Irene WO (0513385A2)	-25,9100; 28,2110; 1526 m	•	•
Robert's Place *	-25,7544; 28,3349	•	•

Note:

* installed for the research project to be used for Catchment 2

3.6.2 Catchment 2: Robert's Place – Tshwane

The additional rainfall station erected near the catchment outlet, at Robert's Place, was used to portray the size of the storm over the catchment. This data is only available for the 2010 rainy season. The weather station installed at Robert's Place is an Oregon Scientific Professional Weather Centre (WMR200). The set-up of the weather station can be seen in **Figure 3-25** and **Figure 3-26**. The measuring devices are connected to a data logger via remote sensing.

Figure 3-25: Data recorder and transmitter for catchment 1

Figure 3-26: WMR200 weather station located near the outlet of the catchment

The data is saved in a log file during extraction from the data logger and was converted into Microsoft Excel using a data conversion program. The code of the conversion program is included in **Appendix C**. The time interval of data recording was 1 min and the measurements had a resolution of 10^{-1} mm.

3.7 Establishing of the rainfall intensities

3.7.1 Willowspruit (Catchment 1)

Daily rainfall from the Ramkie-oor-Willows weather station was used in conjunction with recorded hydrographs to estimate the rainfall intensity. The hydrographs were used to find an indication of the time over which the downpour occurred. The recorded hydrographs were used to identifying the increasing segments **Figure 3-27**.

<u>Cognisance must be taken that this method for calculating the intensity from the time periods of the rising sections in the recorded runoff hydrograph has not been calibrated nor verified. This method to calculate the rainfall intensity was used for all of the analyses and comparisons of Willowspruit (Catchment 1). The method was deemed to be acceptable for comparative trends in the runoff response due to the relative intensities of the storm events.</u>

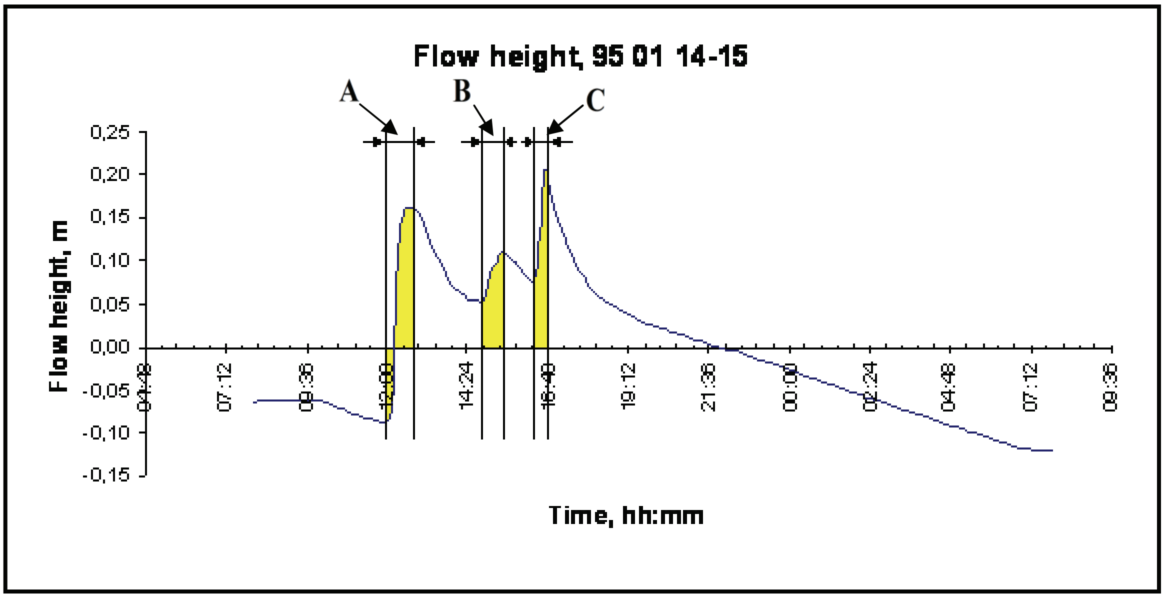


Figure 3-27: Recorded flow depths used to identify the rising portions (A, B and C) of the hydrograph

Information as read from the figure above is included in **Table 3-5**. The intensities of the storm events were calculated by dividing the daily rainfall measured, at the Ramkie-oor-Willows weather station, by the time of the rising legs of hydrograph shown in **Table 3-5**.

Table 3-5: Information of hydrograph rising legs used to calculate rainfall intensity

Recording on 1995/01/14	Period	Start	End	
Δt_{rise}	A	12:00	12:48	48 min
	B	14:48	15:30	42 min
	C	16:24	16:42	18 min
	Total rising period			108 min
				1,8 hr
Daily rainfall				12 mm
Calculated intensity, I				6,67 mm/hr

3.7.2 Catchment 2: Robert's Place – Tshwane (Erf 683 and 685, Equestria 137)

No historic data is available and in this case the rainfall and outflow from the catchment will be recorded for a number of events.

The rain gauge and data recorder-transmitter are reflected in **Figure 3-28** and **Figure 3-29**.

Figure 3-28: Location of autographic rain meter on the south eastern side of the site

Figure 3-29: Installation of the data recorder and transmitter

3.8 Data analysis for Willowspruit (Catchment 1)

Pre and post development levels in urban areas were determined to assess the effect of urban development on peak runoff for compatible storm events. The level of development was determined from aerial surveys conducted by the Tshwane Metropolitan Council.

A similar analysis for larger catchments was undertaken, considering the amount of rainfall (**Section 6)** for different hydrological years.

One of the dominant parameters in rainfall/runoff modelling is the rainfall intensity and volume of discharge. In the assessment of Catchment 1, similar rainstorms that occurred during the pre and post development periods (1993-1995 and 2010) were selected and compared. The selected rainstorms for the analyses need to be of near similar intensity and duration. The use of daily rainfall was to be able to identify storm events, potentially having similar characteristics (Dyson, 2010). After these similar events have been identified (according to daily rainfall), the duration of rainfall were compared. The investigation of hourly rainfall is necessary to determine whether the storm event was of sufficient duration for the total catchment area to contribute to the discharge, as indicated by Dyson (2010). It is important to be aware of successive storm events as this could influence the soil moisture conditions and the resulting flow response measured.

The rainfall data obtained from the SAWS was processed and utilized to identify rainfall events suitable for comparative purposes. Rainfall events were extracted from the daily rainfall data for the rainfall station located in the Willowspruit catchment, Ramkie-oor-Willows. Storm events in the catchment were categorised according to volumetric similarity as suggested by Dyson (2010). The data was grouped in intervals of 10 mm, as illustrated in **Table 3-6** for both analysis periods. Through visual inspection, volumetrically comparable rainfall events (in both the pre- and post development periods) were grouped together and colour coded as seen in **Table 3-6**. Only daily rainfall events for 2010 were extracted into this report as example, rainfall information for Ramkie-oor-Willows, Irene WO and Pretoria Eendracht are included in **Appendix D**.

Table 3-6: Volumetrically categorised rainfall events in 2010 of Ramkie-oor-Willows

	Volumetric category for daily rainfall			
Date	**<10 mm**	**<20 mm**	**<30 mm**	**<40 mm**
2010/01/07	5,0			
2010/01/13		13,0		
2010/01/19	8,4			
2010/01/20	2,0			
2010/01/21	2,0			
2010/01/23	8,5			
2010/01/25	7,0			
2010/01/26	2,0			
2010/01/29	6,0			
2010/02/04		14,0		
2010/02/16				30,3
2010/02/17			26,5	
2010/02/25	8,5			
2010/02/26	6,3			
2010/03/02		11,4		
2010/03/05			29,5	
2010/03/14	4,0			
2010/03/15	6,0			

The volumetrically similar storm events identified were further compared by evaluating the intensity of rainfall (hourly rainfall) recorded at the nearest appropriate rainfall stations. The

Pretoria Eendracht and Irene WO stations are the closest stations recording rainfall intensity (hourly rainfall). These stations were used to determine other storm characteristics, including the approximate extent of the storm over the catchment. The rainfall data for March and April 2010 was not available for the Pretoria Eendracht, due to data collection problems. The daily rainfall is measured at 08:00 am each day, while the intensity data applicable to a specific day includes the hourly rainfall measured from the 9th hour of the applicable date until the 8th hour of the following day. The intensity of three historical rainstorm events, identified as having similar volumetric size, is illustrated in **Figure 3-30** to **Figure 3-32**. The variability in storm pattern can be clearly seen when comparing these figures.

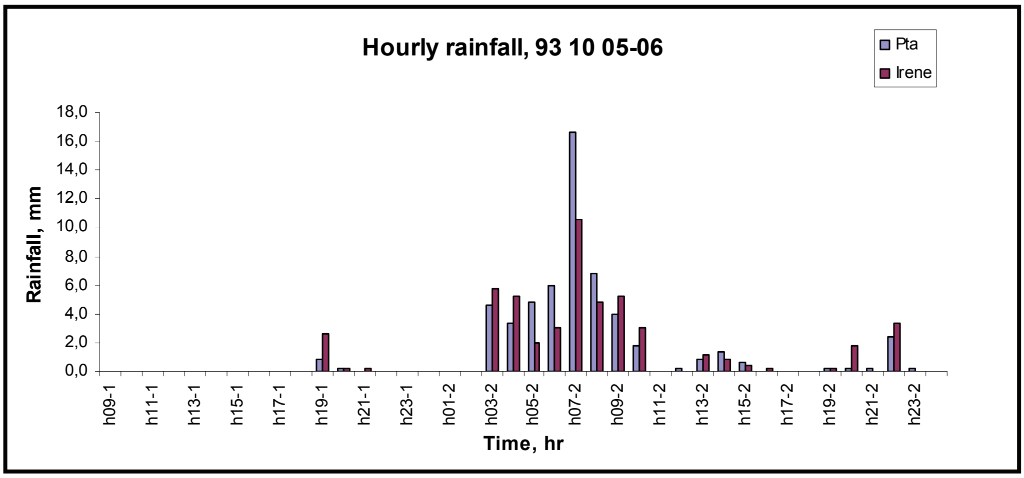


Figure 3-30: Rainfall intensity measured at Pretoria Eendracht and Irene WO, from the 9th hour, on 5 October 93, to the 24th hour on 6 October 1993. Volumetric size of 44 mm.

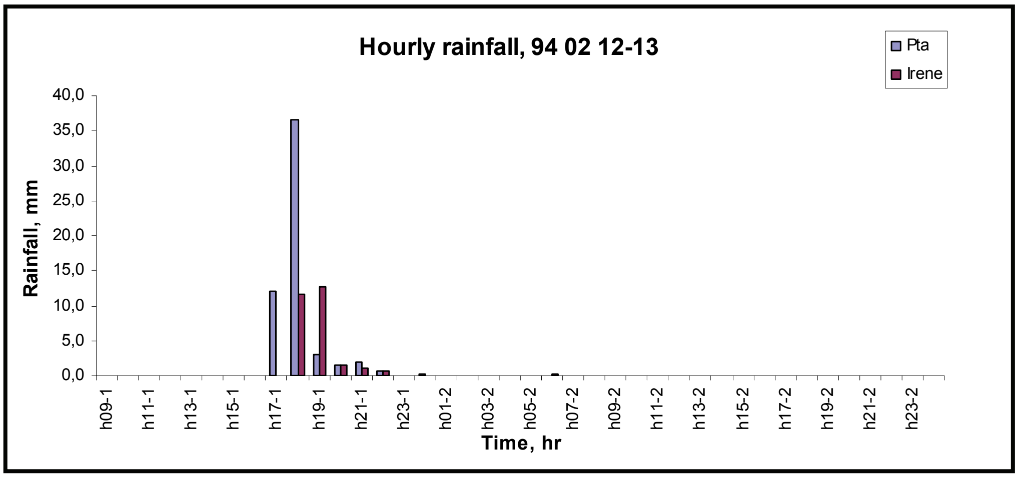


Figure 3-31: Rainfall intensity measured at Pretoria Eendracht and Irene WO, from the 9th hour, on 12 February 94, to the 24th hour on 13 February 1994. Volumetric size of 45 mm.

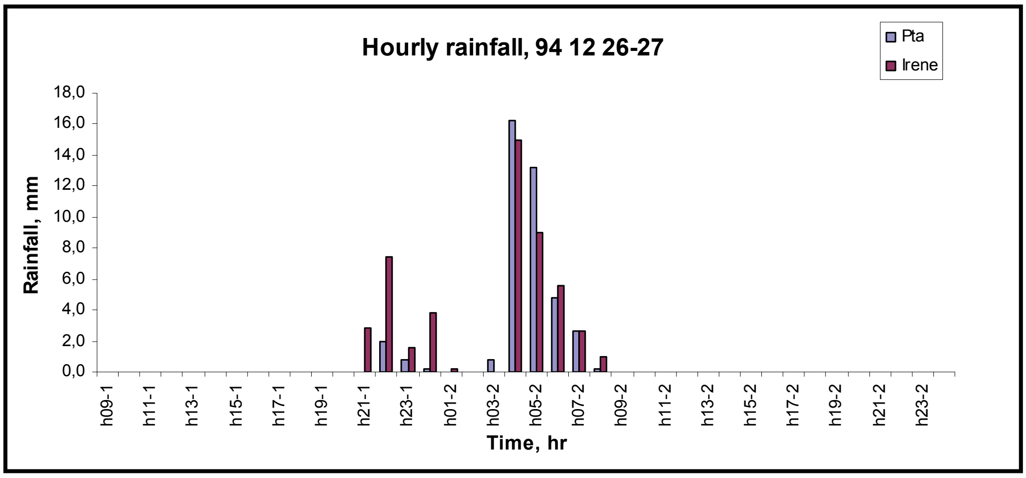


Figure 3-32: Rainfall intensity measured at Pretoria Eendracht and Irene WO, from the 9th hour, on 26 December 94, to the 24th hour on 27 December 1994. Volumetric size of 43 mm.

More weight was given to the depiction of the storm characteristics at the Pretoria Eendracht station, because of the proximity (approximately 15,4 km from the catchment centre). The Irene WO station is approximately 20 km from the catchment centre and its elevation is 148 m higher than the Ramkie-oor-Willows station. The antecedent soil moisture conditions could be considered by comparing events from the same month, thus the same time in the rainfall season.

Parameters which were not individually considered during the analysis include climatic factors, such as temperature, that could affect the amount of evaporation from the area, and seasonal vegetation that could affect the interception of surface runoff. It was assumed that the rainfall is evenly distributed over the whole catchment. The 16 year period between pre- and current development periods under consideration is assumed to be short, minimising the possible climatic changes and the influence thereof.

The runoff produced by the catchment after a specific rainstorm identified, is evaluated for pre- and current development conditions. The evaluation was conducted, comparing the measured flow data from the two periods. The relative change in flow is then compared to the expected change in the peak runoff as determined by the Rational Method's runoff coefficient. This method is widely used in calculating expected flood peaks, because of its simplicity. By evaluating the expected change in runoff, in comparison to the actual runoff produced, possible conclusions could be drawn regarding the applied runoff coefficient (C).

3.9 Modelling of discharge from Robert's Place (Catchment 2)

3.9.1 Methodology

The objective of this investigation for a highly developed catchment was to compare the discharge from the catchment with what would have been obtained from a detailed modelling of the catchment by using EPA SWMM. The weather station recorded individual storm events and the discharge from Zone A and B was calculated from the recorded data at the gauging structures.

A detailed EPA SWMM model was set up to model the discharge rate and volume of discharge. A survey was done on the townhouse complex and detailed measurements were taken to compile a very comprehensive and precise model of the townhouse complex.

The two separate flow gauging stations in Zone A and B were used to record the flow of the stormwater during and after the rain event. The weather station was used to collect rainfall volume and intensity. This data was then processed and compared to the results of the detailed EPA SWMM model.

3.9.2 Event data acquisition for comparison

Recorded rainfall was used as input to the simulation model for the two zones (A and B) The detailed EPA SWMM simulation was run and the output data, discharge, was compared with the discharge data acquired from the two gauging stations (see **Section 3.5.2**).

3.9.3 EPA SWMM Software and setting up the model for Robert's Place

3.9.3.1 Introduction

The public domain software, EPA SWMM, was used to generate the discharge of the town house complex for different rainfall events. The generated EPA SWMM data was compared to the measured data. EPA SWMM is a dynamic runoff-simulator used to determine the quantity and quality of runoff and volumes of runoff over both single events and continuous simulations. The study required a simulation with single rainfall events, quantity and volume runoff generated from the program. Runoff generated by EPA SWMM consists of multiple sub catchments that receive precipitation and produces runoff. EPA SWMM functions on the basis of mass continuity and can handle channels, pumps, pipes, regulators and storage devices. EPA SWMM also hosts other modelling features which were not used for this comparison.

The EPA SWMM software includes different hydraulic processes which could be used to generate runoff from urban development, these include:

- Time varying rainfall;
- Melting and snow accumulation;
- Rainfall interception from depression storage;
- Soil (unsaturated) infiltration;
- Reservoir routing; and
- Overland flow.

3.9.3.2 EPA SWMM Model features

Several environmental compartments are used to conceptualize a drainage system, comprising of atmospheric- and land-surface compartments. The atmospheric compartment is using rainfall input (rain gauge) and the land surface compartment consists of infiltration, surface water storage and surface runoff components. Ground water compartment receives infiltration from the land surface and transfers a portion of the ground water to the transfer compartment, where the transfer compartment then conveys it via various elements (pipes channels pumps and regulators). **Figure 3-33** shows a picture of the EPA SWMM 5 software's main window.

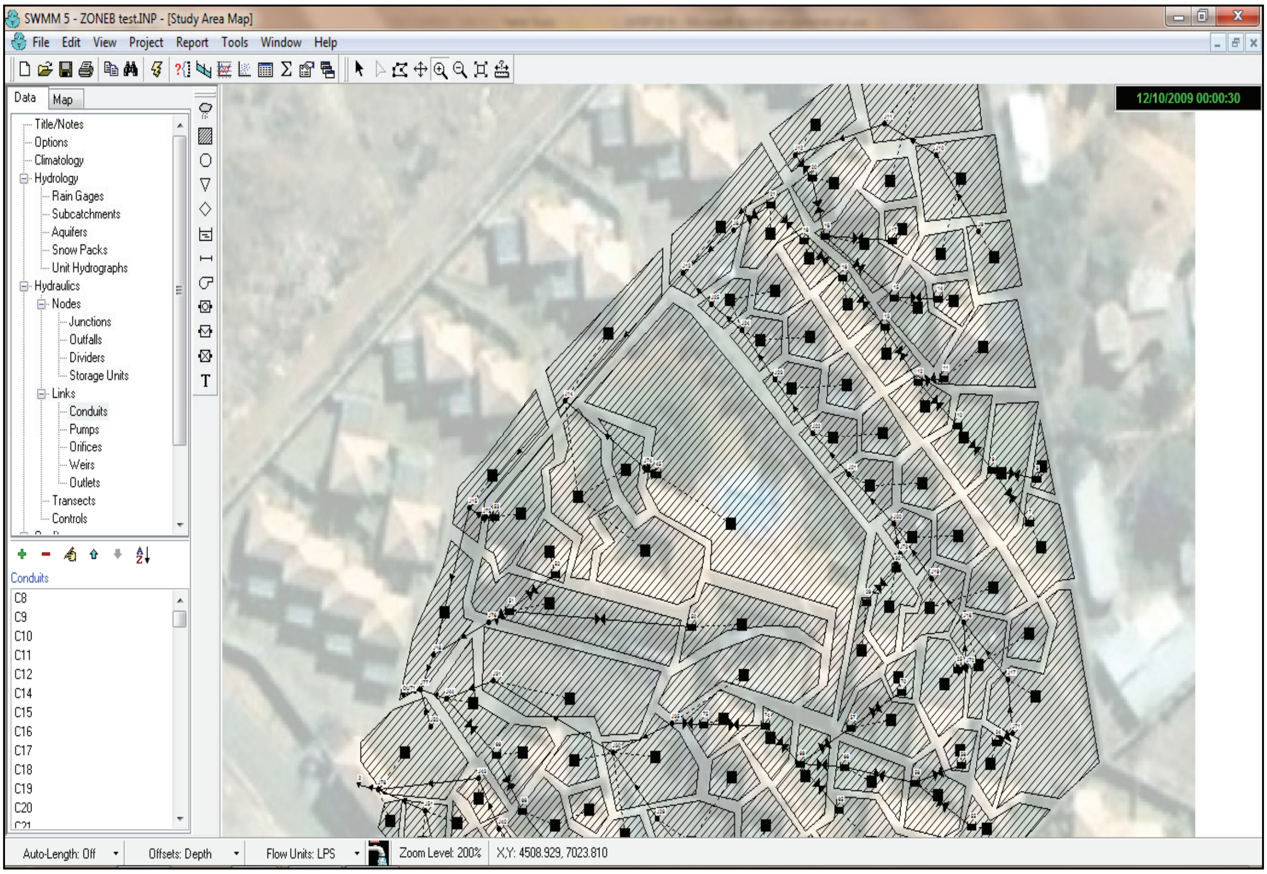
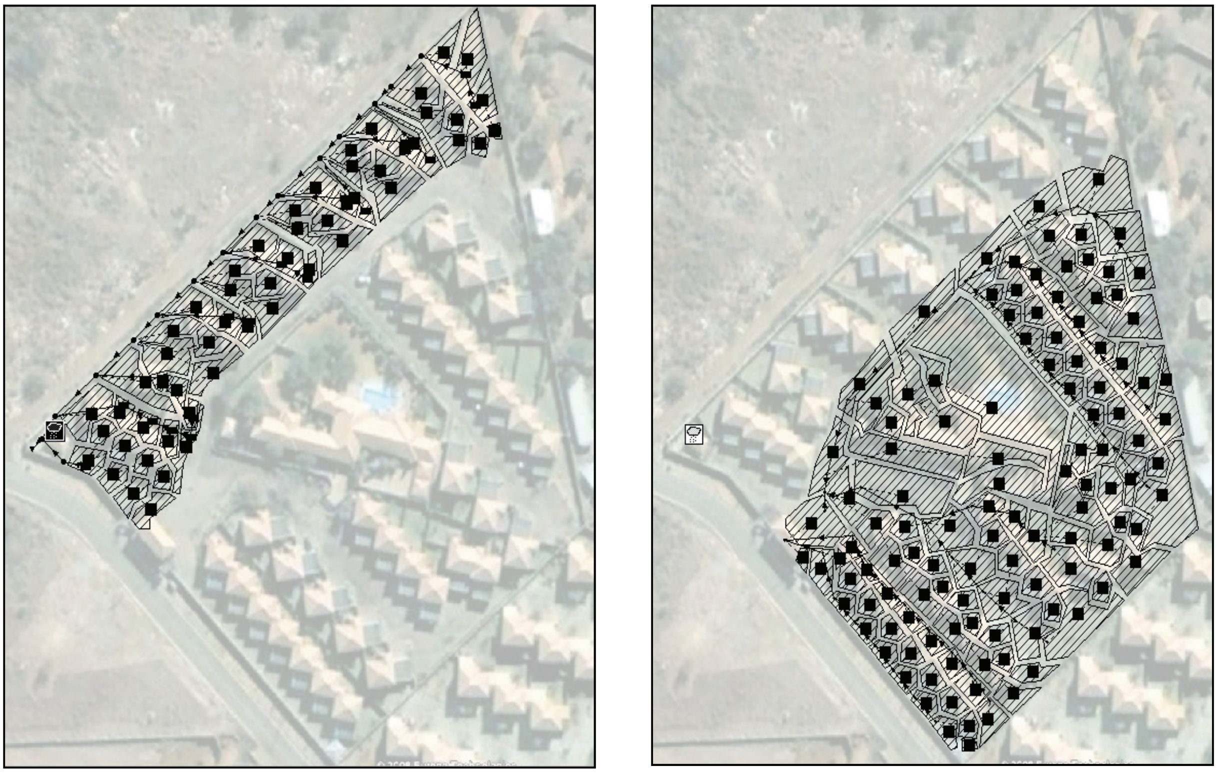


Figure 3-33: Main window view of the EPA SWMM 5 software (Zone B)

3.9.4 Data processing and selection of events for analyses for Catchment 2

3.9.4.1 Subdivision of Catchment 2: Robert's Place

The Robert's Place site was divided into Zone A and Zone B. Zone A comprising the north western area of the residential townhouse complex and Zone B the south easterly side. A Google Earth image reflects the co-ordinates; latitude 25º45'15.88"S and longitude 28º20'8.18"E. Previous studies (Lee and Heaney, 2003) showed that an accurate survey of all the identifiable coverage needs to be identified and surveyed to ensure an accurate micro scale storm models. The detailed surveyed data was used to create the model for Zone A and Zone B, as seen in **Figure 3-34**.



Zone A – EPA SWMM Model

Zone B – EPA SWMM Model

Figure 3-34: Layout of the zones for the Robert's Place catchment.

3.9.4.2 Review of Recorded data for events at Robert's Place

All the data is available in electronic tabular format in **Appendix E**.

The rainfall and recorded flow record for the period from December 2009 to April 2010 is reflected in **Figure 3-35** and **Figure 3-36**

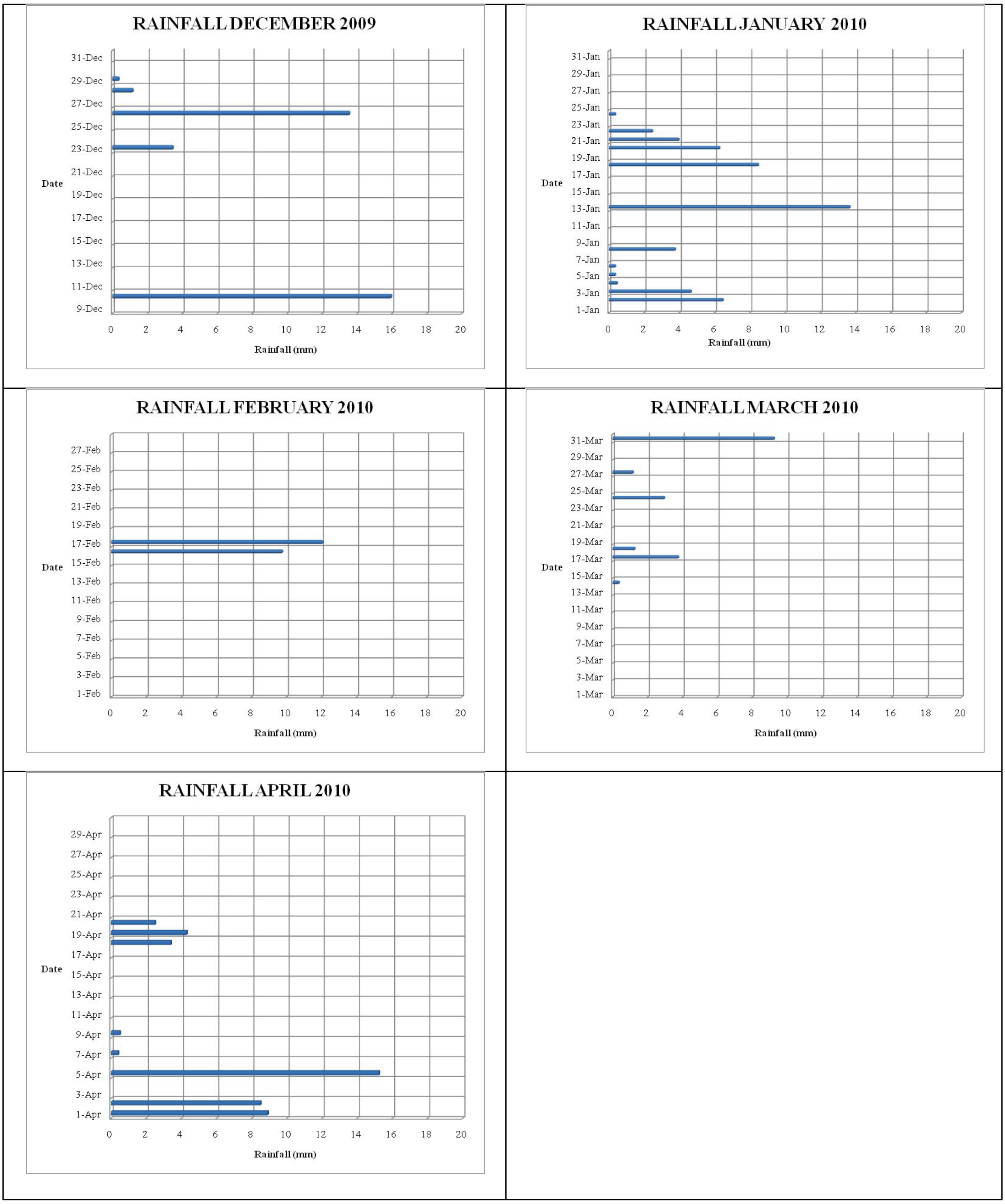


Figure 3-35: Visual representation of the rainfall events during December 2009 to April 2010 at Robert's Place

The solid horizontal lines in **Figure 3-36** represent the times when reliable data was recorded. The selected rainfall events which were selected for the review and modelling are illustrated by vertical lines. All the data elements (rainfall and runoff) were reliable and complete for the selected events.

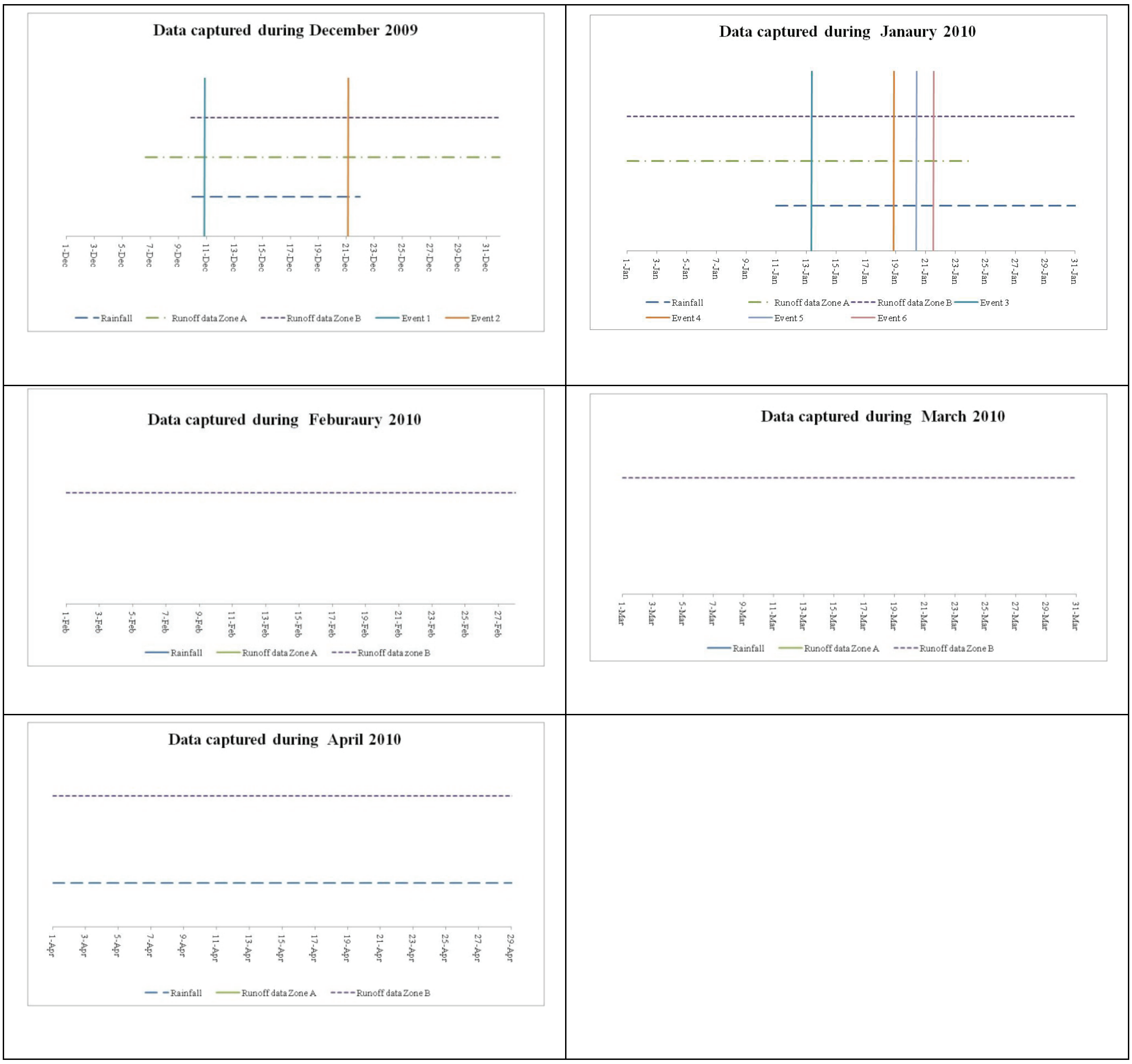


Figure 3-36: Visual representation of the data sets and the selected events which were evaluated

4 Review of the Willowspruit catchment

4.1 General

In the following two sections, the results obtained for the two catchments (Catchment 1 – Willowspruit (**Section** 4**)** and Catchment 2 the densely developed town house security complex **(Section 5)**). The comparison of the recorded and calculated peak discharge from the catchments is discussed. **Section 6** reflects the investigation of the effect of urban development on runoff response from similar hydrological years based on a mass balance approach for larger catchments.

4.2 Introduction

The runoff response from the catchment, under pre- and current development conditions, is reflected in this section. The aim of this section is to determine the change in runoff response on a comparative basis for similar storm events during the pre- and post development phases of the catchment. The periods selected for comparison for catchment 1 are 1993-1995, for the pre-development condition, and 2010 for the current (post) development condition for catchment 1.

Firstly, the volume of runoff is considered in **Section 4.3**. Next, storm events that are similar based on rainfall volume, peak flows are compared (**Section 4.4**). Finally, in **Section 4.5**, similar storm events based on the downpour volume and estimated intensity are compared.

4.3 Volume of runoff

It is expected that the volume of runoff increases with an increase in development (Van Dijk and Van Vuuren, 2006). In an attempt to compare the volume of runoff produced in the catchment, the calculated hydrographs were reviewed. Volume of runoff was calculated as specified in **Section 3.8**. The results were plotted and linear trend lines drawn as illustrated in **Figure 4-1.**

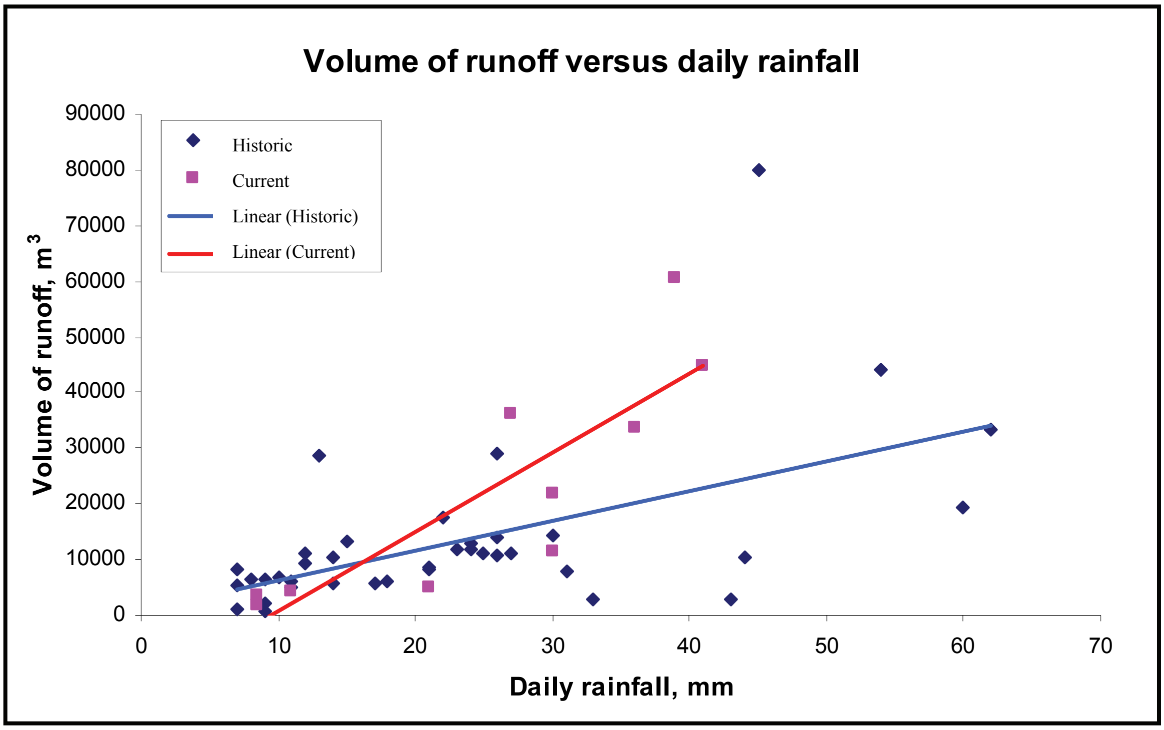


Figure 4-1: Volume of runoff calculated as resulting from daily rainfall events

The trend lines indicate that the larger daily rainfall events did produce more volume of runoff. The amount of data available for post development conditions reduces the accuracy of the trend lines. The trends depicted in **Figure 4-1** are in accordance with the view of increased volume of runoff as a result of less infiltration and reduced interception by vegetation (Semadeni-Davies et al., 2008; Blöschl et al., 2007). The problem with the comparison however is that 2009/2010 was a fairly wet hydrological year and that the recorded data only covers the latter part of the hydrological year.

4.4 Volumetrically similar storm events

The approach followed is as suggested in **Section 3.8**. Volumetrically similar storm events were identified and the intensity of the storm events visually inspected. The hydrographs were compared based on the rainfall measurements at the Pretoria Eendracht and Irene WO weather stations. A storm event was only considered if the length of the storm was longer than the time of concentration (T_C) for the Willowspruit catchment (as calculated in **Section 3.2**). It was assumed that the storm was of sufficient length over the catchment area under consideration if the length was sufficient at both the Pretoria Eendracht and Irene WO stations. It was further assumed that entries at both these stations indicate a regional storm event and not just a localised event as regularly found with thunderstorms (Dyson L., 2010).

4.5 Overview of rainfall events

Figure 4-2 illustrates the maximum flow rates produced by rainfall events of certain volumetric size (daily rainfall over the catchment). The storm events identified were neither discretized with regards to the hourly distribution pattern nor the antecedent soil moisture conditions.

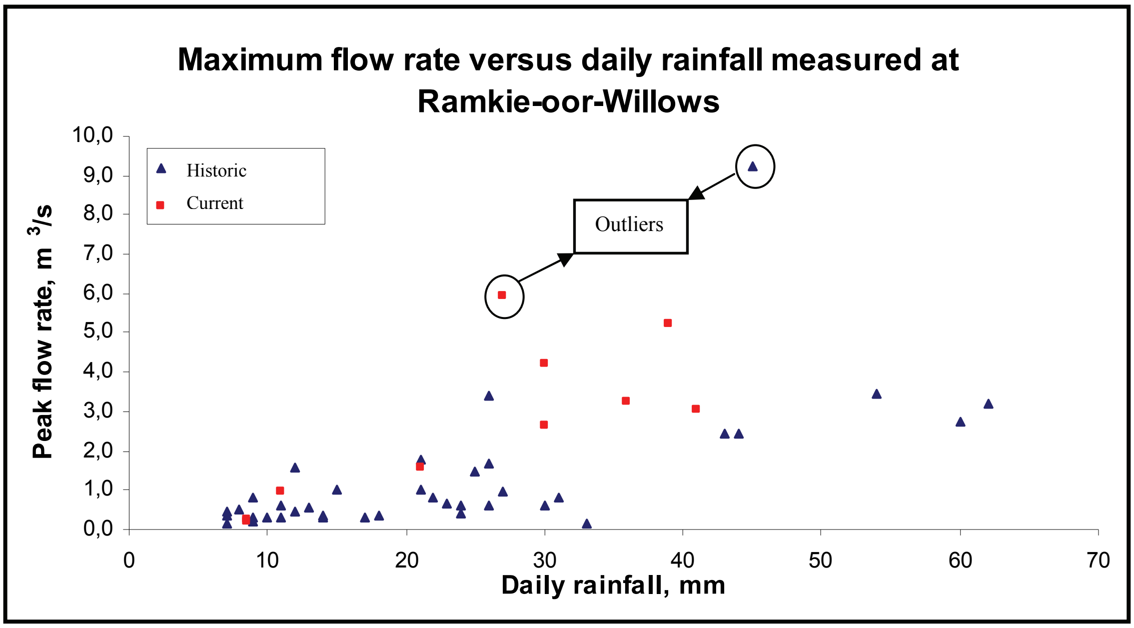


Figure 4-2: Maximum flow rate versus volumetric size of the storm event

Possible outliers were identified from the plotted data in **Figure 4-2**. These events reflected that on 12 February 1994 a 45 mm rainstorm produced a peak flow rate of 9,24 m^3/s. On 17 February 2010 a 27 mm rainstorm produced a peak flow rate of 5,93 m^3/s. By considering the available hourly rainfall data, it is evident that the events took place over a short period, thus it could be concluded that the intensities were abnormally high and it is likely that the spatial and temporal distribution was erratic. The hourly rainfall measured at Pretoria Eendracht and Irene WO is depicted in **Figure 4-3** and **Figure 4-4** for these two events.

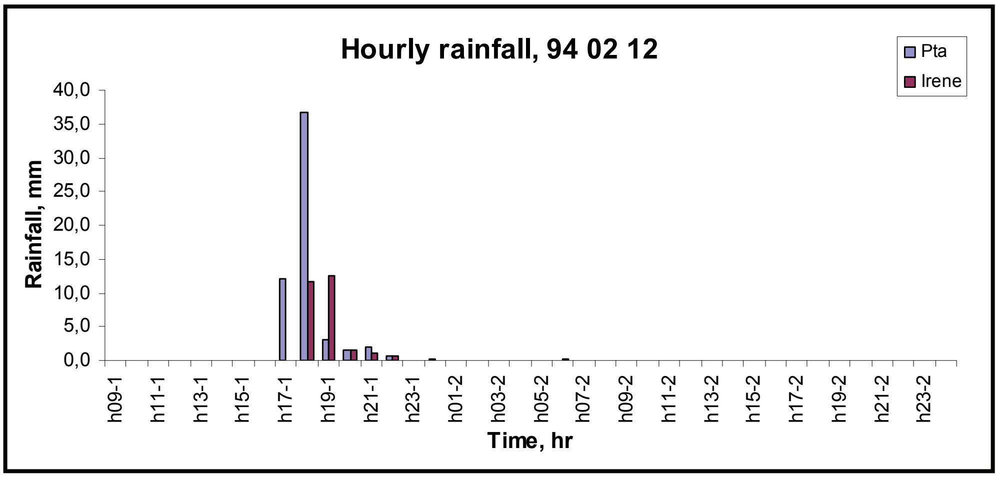


Figure 4-3: Hourly rainfall at Pretoria and Irene stations, 12 February 94

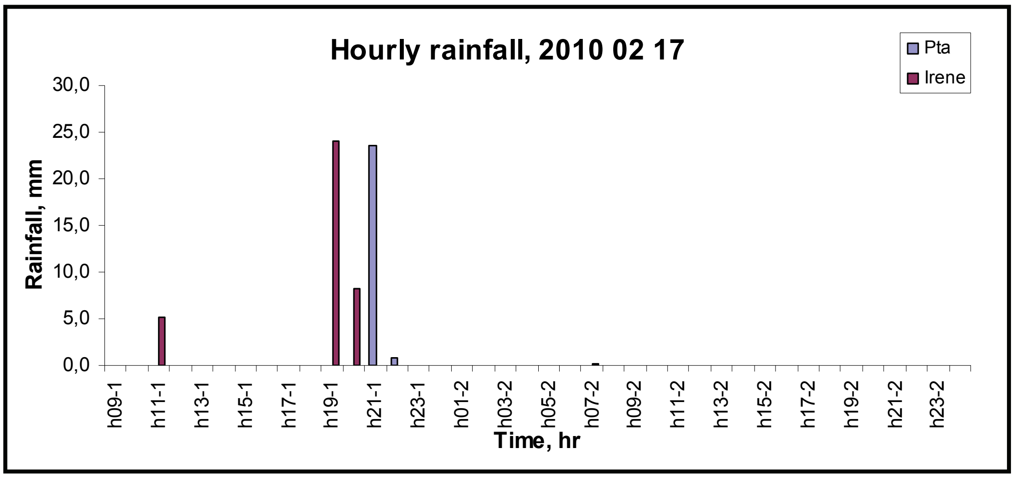


Figure 4-4: Hourly rainfall at Pretoria and Irene stations, 17 February 2010

The data was then plotted without the outliers identified above. Possible trend lines were fitted to the data to provide a comparison between the historic and current events as seen in **Figure 4-5.**

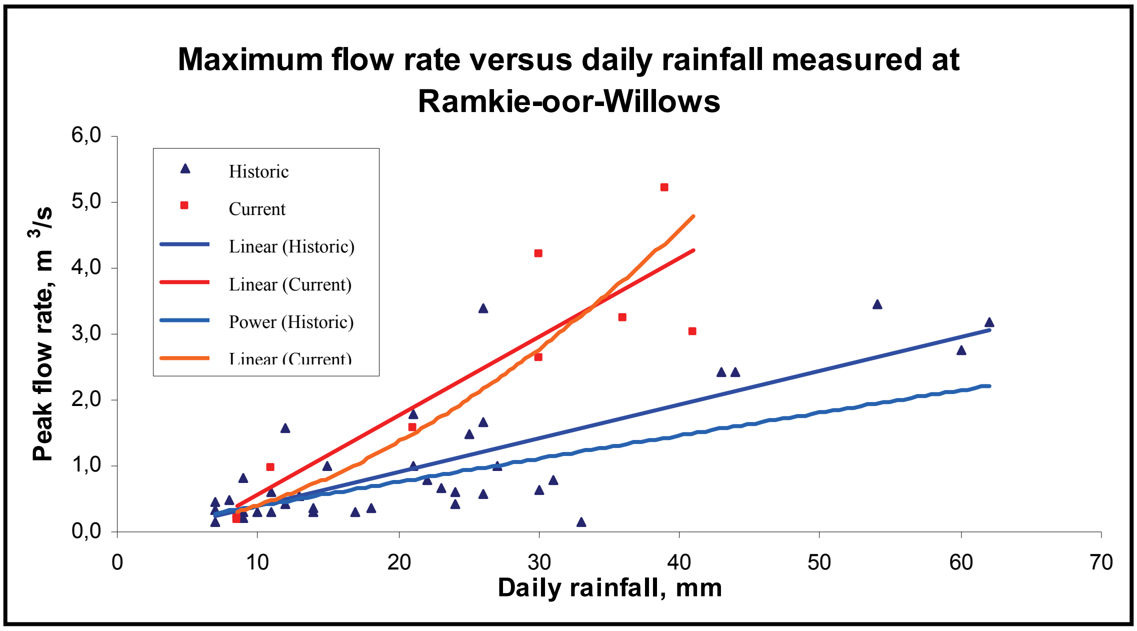


Figure 4-5: Maximum flow rate versus volumetric size of the storm event excluding outliers, with possible trend lines

The trend lines based on a limited amount of data in **Figure 4-5** indicate that the post development's peak flow rate values were higher than the pre-development results. It should however be recognised that the data is of a wetter period.

The increase in peak flow rate, as a function of rainfall, is as could be expected. An increase in volume of rainfall could mean an increase in intensity over an extended time frame resulting in the increased peak flow rate produced at the outlet point. According to Semadeni-Davies et al. (2008) it is expected that larger storm events produce peak flow rates of similar magnitude for undeveloped and developed conditions. From the data plotted above it would seem that the peak flow rates produced do not converge as the size of the storm event increases.

The variability in the storm events and the lack of rainfall intensity data within the catchment for the pre-development period makes comparison of storm events difficult and uncertain.

Two storm categories (A and B) were selected based on similar daily rainfall volumes measured at Ramkie-oor-Willows and hourly rainfall distributions at Pretoria Eendracht and Irene WO. Category A events reflect daily rainfall of between 36 and 46 mm, while category B events represent daily rainfalls of between 25 and 36 mm. It was assumed that these storm events have occurred over the whole catchment area for a period longer than the time of concentration.

4.5.1 Storm category A (daily rainfall between 36 and 46 mm)

The first storm events (category A) identified for comparison purposes are considered below. The main characteristics of the rainfall and hydrographs are indicated in **Table 4-1.** Rainfall data for Pretoria Eendracht for the March and April 2010 was not available due to technical problems experienced at the station.

Table 4-1: Main characteristics of rainfall and runoff for storm events type A

Date	Daily rainfall (mm)	Q_{MAX} (m^3/s)	Pretoria max (mm/hr)	Irene max (mm/hr)
1993/10/05	44	2,42	16,6	10,6
1994/12/26	43	2,43	16,2	15,0
2010/03/31	41	3,02	-	8,6
2010/04/05	39*	5,23	-	3,6

Note:

* Daily rainfall obtained from autographic rain meter installed at Robert's Place

If the storm events in **Table 4-1** are assumed to be comparable, even though great variability exists in the storm patterns, the peak flow rate seems higher for post development conditions.

The pre-development events delivered maximum flow rate values within 0,5% of one another. The post development's maximum flow rates were greater than the pre-development maximum flow rates. It is important to review the time of the events as the antecedent soil moisture conditions could significantly influence the runoff. For the post development events the antecedent soil moisture condition is expected to be wetter, because the events occurred later in the rainfall season and that the average rainfall in the 2009/2010 was larger than during 1993/1994.

The graphical illustrations of hourly rainfall, as well as the hydrographs, for the above storm events, are included as **Figure 4-6** to **Figure 4-13**. The time base on the rainfall histograms and the hydrographs are 36 hours, starting at 08:00 am.

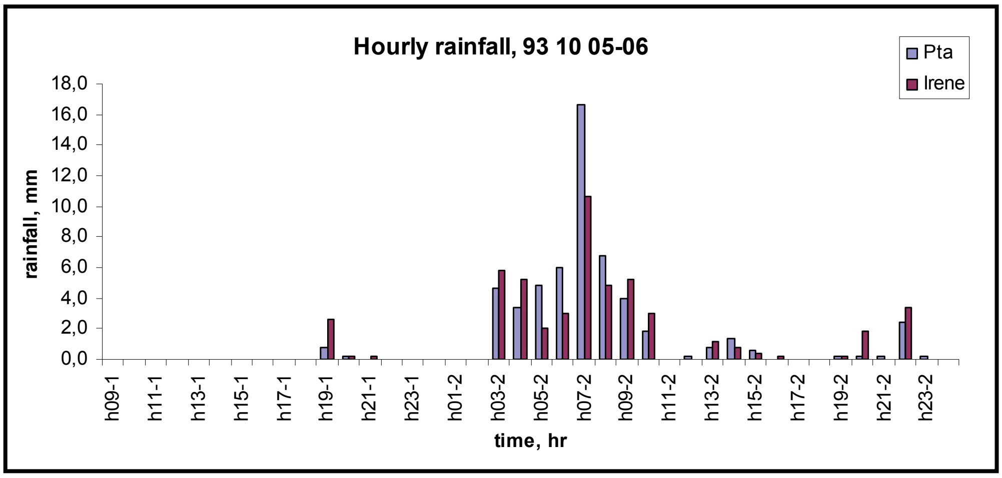


Figure 4-6: Hourly rainfall at Pretoria Eendracht and Irene stations, 05 October 1993

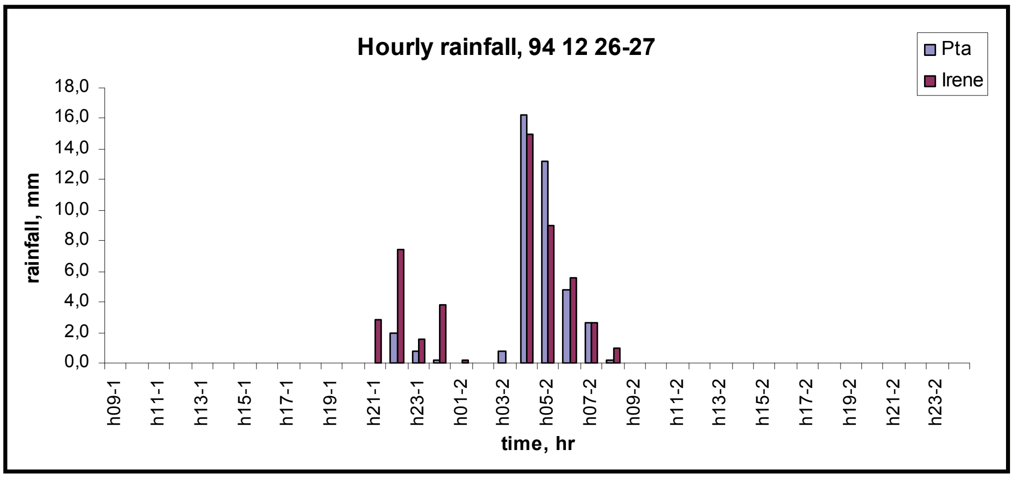


Figure 4-7: Hourly rainfall at Pretoria Eendracht and Irene stations, 26 December 1994

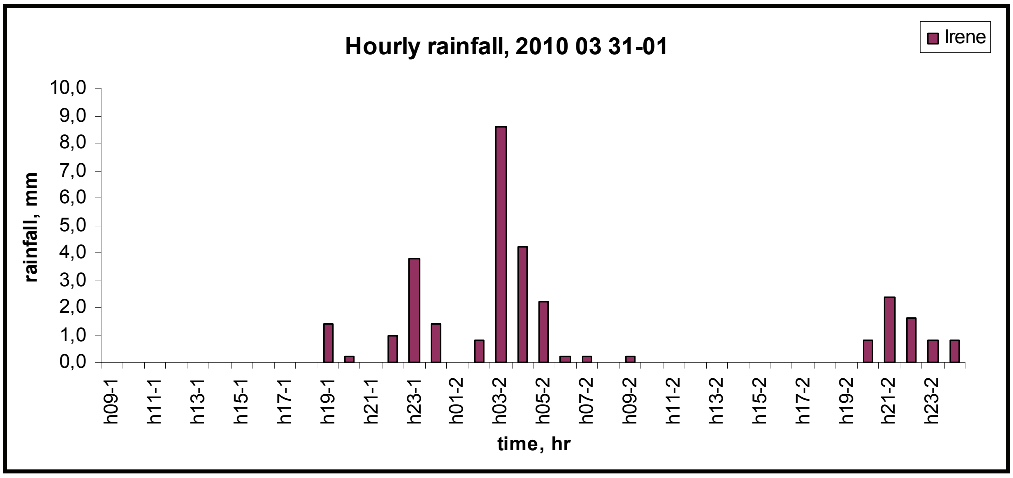


Figure 4-8: Hourly rainfall at Irene station, 31 March 2010

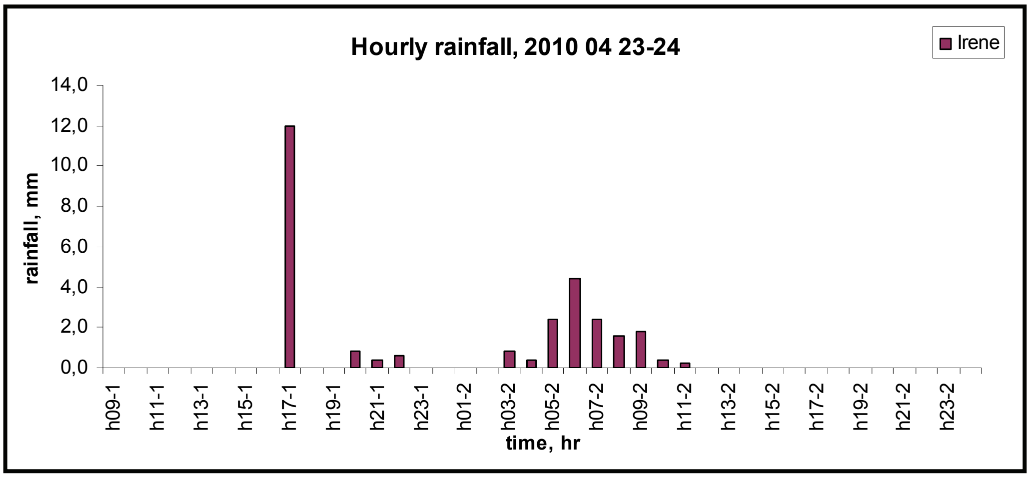


Figure 4-9: Hourly rainfall at Irene station, 05 April 2010

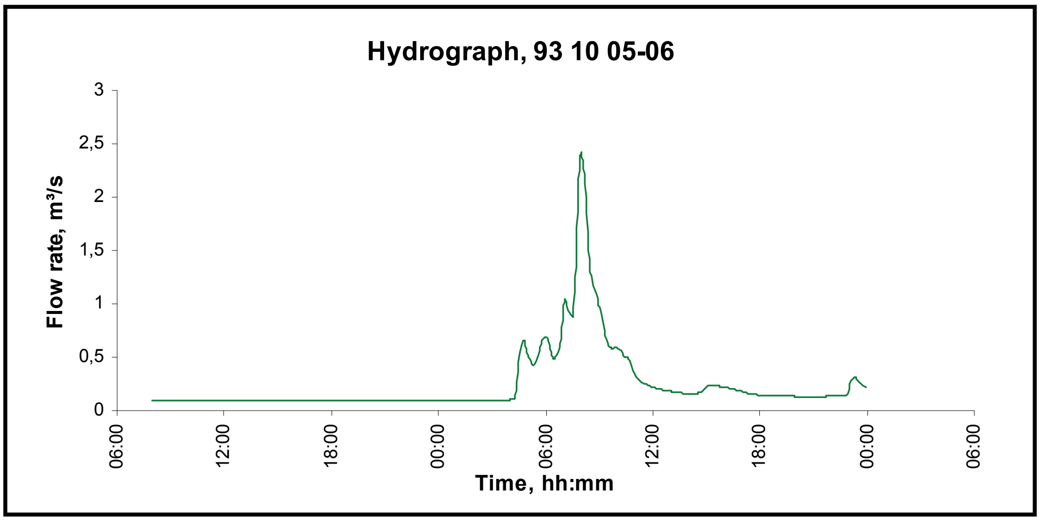


Figure 4-10: Hydrograph of 05-06 October 1993

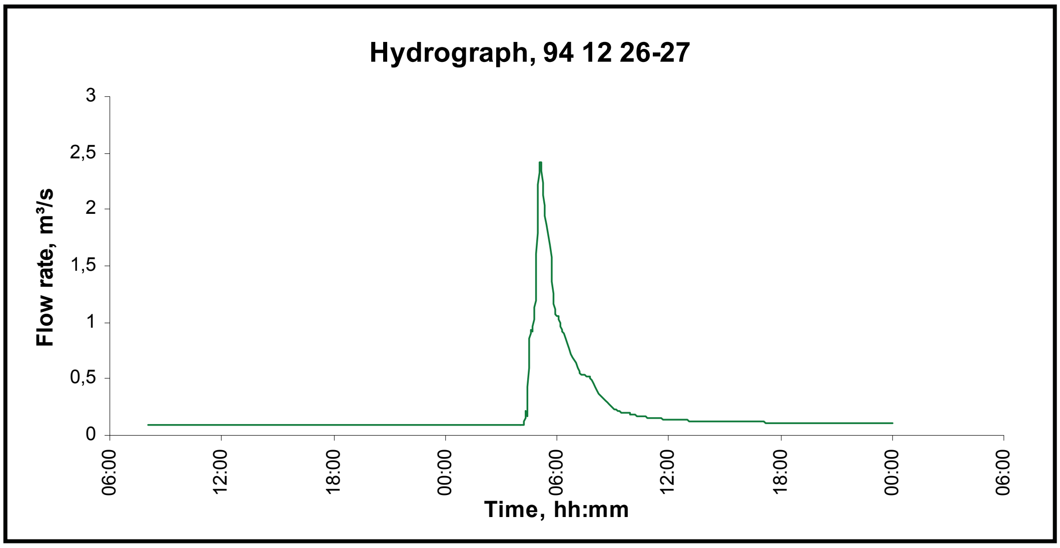


Figure 4-11: Hydrograph 26-27 December 1994

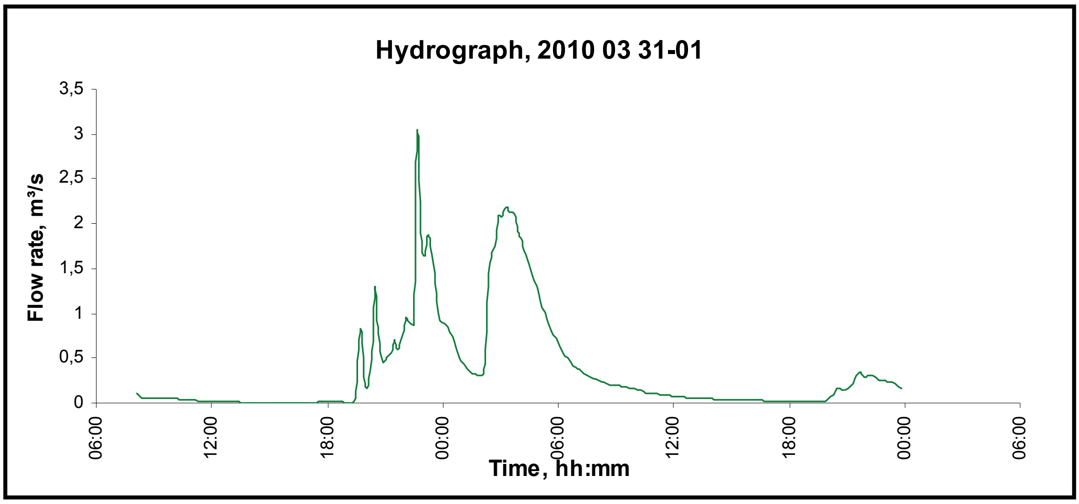


Figure 4-12: Hydrograph 31-01 April 2010

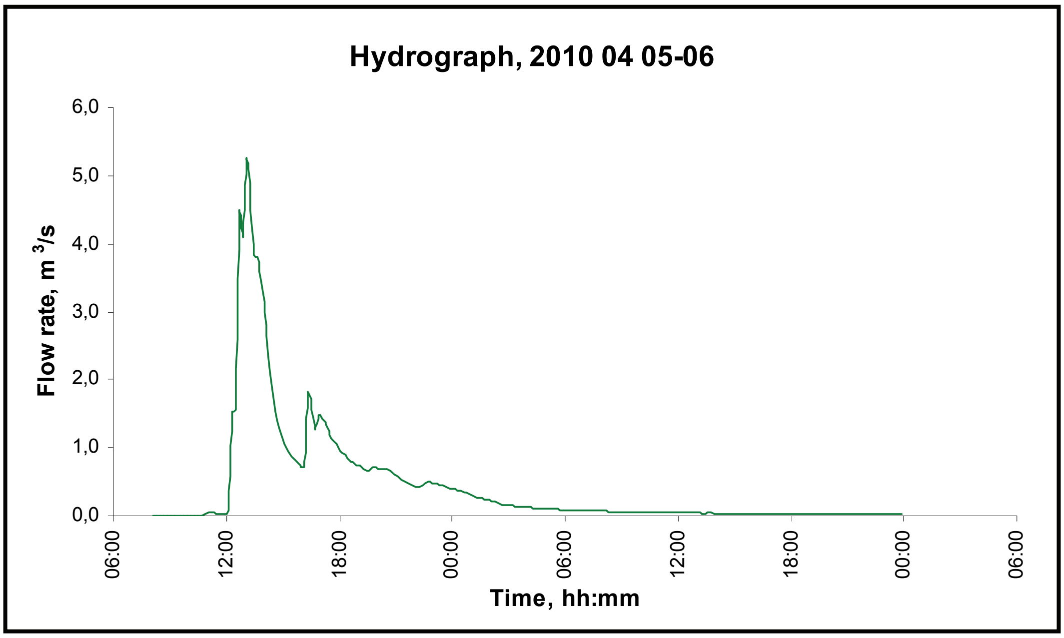


Figure 4-13: Hydrograph 05-06 April 2010

The slope of the hydrographs' decreasing leg is steeper in the pre-development response measured, which could be an indication that less detention storage is available in the catchment. This also concurs with the argument that stormwater from the developed areas is routed through the catchment.

4.5.2 Storm category B (daily rainfall between 25 and 36 mm)

The maximum discharge from storm category B events shows great variability. **Table 4-2** indicates the main characteristics of the rainfall and runoff for the storm events type B.

Table 4-2: Main characteristics of rainfall and runoff for storm events type B

Date	Daily rainfall (mm)	Q_{MAX} (m^3/s)	Pretoria max (mm/hr)	Irene max (mm/hr)
1993/10/25	26	1,68	8,4	9,6
1993/12/07	31	0,79	13,4	18,4
1994/12/15	25	1,50	30,2	12,6
1995/04/09	26	3,38	12,8	1,8
2010/02/16	30	2,64	5,6	10,4
2010/03/05	30	4,20	1,4	12,2
2010/04/22	36	3,24	-	12

If only the events occurring in March and April are compared – to incorporate the antecedent soil moisture conditions of the preceding wetter months – it was found that the post development peak flow rates were respectively 1,24 and 0,96 times the peak flow rate of the event that occurred in April 1995. It should be noted that the volumetric size of the pre-development storm was smaller than the 2010 events. **Assuming the events to be comparable, it could be said that the increase in the peak flow rate for post development conditions is not as significant as predicted by the Rational Method (Section 2).**

4.6 Storm events evaluated based on the daily rainfall and hourly intensity

In this section, storm events were identified as similar based on the daily rainfall and the intensity. The storm events were assumed to be uniform over the whole catchment area and constant for the length of time identified from the hydrographs as described above. The estimated storm intensity is plotted against the daily rainfall in **Figure 4-14.**

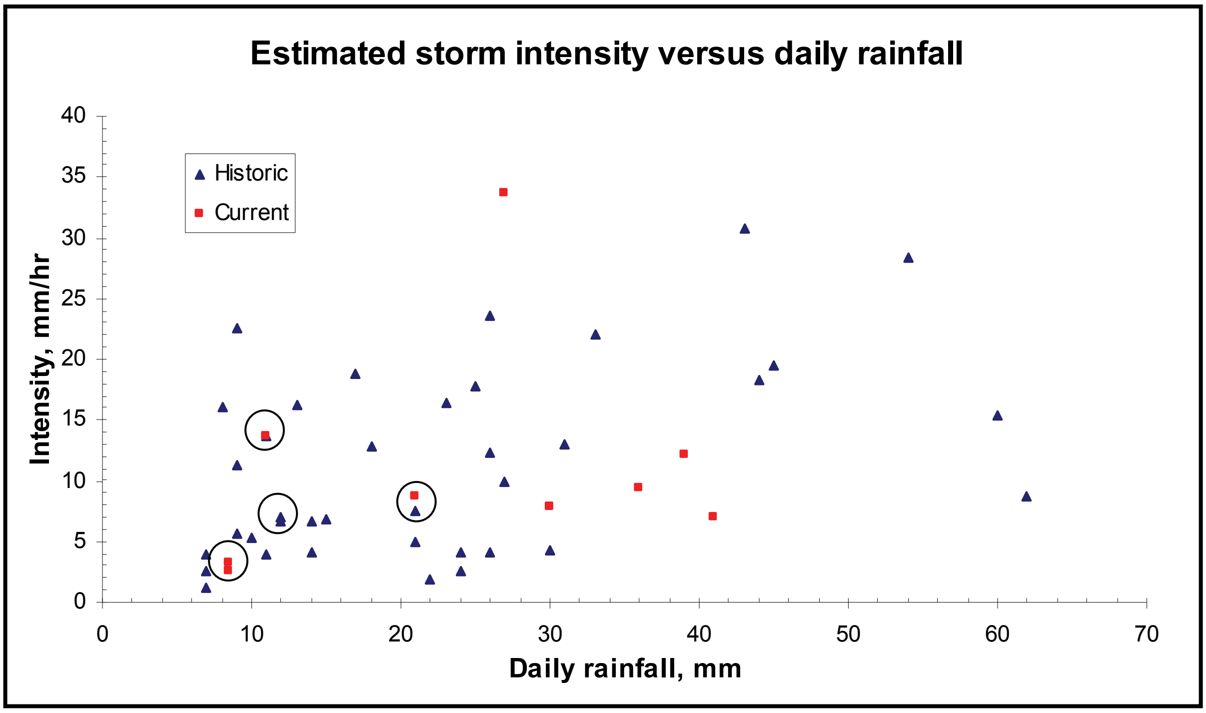


Figure 4-14: Estimated storm intensity versus daily rainfall

Figure 4-14 reflects that the rainfall intensity varies greatly for similar daily rainfall. Storm events were identified for which the **flow rates could be compared based on similar values of daily rainfall and estimated rainfall intensity**. It is important to be aware of the possibility that the downpour could have occurred fragmented, which would result in the peak flow rate to be much less as the hydrograph in such an incident would consist of a number of peaks. It is also stressed that assumptions based on one rainfall measuring station is not necessarily representative of the actual occurrence of rainfall over the whole catchment area.

A three dimensional bubble-plot of the storm events is reflected in **Figure 4-15**, which indicates daily rainfall and calculated rainfall intensity on the horizontal and vertical axes while the relative magnitude of the peak flow rate is depicted by the size of the bubble.

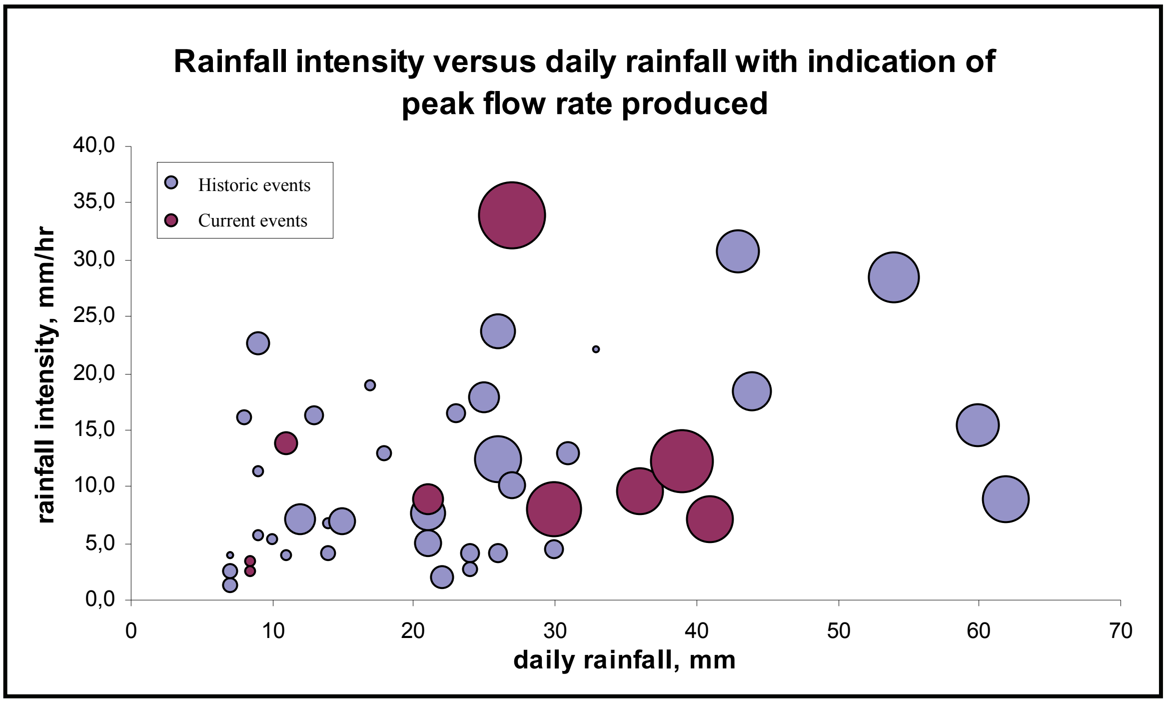


Figure 4-15: Three dimensional bubble-plot depicting: daily rainfall, rainfall intensity and peak flow rate

From the figure above it would seem that the peak flow rate for post development conditions is generally larger than the peak discharge from pre developed conditions. It should be noted that the amount of data limits the attempt to compare the peak runoff magnitude for similar events. The soil moisture conditions and other climatological factors were not considered either.

Two similar rainfall events (similar daily rainfall and intensity) occurred in the historic data set and two similar events occurred in the 2010 data set. These events were compared and included in the report to give an indication of the appropriateness of comparing events identified from the intensity/daily rainfall plot. These events are discussed in next section.

4.6.1 Pre-development events with similar daily rainfall and intensity

The two similar events, as identified above, occurring under pre-development conditions are reflected on in **Table 4-3.**

Table 4-3: Intensity and volumetrically similar events under pre-development conditions

Date	Daily rainfall (mm)	I (mm/hr)	Q_{MAX} (m^3/s)	$\Sigma\Delta t_{RISE}$ (min)
1995/01/14	12	6,67	0,43	108
1995/04/30	12	7,06	1,58	102

The hydrographs of the two events are illustrated in **Figure 4-16** and **Figure 4-17**. The second event yielded a maximum flow rate of 3,66 times the magnitude of the first event.

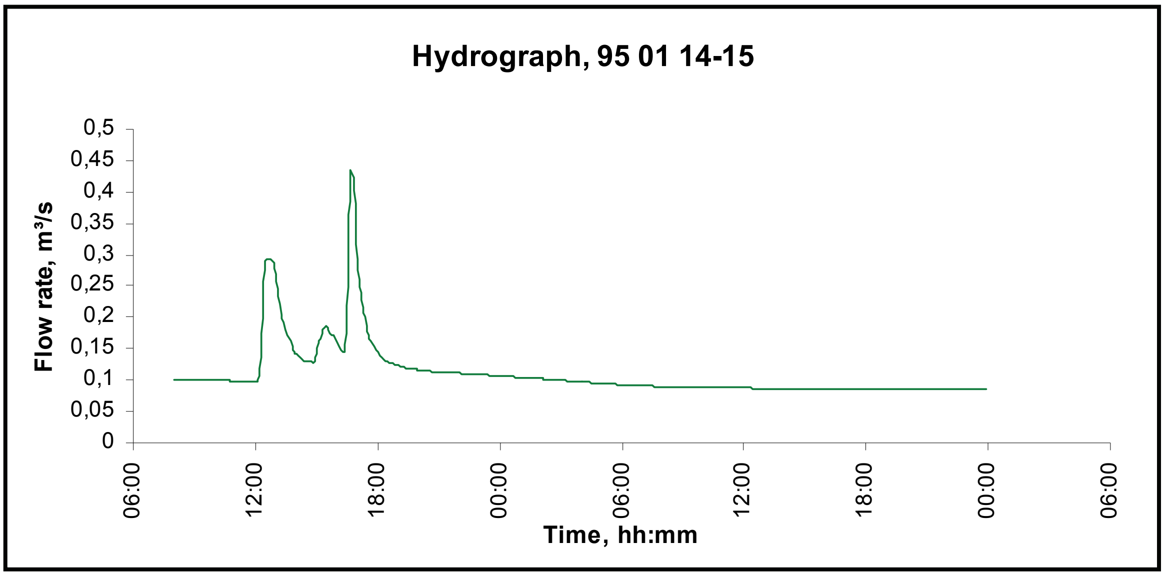


Figure 4-16: First event in the pre-development set of similar intensity storm event, 14 January 1995

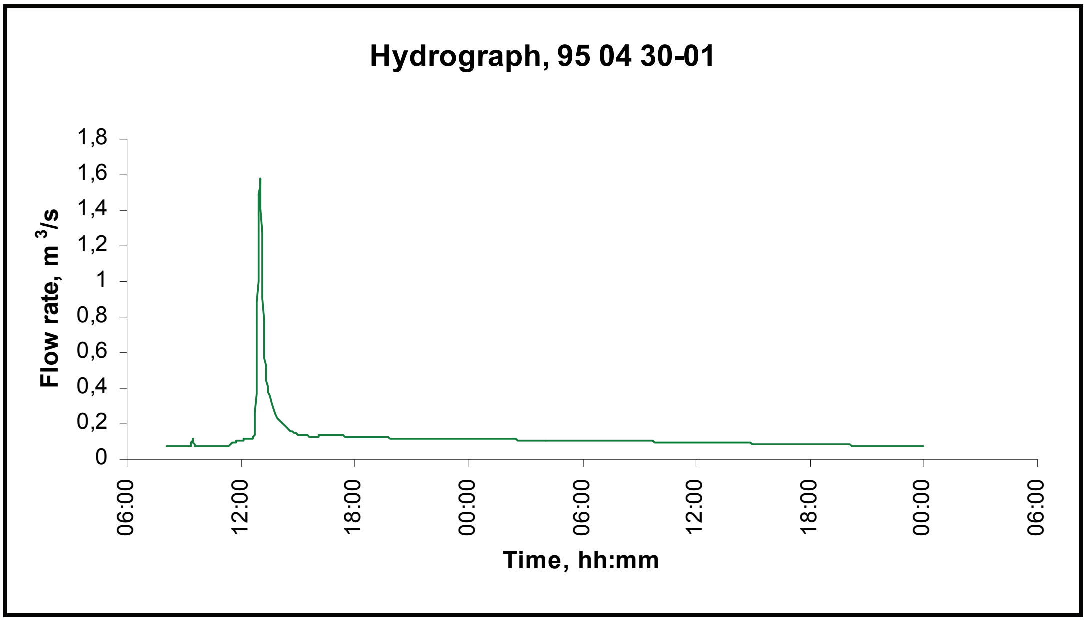


Figure 4-17: Second event in the pre-development set of similar intensity storm event, 30 April 1995

From the hydrographs it is clear that the first event is fragmented while the second event occurred over a short period. This contributed to a much higher discharge from the second event. Furthermore, the second event occurred late in the rainfall season when soil moisture conditions are expected to be saturated.

4.6.2 Post development events with similar daily rainfall and intensity

The two similar events occurring under the current (post) development conditions are reflected on in **Table 4-4**. The daily rainfall is small and could suggest that the storm event was not uniform over the whole catchment.

Table 4-4: Intensity and volumetrically similar events under current development conditions

Date	Daily rainfall (mm)	I (mm/hr)	Q_{MAX} (m^3/s)	$\Sigma\Delta t_{RISE}$ (min)
2010/01/23	8,5	3,27	0,19	156
2010/02/25	8,5	2,50	0,25	204

The second event (2010/02/25) yielded a peak flow rate of magnitude 0,247 m^3/s which was 1,28 times higher the first event. **Figure 4-18** indicates a variance of the discharge, while **Figure 4-19** indicates the discharge from a uniformly distributed event.

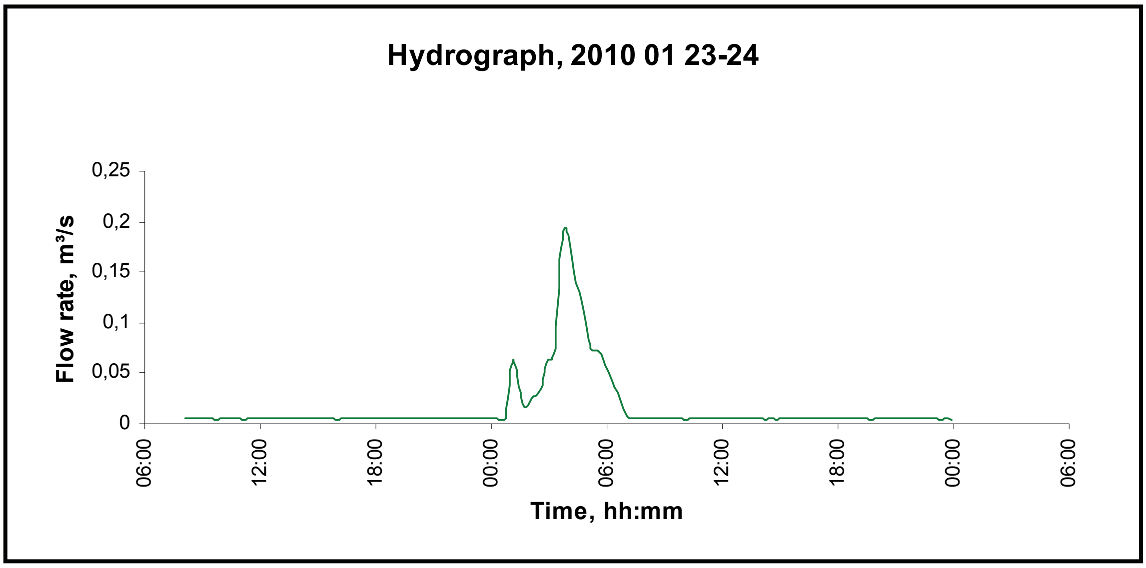


Figure 4-18: First event under post development conditions, of similar storm intensity, 23 January 2010

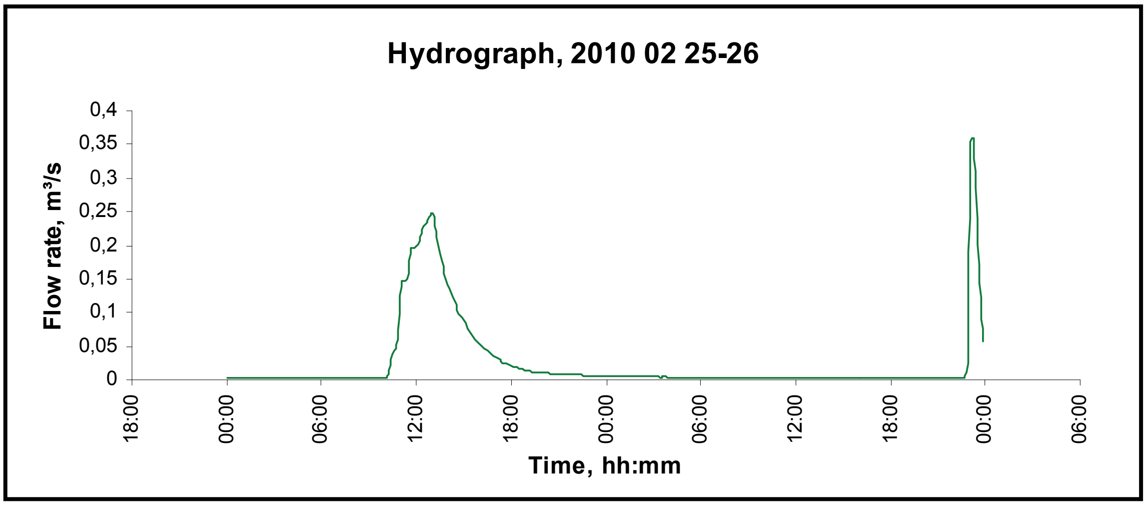


Figure 4-19: Second event under post development conditions, of similar storm intensity, 25 February 2010

4.6.3 Comparison of pre- and post-development events with similar daily rainfall and intensity

Two sets of events identified for comparison between the pre-development and post development peak flow rate are summarised in **Table 4-5.**

Table 4-5: Intensity and volumetrically similar events identified for comparing of peak flow rates produced

Date	Daily rainfall (mm)	I (mm/hr)	Q_{MAX} (m^3/s)	$\Sigma\Delta t_{RISE}$ (min)
1995/04/21	11	13,75	0,61	48
2010/03/02	11	13,75	0,96	48
1994/10/28	21	7,50	1,78	168
2010/04/01	21	8,75	1,58	144

For the first set of events, the post development peak flow rate is 1,58 times greater than under the pre-development conditions. The daily rainfall measured at Ramkie-oor-Willows was 11 mm for both events, but uncertainty exists of the spatial distribution of the storm. The hydrographs for these two events are included as **Figure 4-20** and **Figure 4-21**. The base flows depicted on these graphs cannot be deemed representative as the flow gauging station could not measure the lower range of flow height due to the bottom pipe outlets being open.

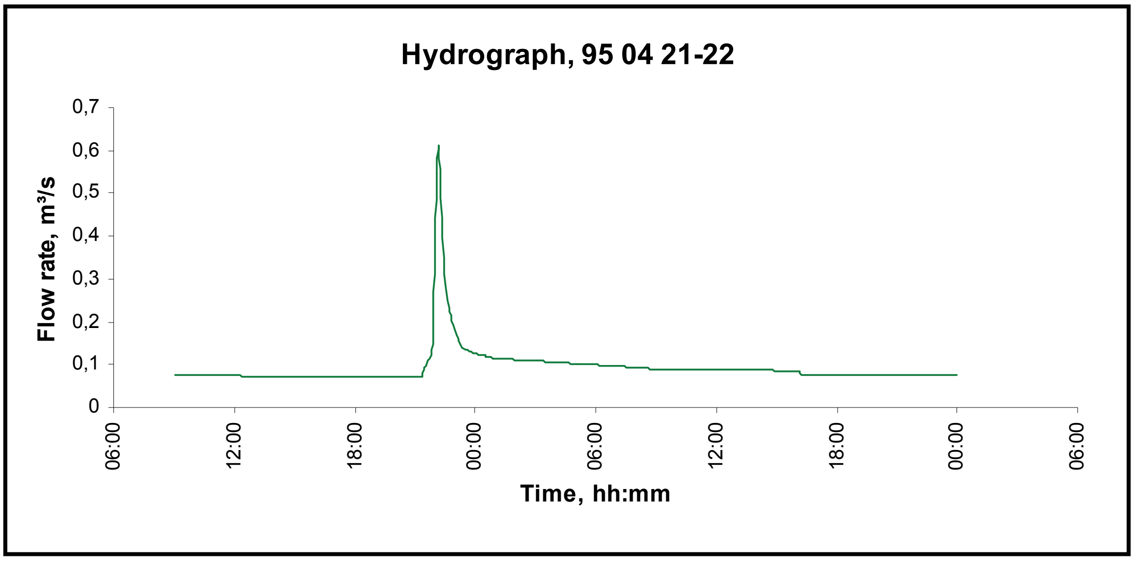


Figure 4-20: Hydrograph produced under pre-development conditions for storm of I=13,75 mm/hr

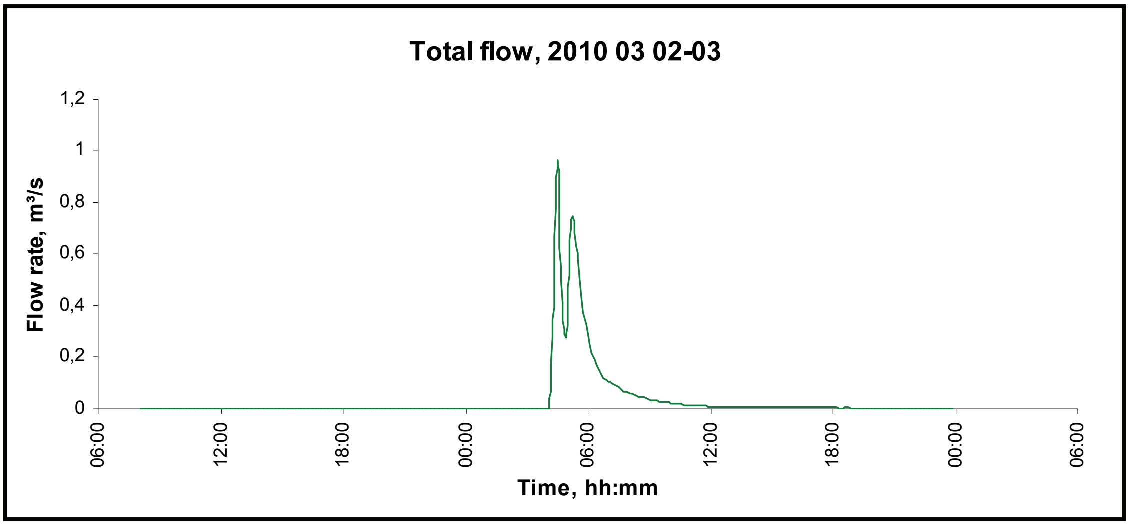


Figure 4-21: Hydrograph produced under current development conditions for storm of I=13,75 mm/hr

The second set of events rendered a post development peak flow rate of 0,89 times the peak flow rate produced under the pre-development conditions. Both the events seem to be fragmented as is clear from **Figure 4-22** and **Figure 4-23**. The second event occurred later on in the rainy season and it received 41 mm of rainfall on the day prior to the event, thus the antecedent conditions should in theory have contributed to the higher peak discharge.

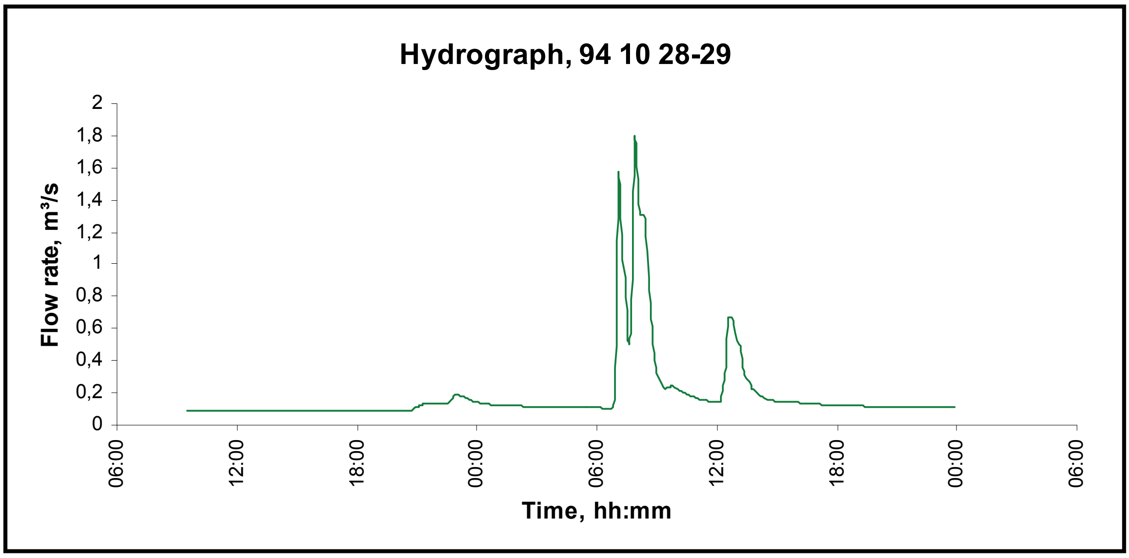


Figure 4-22: Hydrograph produced under pre-development conditions for storm of I=7,5 mm/hr

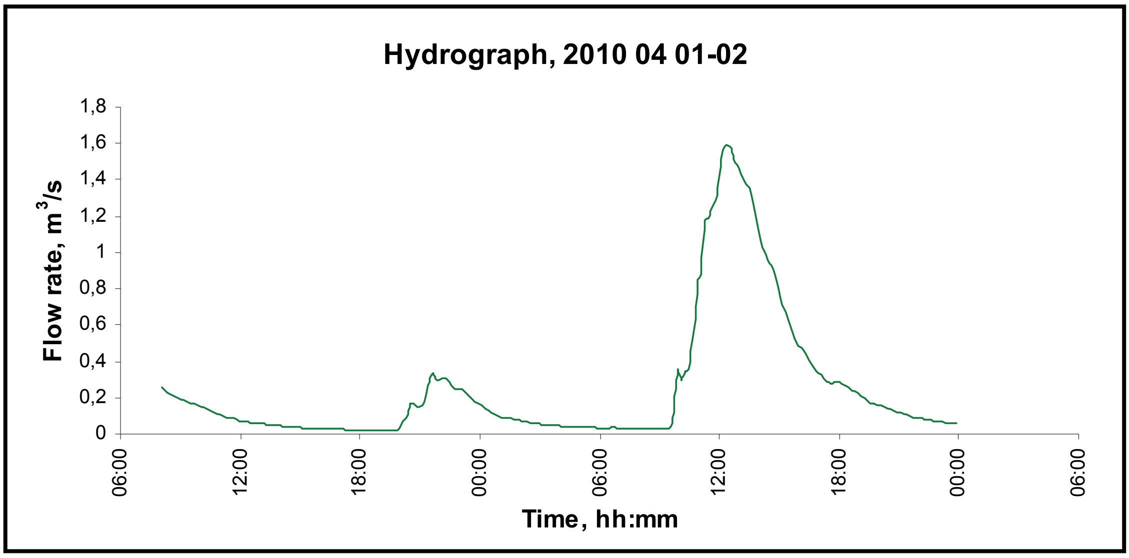


Figure 4-23: Hydrograph produced under current development conditions for storm of I=8,75 mm/hr

4.7 Conclusion

It can be concluded from this section that the data suggests that the increase in peak discharge is due to urban development, but this increase is not as large as predicted by the Rational Method. Furthermore it is clear that the antecedent conditions may have less of an effect on the peak runoff for low intensity storms than initially anticipated (**Section 4.6.3**). The effect of intensity, which should be the main influence, on peak discharge could not be determined accurately due to a lack of accurate data and the length of the data sets.

Although some general effects of staggered rainfall could be linked to the peak discharge rates. It must still be concluded that the amount of comparable events was too little to determine the effect of staggered rainfall over the catchment.

The effect of spatial distribution could not be determined due to an insufficient amount of rain gauges that were available for this analysis.

5 Comparison of the measured and modelled catchment response to rainfall at Robert's Place

Robert's Place development was subdivided in two zones (A and B) as was discussed in **Section 3.5.2**. The results of the assessment for the two zones are discussed below.

5.1 Results and discussion of Zone A

For Zone A the crump weir was used to measure the discharge from the crump weir into the stormwater system. This data was compared to the results from the simulated runoff using the EPA SWMM 5 model. The data for Zone A spans from middle of December 2009 to the beginning of February 2010 the rest of the discharge data was lost due a faulty pressure transducer. The calculated discharge from the catchment resulting from the usage of the EPA SWMM 5 model, were compared with the measured discharges from the selected individual events. **Table 5-1** compares the following parameters resulting from the modelling of the catchment and the recorded data set for the catchment:

- Volumes of the precipitation;
- Total volume of measured and modelled runoff.

Table 5-1: Comparison of measured and modelled discharges for Zone A

Date	Event number	Calculated Volume of Rainfall (m^3)	Time of Concen tration (min)	Peak discharge (m^3/s)		Volume of outflow (m^3)	
				Measured	Modelled	Measured	Modelled
10-Dec-09	A1	88,1	7,67	0,007	0,016	13,46	23.0
21-Dec-09	A2	76,6		0,011	0,005	16,00	20,0
13-Jan-10	A3	75,4		0,070	0,006	41,95	18,0
18-Jan-10	A4	46,8		0,014	0,006	13,75	10,0
20-Jan-10	A5	34,6		0,007	0,002	9,50	8,0
21-Jan-10	A6	22,0		0,002	0,001	3,46	3,0

The events listed in **Table 5-1** were all assumed to be single storm events, with the rainfall used in the modelling similar to the recorded rainfall which is reflected in the **Figure 5-1** to **Figure 5-6**. On the 10^{th} of December 2009, the rain storm occurred over a time span (2,5 hours) much longer than the time of concentration and consisted of two separate events. As input to the modelled discharge, the recorded rainfall data (**Figure 5-1** to **Figure 5-6**), were used as input to the "atmospheric compartments" in EPA SWMM. It was assumed that the rainfall was spatially uniformly distributed.

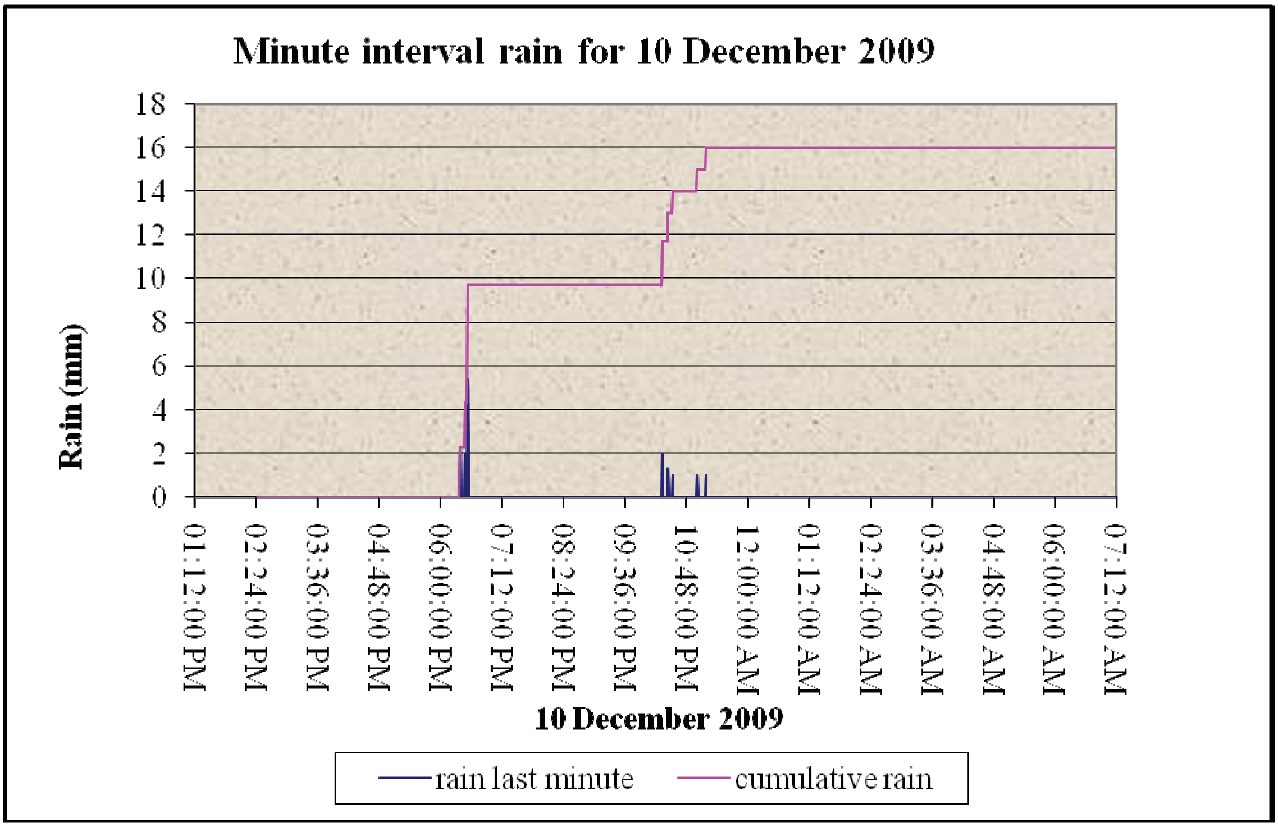


Figure 5-1: Recorded rainfall intensity per minute: Event 1

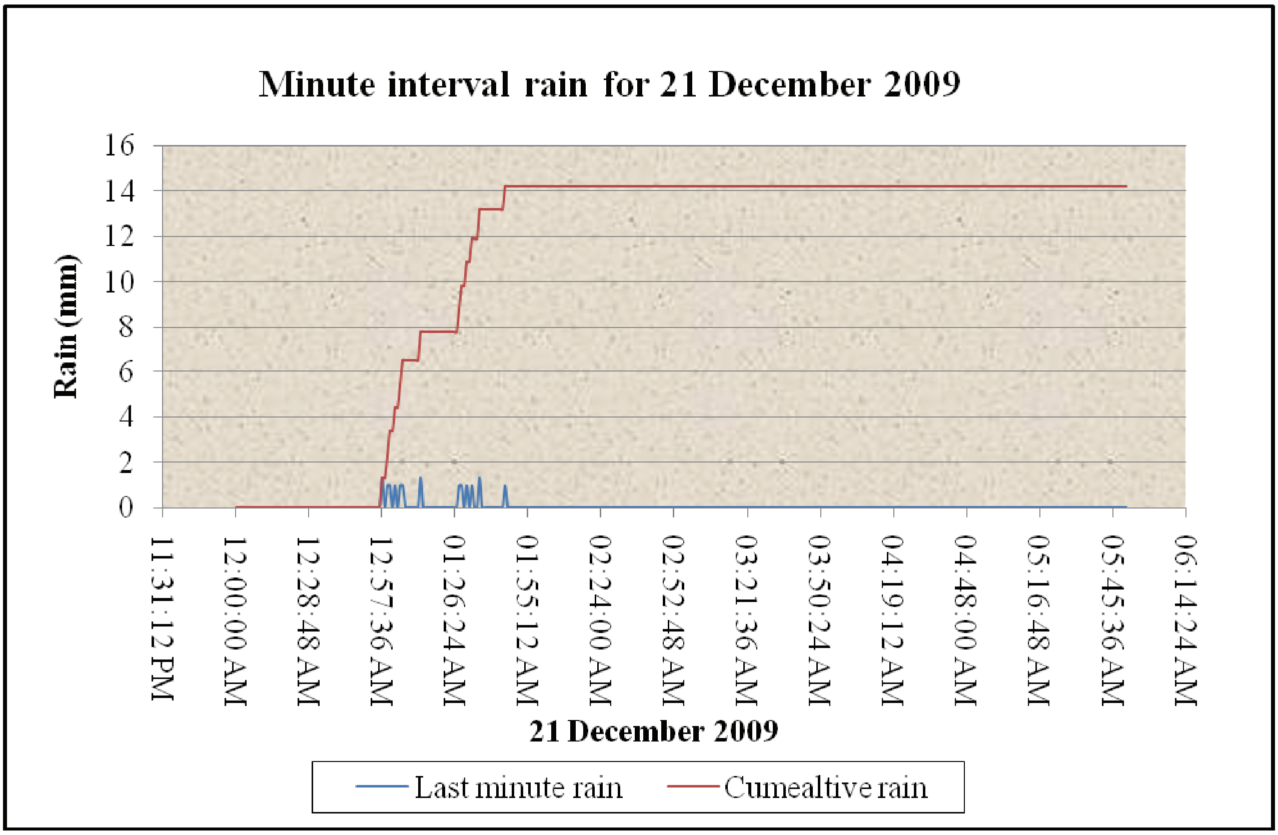


Figure 5-2: Recorded rainfall intensity per minute: Event 2

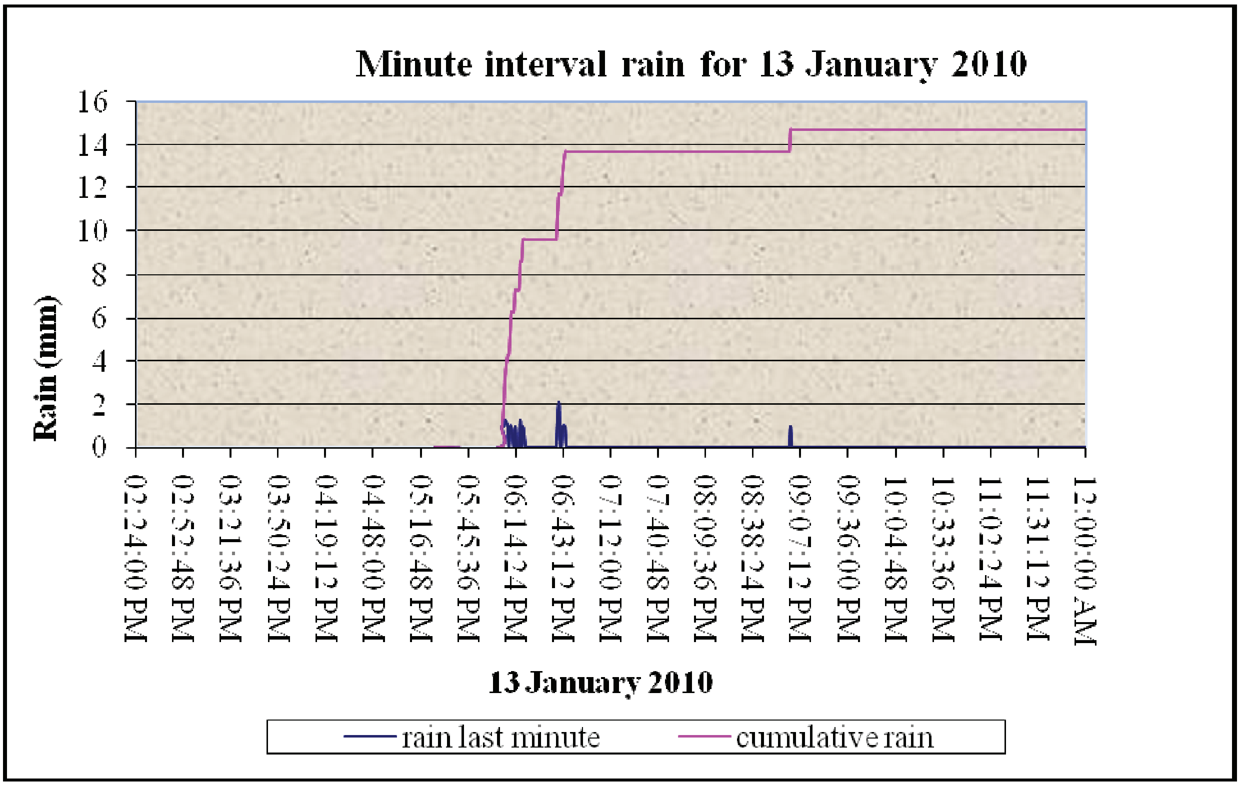


Figure 5-3: Recorded rainfall intensity per minute: Event 3

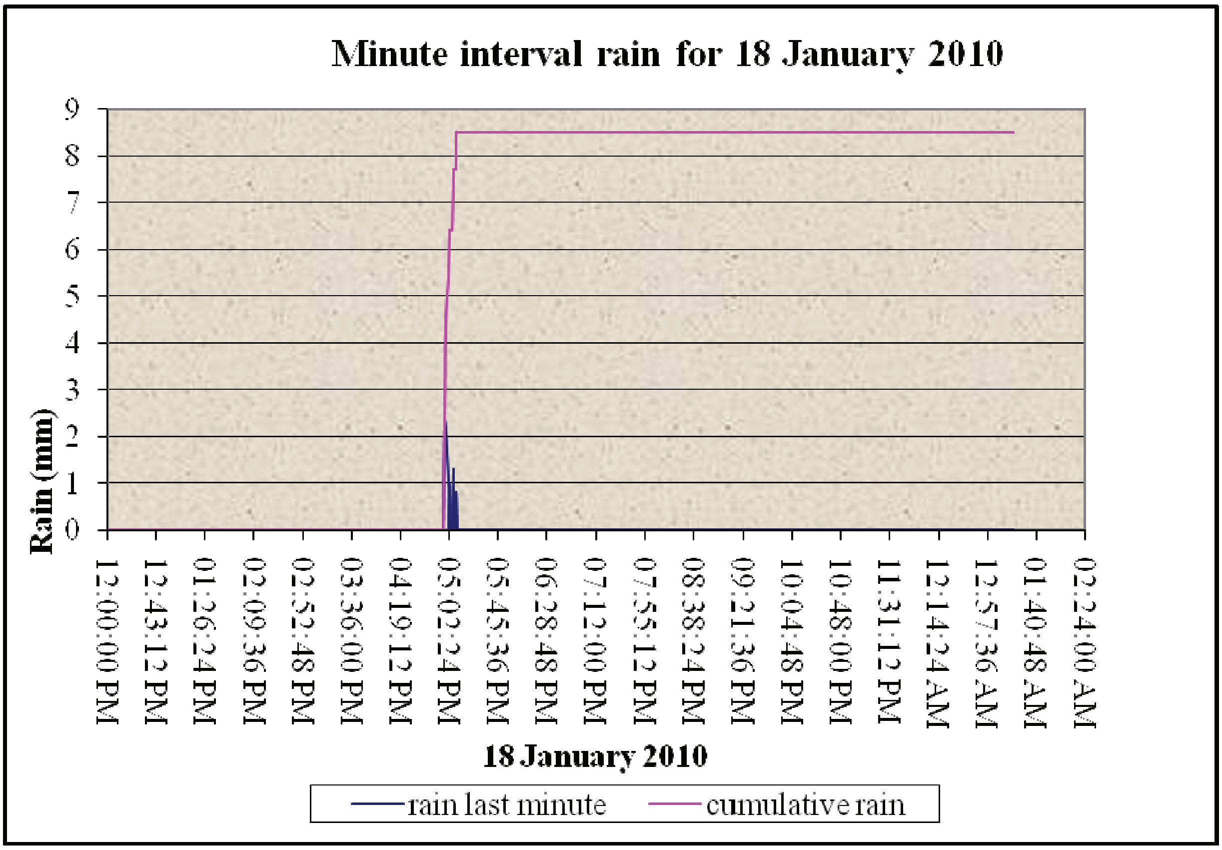


Figure 5-4: Recorded rainfall intensity per minute: Event 4

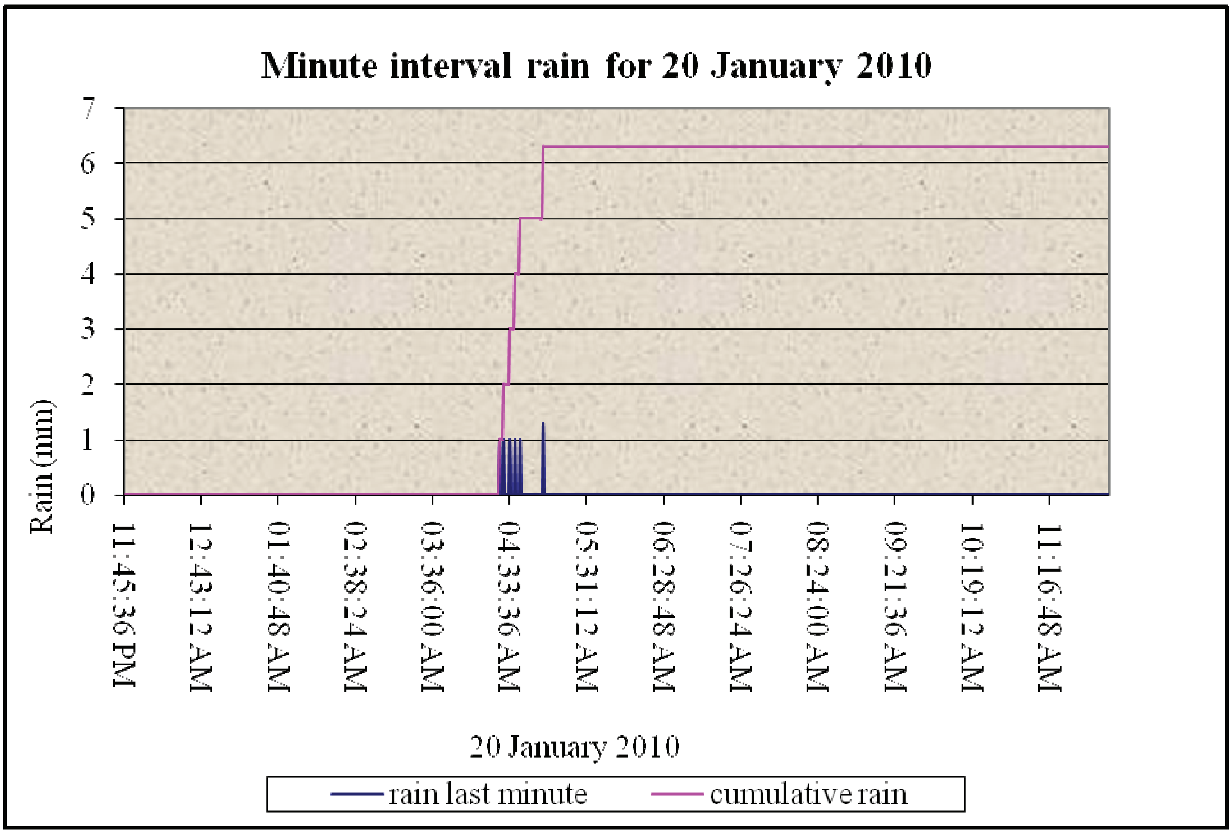


Figure 5-5: Recorded rainfall intensity per minute: Event 5

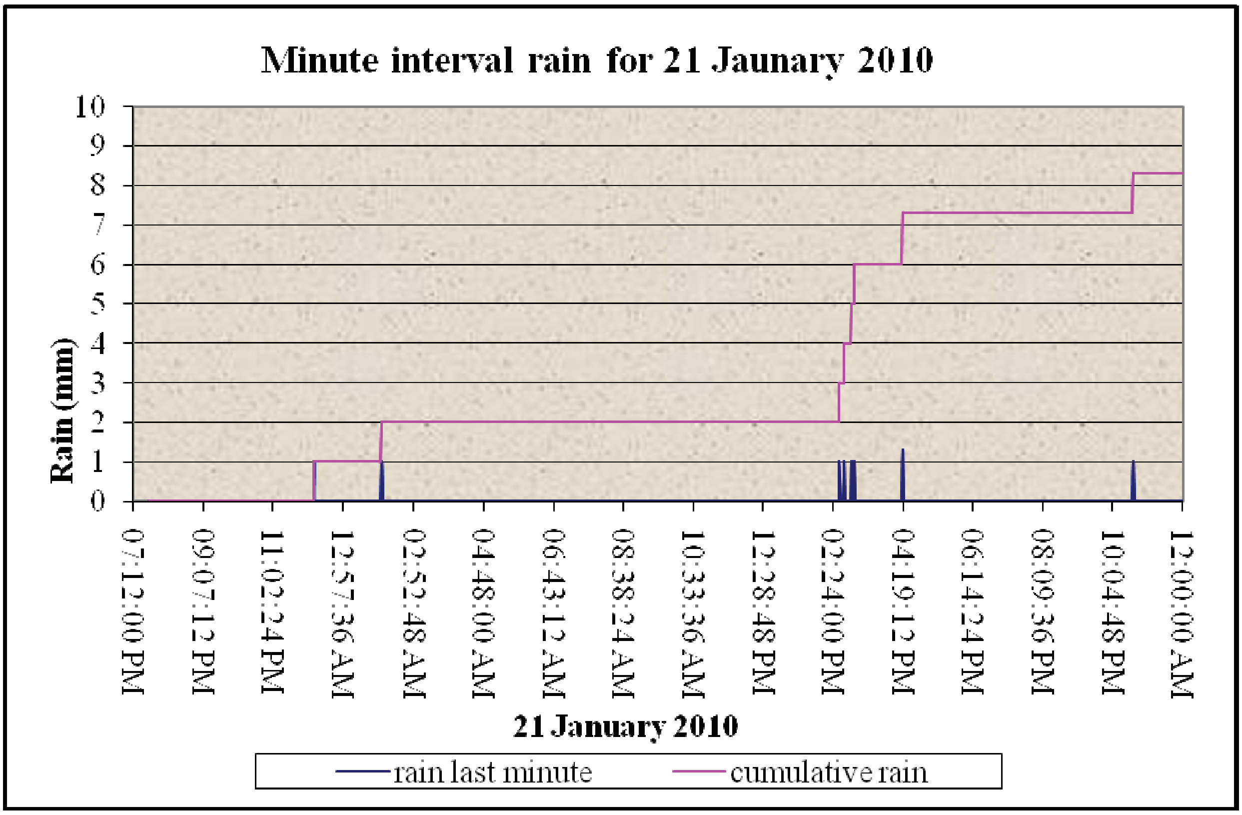


Figure 5-6: Recorded rainfall intensity per minute: Event 6

Figure 5-7 compares the measured and the modelled runoff peaks.

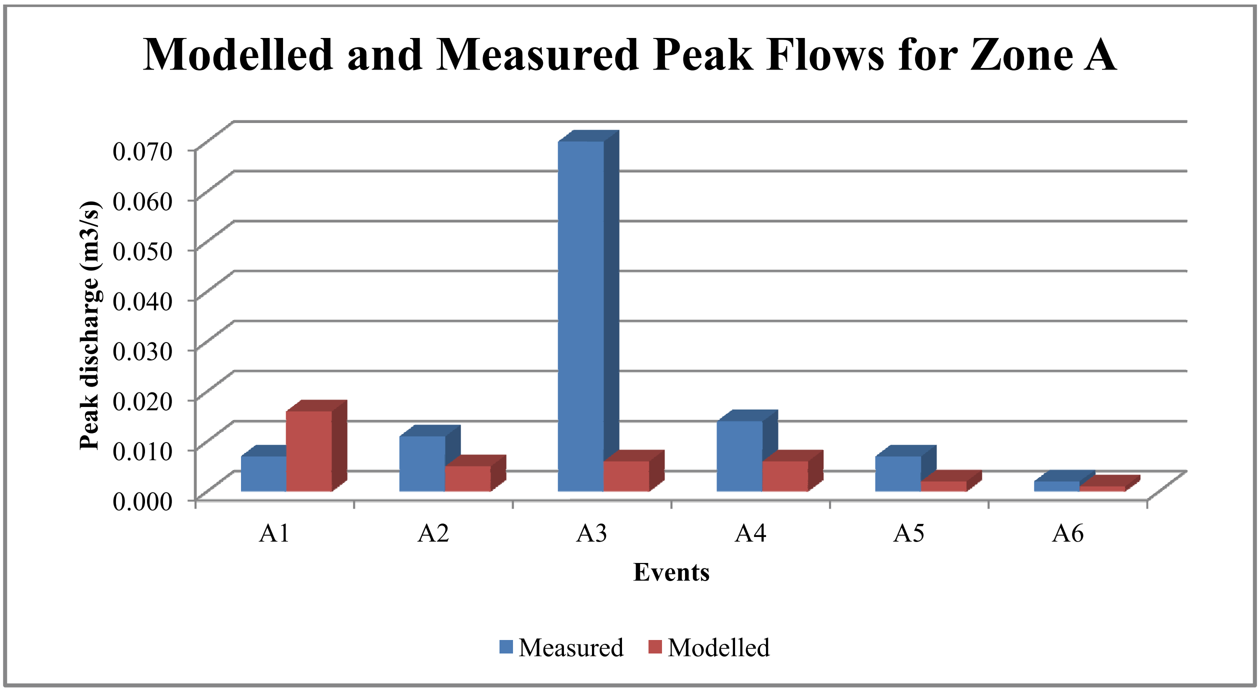


Figure 5-7: Visual comparison of the measured and recorded runoff rates for the different events for Zone A

Figure 5-8 visually compares the calculated rainfall volume, recorded measured volume and the modelled runoff volume for the different storm events.

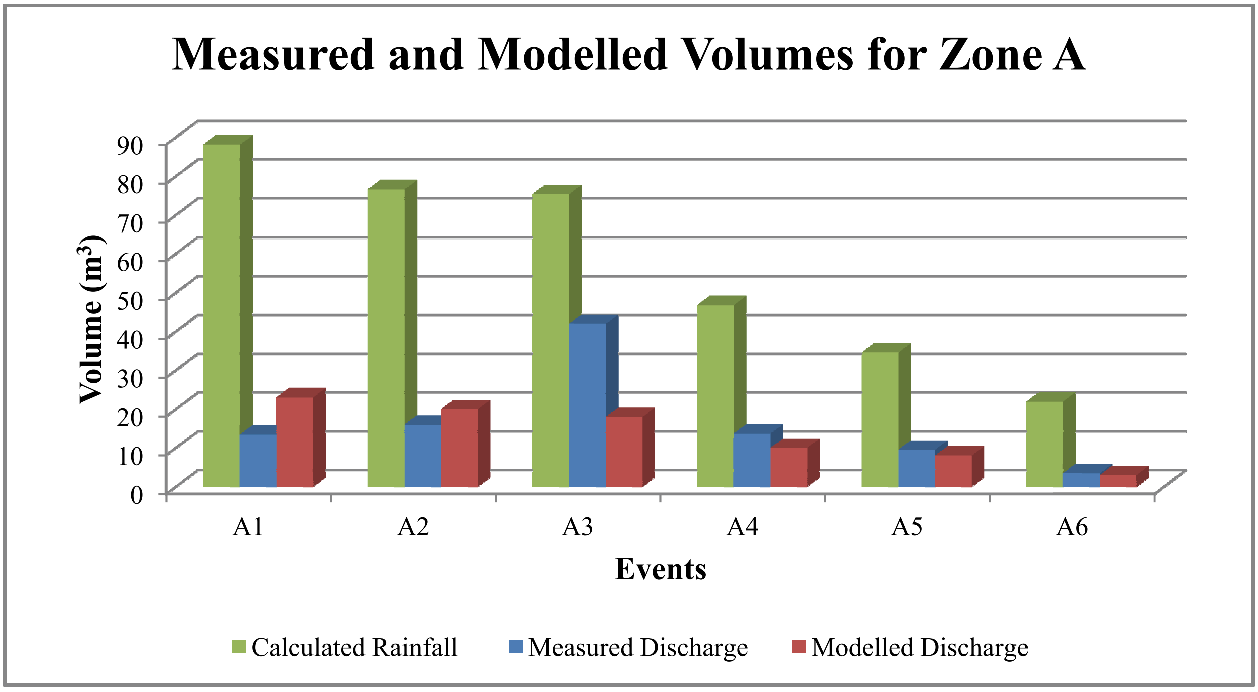


Figure 5-8: Comparison of the total volumes of runoff for Zone A

In some events, the recorded volume of discharge was more than the modelled volume of discharge. This could be due to some difficulty experienced with ensuring that the drainage structure (crump weir) remained clean from any material as well as the feed pipe to the pressure transducers remains unblocked.

5.2 Results and description of Zone B

The discharge for Zone B was measured with a 90º V-notch.

Similar to the above description for Zone A, calculations were conducted to determine the volume of rainfall for the different events for Zone B and the these were modelled using EPA SWMM from which it was possible to compare the modelled and recorded flow rates and volumes of discharge from Zone B.

Table 5–2 reflects the comparative results

Table 5-2: Comparison of measured and modelled discharge volumes and flow rates for the different events for Zone B

Date	Event number	Calculated Rainfall Volume (m^3)	Time of Concentration (min)	Peak Discharge (m^3/s)		Volume of outflow (m^3)	
				Measured	Modelled	Measured	Modelled
10-Dec-09	B1	196	6,99	0,012	0,024	15,5	61,0
21-Dec-09	B2	170		0,011	0,017	11,3	52,5
13-Jan-10	B3	168		0,044	0,020	35,4	51,0
18-Jan-10	B4	104		0,250	0,017	10,0	29,0
20-Jan-10	B5	77		0,007	0,006	4,0	20,0
21-Jan-10	B6	49		0,002	0,003	1,5	11,0

In Zone B, similar to what was found in Zone A; the measured discharge rates for some events are smaller and others are larger than the modelled discharge rate.

Figure 5-9 provides a visual comparison of the measured and the modelled peak flow rates.

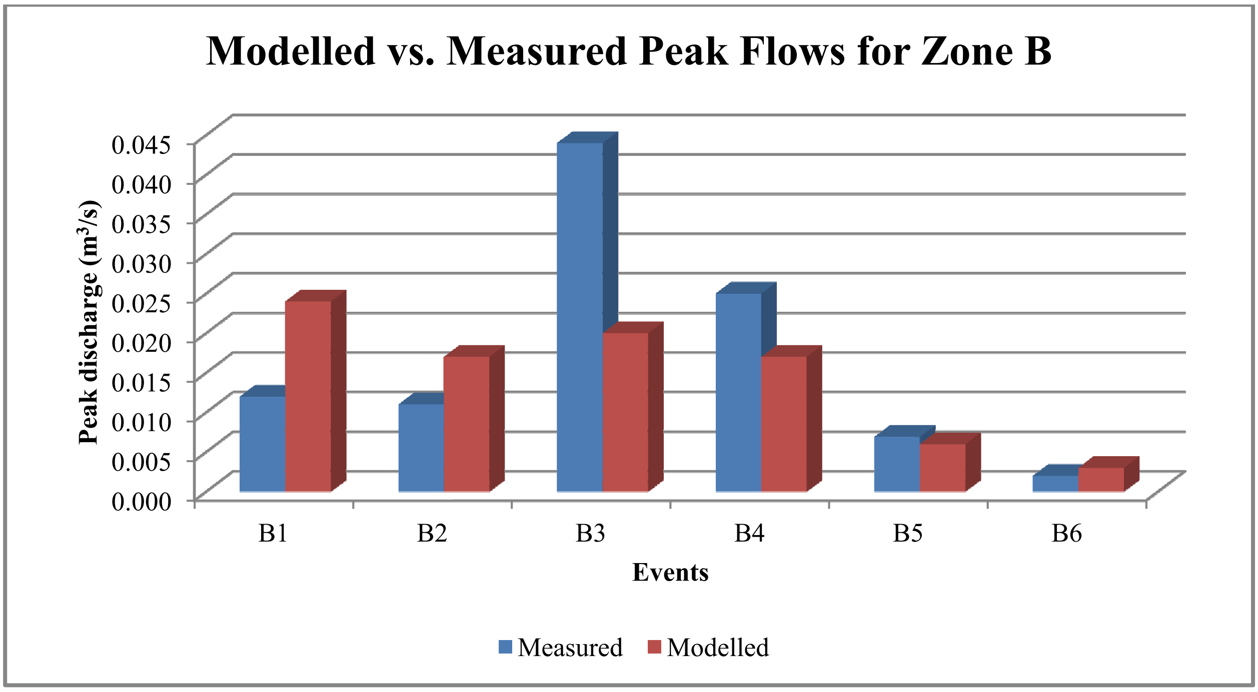


Figure 5-9: Visual comparison of the measured and recorded runoff rates for the different events for Zone B

Figure 5-10 indicates to the volume of runoff calculated from the precipitation, the measured runoff and the modelled runoff volume for Zone B. In all events, the recorded volume of discharge is less than the modelled volume of discharge. This suggests that the modelling provides a larger discharge volume than what was physically measured.

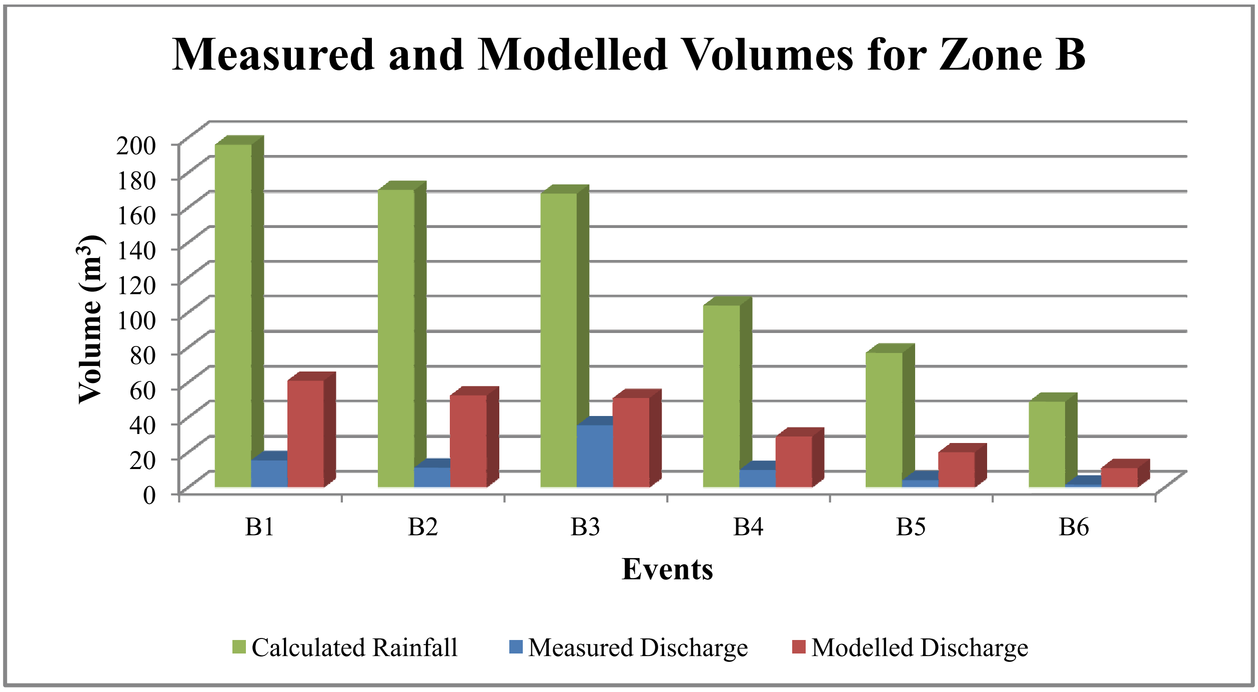


Figure 5-10: Comparison of the total volumes of runoff for Zone B

Table 5-3 reflects a summary of the comparison between modelled and recorded peak discharge for Zone A and B.

Table 5-3: Relationship between the modelled and the measured peak discharges for

Zones A and B

Zone	Event #	Peak modelled discharge (m^3/s)	Peak measured discharge (m^3/s)	Modelled divided by the measured discharge rate
A	1	0,016	0,007	2.29
	2	0,005	0,011	0.45
	3	0,006	0,07	0.09
	4	0,006	0,014	0.43
	5	0,002	0,007	0.29
	6	0,001	0,002	0.50
B	1	0,024	0,012	2.00
	2	0,017	0,011	1.55
	3	0,020	0,044	0.45
	4	0,017	0,25	0.07
	5	0,006	0,007	0.87
	6	0,003	0,002	1.50

Note:
**#	The duration of events 1, 3 and 5 were staggered while event 4 was short**

Table 5-4 reflects the important settings used in the EPA SWMM software to generate discharge from Zones A and B.

Table 5-4: Variables used in the EPA SWMM model for Zone A and B

Description of the Variable	Units	Value of the Variable
Flow units	m^3/s	Calculated
Infiltration	none	Green ampt method
Flow routing	none	Dynamic wave
Start date	none	Individual storm events
Dry days	days	0
Report step	seconds	30
Wet step	minutes	1 min intervals
Dry step	hours	1
Routing step	seconds	30
Force main equation	none	Hazen-Williams
Conduit roughness	none	0,015
Sub catchment slope	%	2

When calculating peak urban runoff with the Rational Method, the discharge coefficient is normally larger than 0,6. **Based on the measured discharge from Zone A and B, the calculated discharge peak is as low as 0,37 for Zone A and 0,2 for Zone B.**

This could probably be ascribed to routing of the discharge from impervious (disconnected) surfaces across pervious surfaces, like grass and flower beds, to the outlets. In the case of Zone B the weighted connected impervious area is less than that for Zone A.

Table 5-5 and **Table 5-6** illustrate this for Zone A and Zone B.

Table 5-5: Runoff area distribution for Zone A of the catchment

Area distribution			
Area (m^2)	**Weighted Impervious (%)**	**Weighted Connected impervious (%)**	**Other**
5 451,0	62,7	36,8	0,5

Table 5-6: Runoff area distribution for Zone B of the catchment

Area distribution			
Area (m^2)	**Weighted Impervious (%)**	**Weighted Connected impervious (%)**	**Other**
13994,6	62,2	19,5	18,3

5.3 Temporal storage structures

The difference between the measured discharge and the modelled discharge (EPA SWMM) for the different events, are significant. This could probably be due to the large temporal interception volume in both the catchments A and B which were investigated here.

5.4 Conclusion

The staggered/inconsistent nature of the recorded rainfall and the large variance of the intensity between the different events which were discussed above, illustrate the complexity of storm events and the extent of simplification which are required to be able to model even small catchments.

A scientific conclusion cannot be reached based on the limited number of events analyzed for the comparison between recorded and modelled runoff, although it seems that the limited data suggests that the recorded data produces less runoff volumes, with some lower peak flow rates than the modelled response from highly developed catchments, this supports the hypothesis that development has the potential of lower the peak discharge rates.

6 Assessment of the hydrological records to establish the influence of development on the runoff response from larger catchments

Although the initial objectives did not include a review of larger catchments, some catchments were selected and are discussed below for hydrological years with similar:

- Accumulated rainfall;
- Temporal distributions of rainfall; and
- Antecedent conditions.

The selected catchments all experienced extensive urban development over the record period. The following catchments were thus reviewed:

- Rietvlei Dam catchment (A2R004) which has a low urban development component, but a large agricultural component;
- Daspoort catchment (A2H007) which had high urban development; and
- Kameeldrift catchment (A2H028) which has intermediate urban development.

6.1 Rietvlei Dam Catchment (A2R004)

6.1.1 General information

Figure 6-1 below indicates the location of Rietvlei Dam (A2R004).and the contributing catchment area

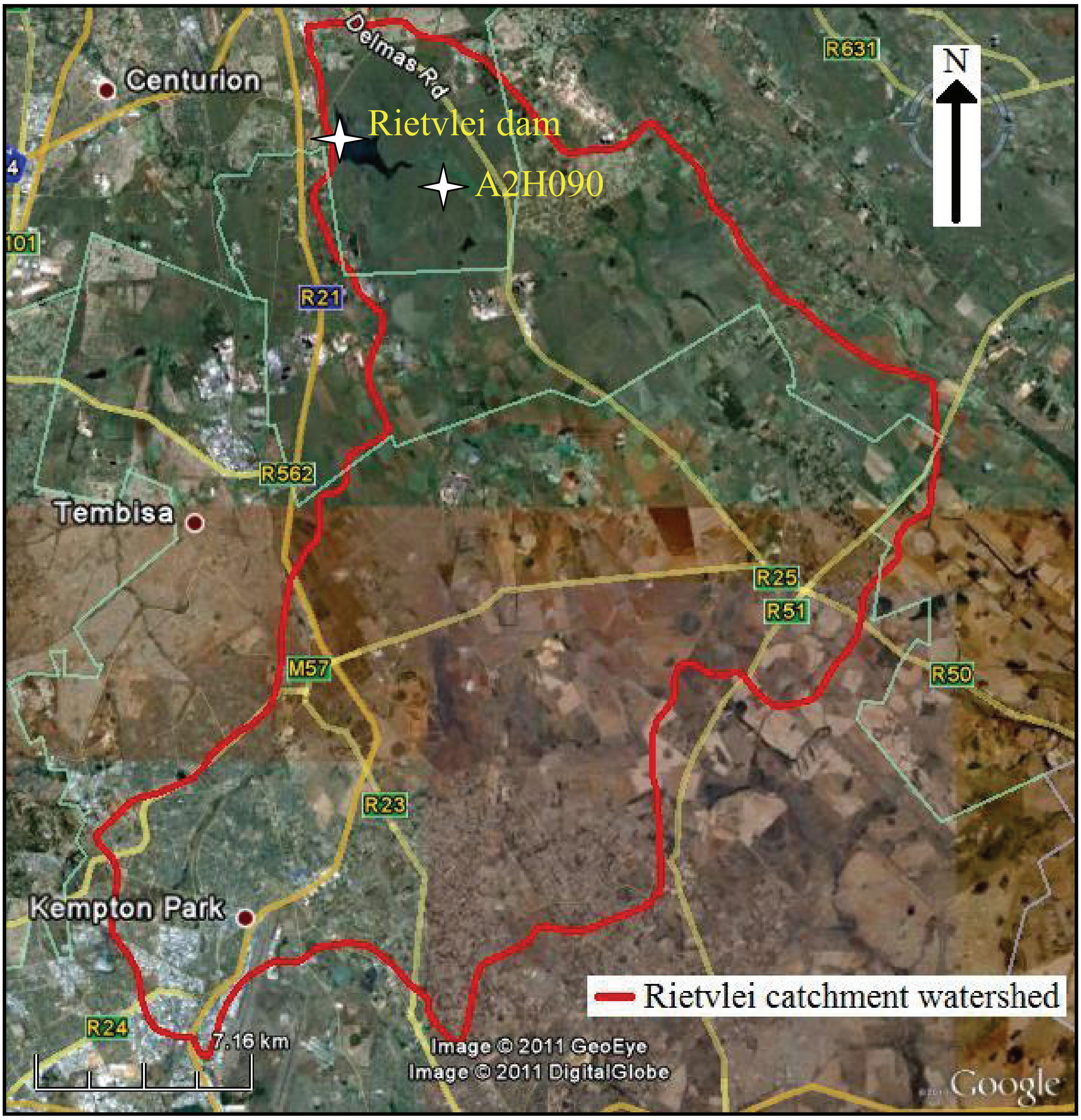


Figure 6-1: Catchment area of Rietvlei Dam

Table 6-1 provides some details of the Rietvlei Dam catchment, which was relatively undeveloped until the middle 1980's.

Table 6-1: Main features of the Rietvlei Dam catchment

Parameter	Value	
Impoundment	Rietvlei Dam	
Area of catchment (A2R004)	481 km^2	
River	Hennops River	
Rainfall region	A21A	
Gauging stations	A2R004 & A2H090	
MAP	684 mm	301,64 Mm^3
MAR	33,4 mm	14,73 Mm^3
Period for which flow records are available	October 1933 to present *	

Note:

* Data up till 1988 was reviewed.

The percentage urban development in the catchment was calculated by reviewing the 1:50 000 topographical maps. GISap (Geographical Information System application) software was used to determine the urban development levels for 1952 and 1983. The percentage development for 1940 was also considered, this information was obtained from an unpublished report by Christie (2008). **Figure 6-2** below shows the urban development for 1952 and 1983. **Table 6-2** summarises the land use percentages in the catchment (Christie, 2008).

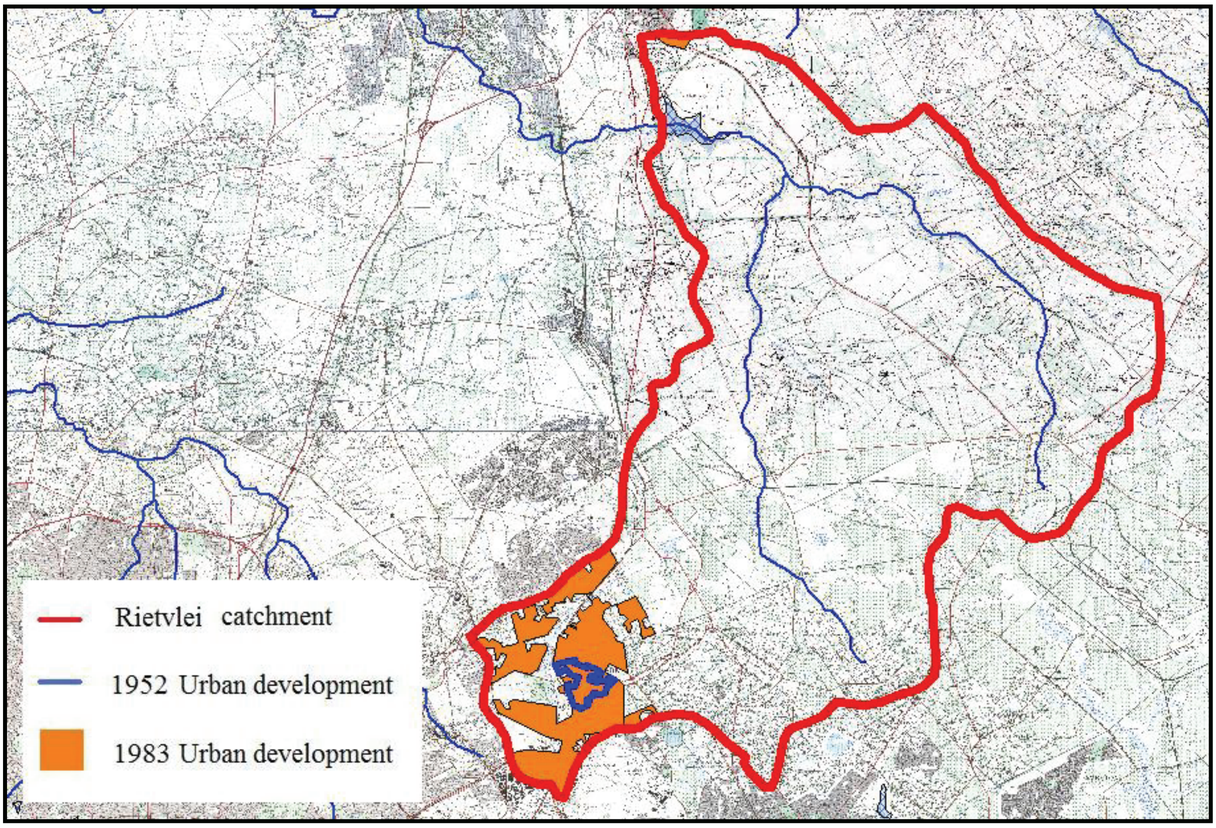


Figure 6-2: Rietvlei catchment with indications of urban development

Table 6-2: Summary of Rietvlei urban development percentages

Land use variation in the Rietvlei Dam catchment area					
% of Catchment developed	**Year**				
	1940#	1952	1960 #	1980 #	1983
Urban development	0	0,44	2	7	6,12
Agricultural development #	26 #	Not determined	40	59	Not determined
Rural state	74 #		58	34	

Note:

Information obtained from an unpublished report by Christie (2008).

6.1.2 Methodology for the hydrological review of similar hydrological years for the Rietvlei Dam catchment

The methodology to identify the influence of urban catchment development on runoff volume is based on the comparison of selected rainfall and runoff records for years for which the development levels are known and where the recorded yearly rainfall are comparable.

The initial data selection was based on comparing hydrological years which had similar regional rainfalls, which were close to the MAP of the catchment as shown in **Appendix F**. A detailed analysis was then conducted to determine the exact amount of rainfall for each catchment. This required an identification of which rainfall stations were located in close proximity to the catchments and then assessing which rainfall stations were operational for the required analysis year, as shown in **Appendix G**. The rainfall records from each individual rainfall station were used to calculate the rainfall volumes for each catchment. This was done by recognising the contribution from each area associated with a specific rainfall station (Theissen polygon method). Respective rainfall contribution had to be calculated for the selected hydrological years based on which rainfall stations were available, the summary of which is reflected in **Table 6-3**.

The flow data that was used in the comparison, was obtained from the Surface Water Resources of South Africa 1990, Volume I (Pitman, 1994) published by the Water Research Commission . Although due care was taken not to use patched flow records, the hydrological record for 1980, contained patched data for December 1980.

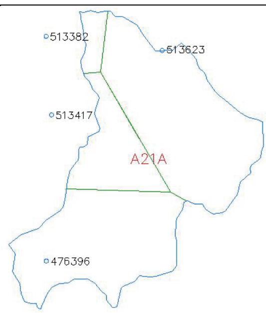
The catchment runoff volumes for the selected hydrological years were calculated from subtracting the return flows produced by the wastewater treatment works (WWTW), from the recorded flow at the flow gauging stations. The inflow to Rietvlei Dam (A2R004) incorporates the discharge from the Hartbeesfontein WWTW which was commissioned in 1977. The monthly WWTW flow volumes were obtained from the BKS report, “Assessment of the Water Availability in the Crocodile (West) River Catchment” (BKS, 2008) as shown in **Appendix H**. The gauging station, A2R004, is located upstream of Rietvlei Dam and therefore abstractions from Rietvlei Dam were not considered.

Table 6-3: Tabular and visual representations of area allocations of rainfall stations for the Rietvlei Dam catchment area (A2R004)

Area contribution of rainfall stations for the Rietvlei Dam catchment area (A2R004)	Visual representation of area allocations for rainfall stations in the Rietvlei Dam catchment area (A2R004)

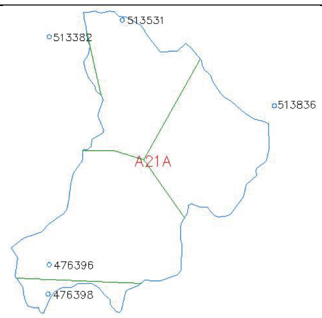
1940 and 1949

Section	Position	% of catchment area allocated to rainfall station
476	396	37
513	382	3
513	417	15
513	623	45
		100



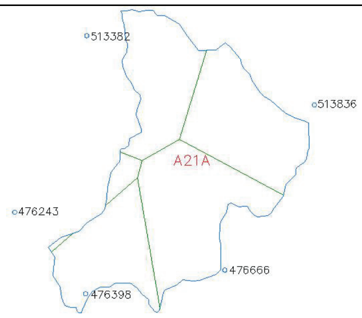
1960

Section	Position	% of catchment area allocated to rainfall station
476	396	40
476	398	4
513	382	1
513	531	26
513	836	29
		100



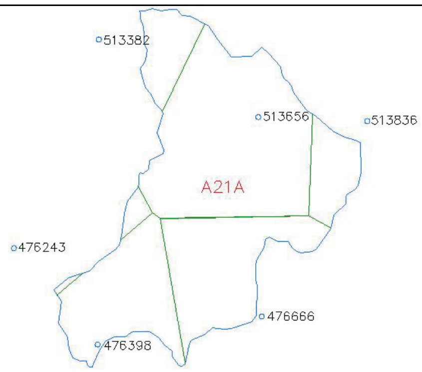
1969 and 1970

Section	Position	% of catchment area allocated to rainfall station
476	243	2
476	398	20
476	666	30
513	382	21
513	836	27
		100



1980

Section	Position	% of catchment area allocated to rainfall station
476	243	2
476	398	20
476	666	22
513	382	7
513	656	43
513	836	7
		100



The spatial and temporal distribution of rainfall and the antecedent catchment conditions are two main drivers which could distort the relationship between runoff and rainfall. Recorded yearly regional rainfall for three preceding years of each from the selected hydrological years was plotted in **Figure 6-3**. The temporal distribution was reviewed by normalising the accumulated calculated weighted rainfall for each of the selected hydrological as reflected in **Figure 6-4**. This indicated the relative intensities with which the rainfall occurred.

The rainfall data was reviewed to ensure that the data sets were comparable by:

- Comparing the yearly calculated rainfall for the selected hydrological years (**Figure 6-3**).
- Assessing the regional yearly rainfall which occurred for three years prior to each of the selected comparison years (**Figure 6-3**).
- Evaluating the accumulated normalised rainfall to determine the temporal rainfall distributions for the selected hydrological years as reflected in **Figure 6-4.**

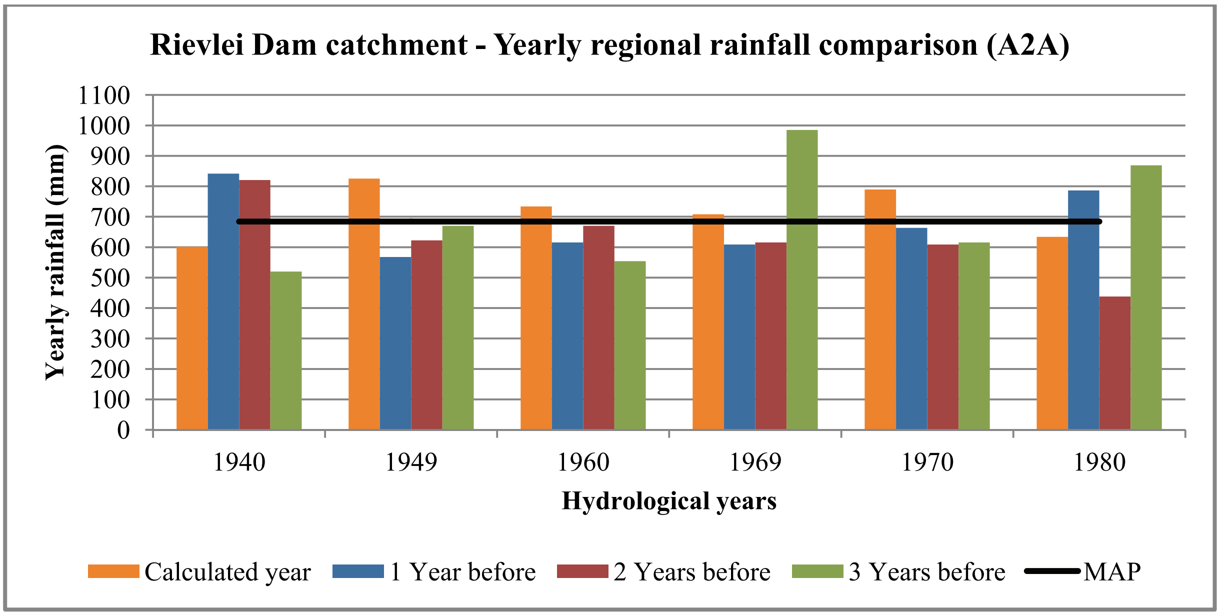


Figure 6-3: Yearly regional rainfall comparison for Rietvlei catchment

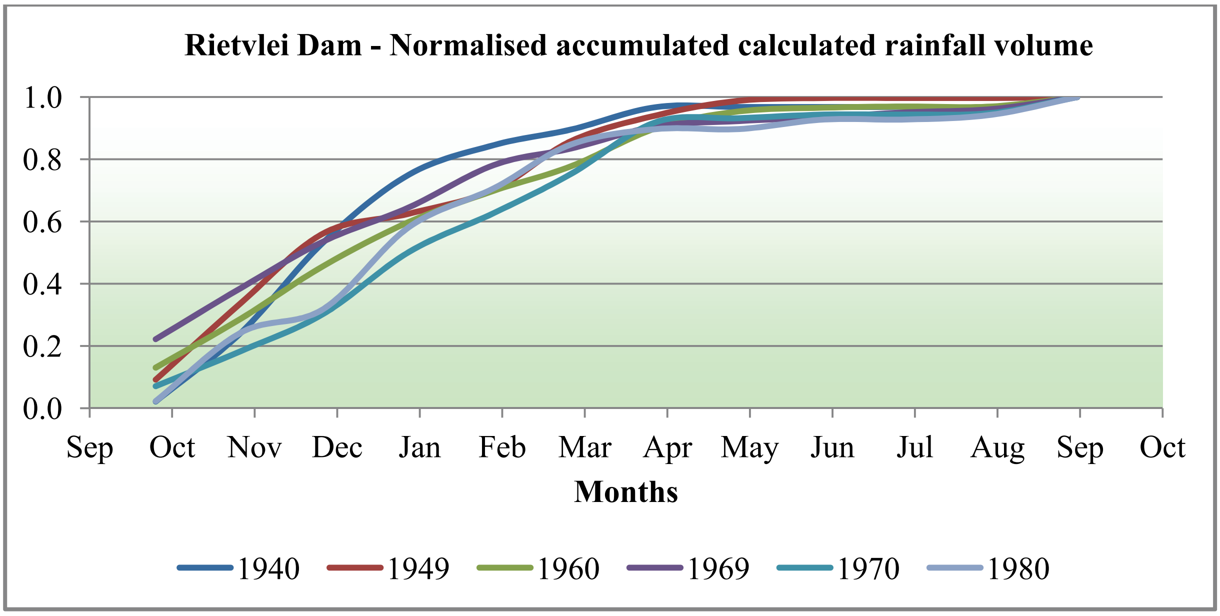


Figure 6-4: Normalised accumulated rainfall volume for Rietvlei catchment

Based on the assessment it was concluded that:

- The runoff for the hydrological year 1940 might be accentuated by the high rainfall in the preceding years. This record was however maintained for the comparison.
- The calculated rainfall for 1949 was higher than that of the other selected hydrological years and was eliminated from the comparison.
- The calculated rainfall for 1970 was higher than that of the other selected hydrological years and therefore it was excluded from the comparison.

The accumulated rainfall volume was then determined for those selected hydrological years (**Figure 6-5**). The percentage runoff was calculated from the total runoff volume (**Figure 6-6)** and the total calculated rainfall volume (**Figure 6-5**). This calculated percentage runoff was then plotted for the years in which development data was known **(Figure 6-7)**

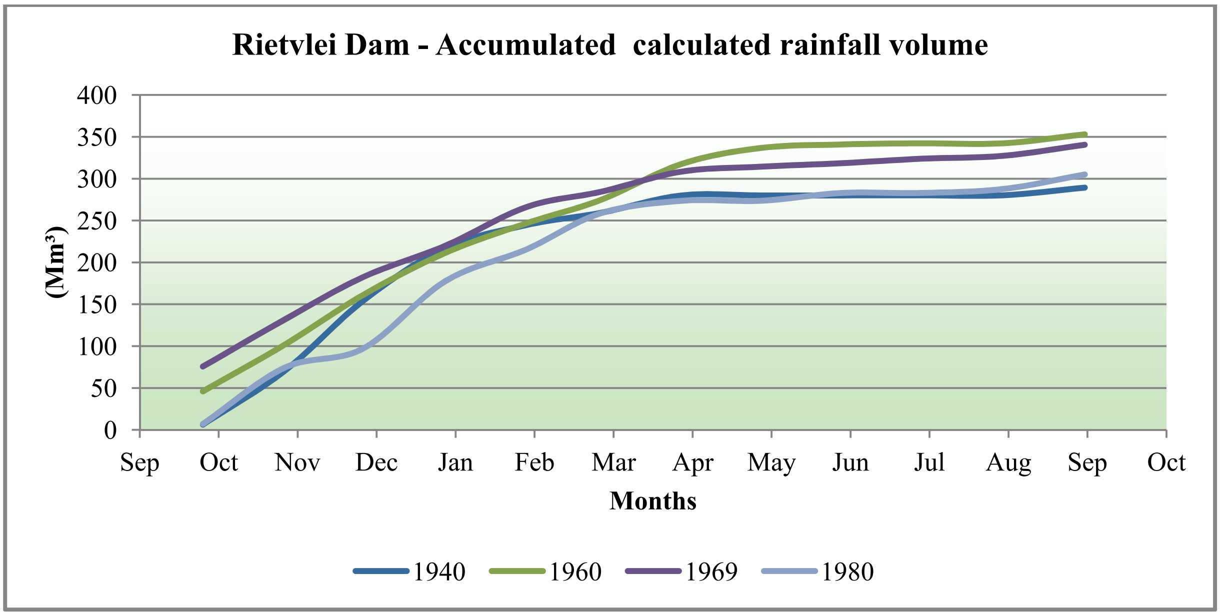


Figure 6-5: Accumulated calculated rainfall volumes for Rietvlei Dam catchment

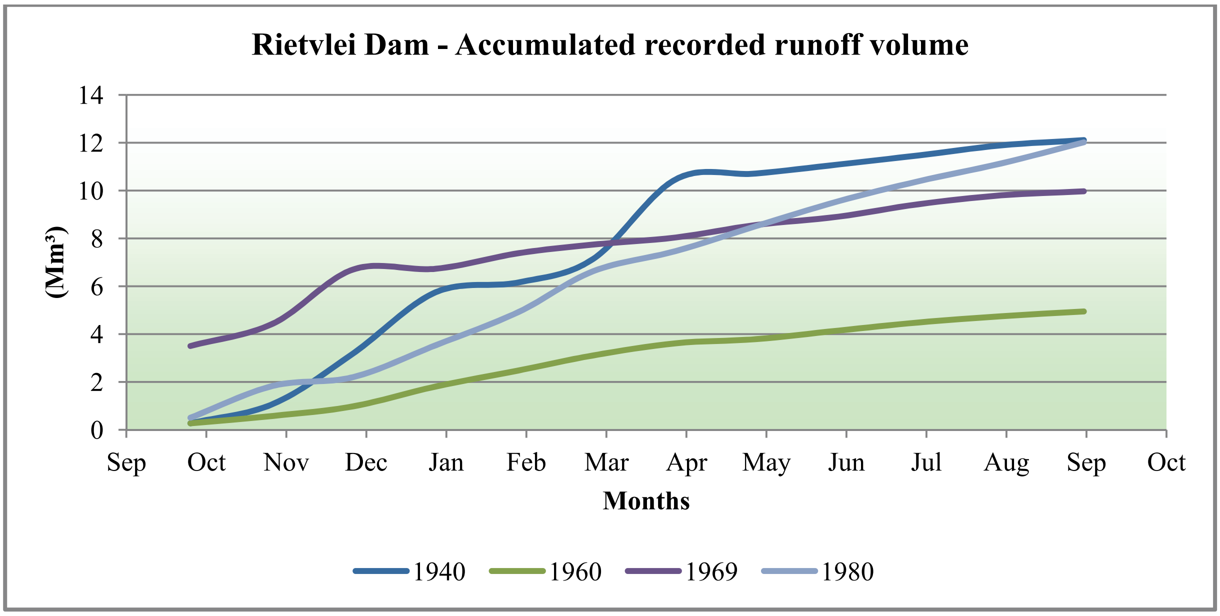


Figure 6-6: Accumulated recorded runoff volumes for Rietvlei Dam catchment

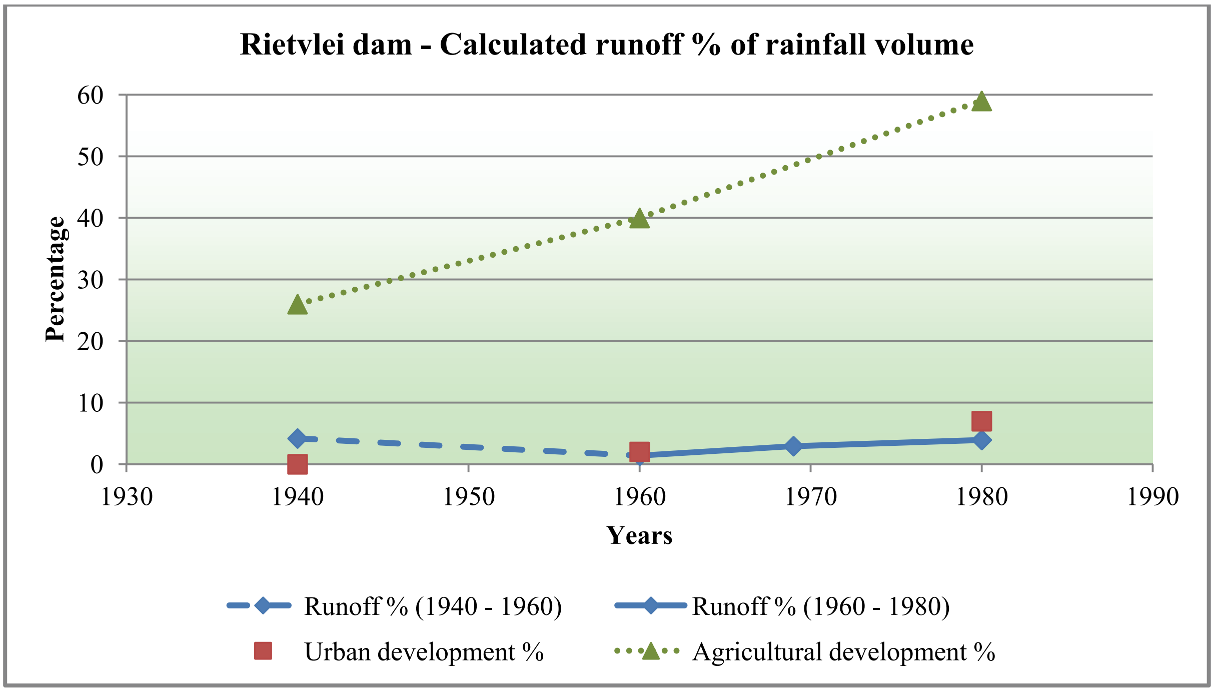


Figure 6-7: Final comparison of runoff as a percentage of rainfall with percentage urban and agricultural development of the Rietvlei catchment area

Reviewing the response from the Rietvlei Dam catchment, the results suggests that the runoff increased with an increase in urban- (0,4 % in 1952 to 6,1 % in 1983) and agricultural (26% in 1940 to 59 % in 1980) development in the catchment.

The percentage urban development is however small and the change in catchment response can therefore be attributed to the influence of agricultural activity in the catchment.

6.2 Daspoort catchment area (A2H007)

6.2.1 General information

Figure 6-8 indicates the location of Daspoort gauging station (A2H007) and the contributing catchment area.

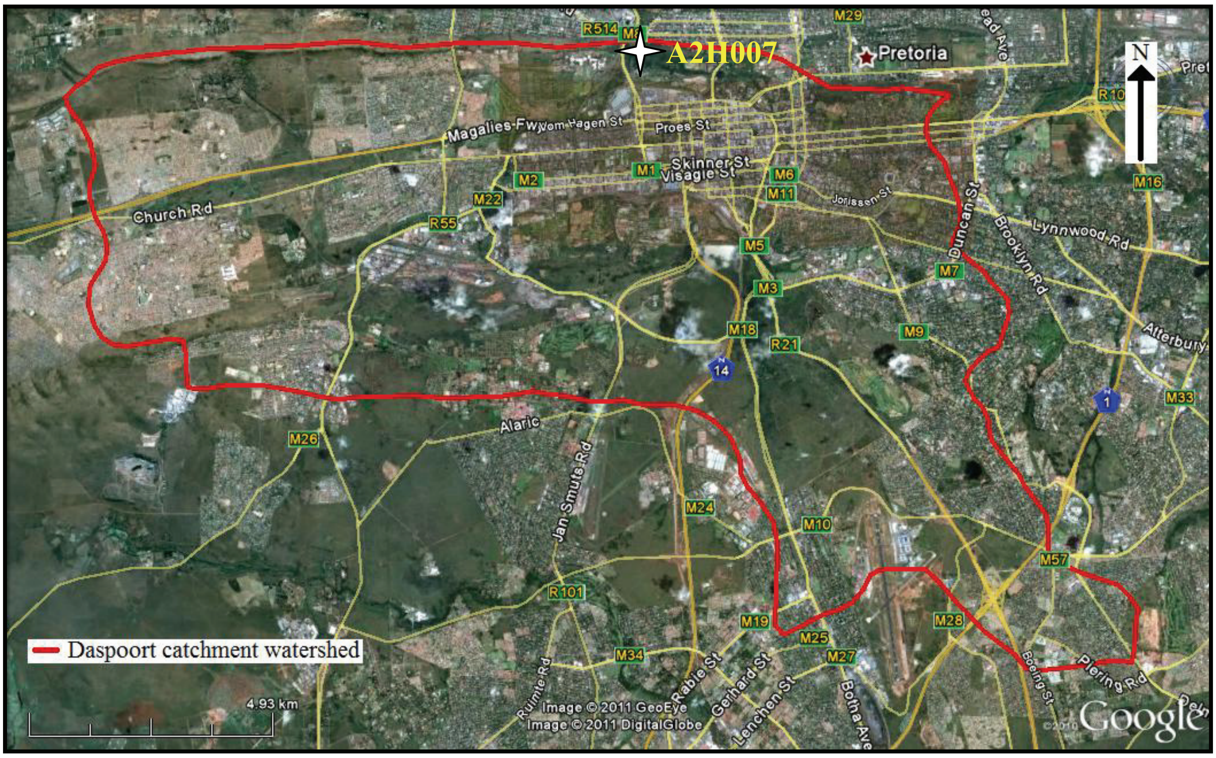


Figure 6-8: Catchment area of Daspoort gauging station (A2H007)

Table 6-4 provides some details of the Daspoort gauging station (A2H007) catchment.

Table 6-4: Main features of the Daspoort catchment

Parameter	Value	
Name	Daspoort	
Area of catchment	145 km2	
River	Apies River	
Rainfall area	A23D	
Gauging station	A2H007	
MAP	706 mm	102,37 Mm³
MAR	103 mm	14,93 Mm³
Period for flow records are available	October 1920 to September 1950	

The extent of the recorded flow data (1920 to 1950) limits the assessment to the development for this period. Urban development information was determined for 1939 and 1958 for which data of the land use could be obtained. The procedure which was used was discussed before in **Section 6.1.2**. **Figure 6-9** shows the urban development for 1939 and 1958 for the contributing area to the flow gauge station at Daspoort. Intensive urban development occurred in this catchment as is reflected in **Table 6-5**.

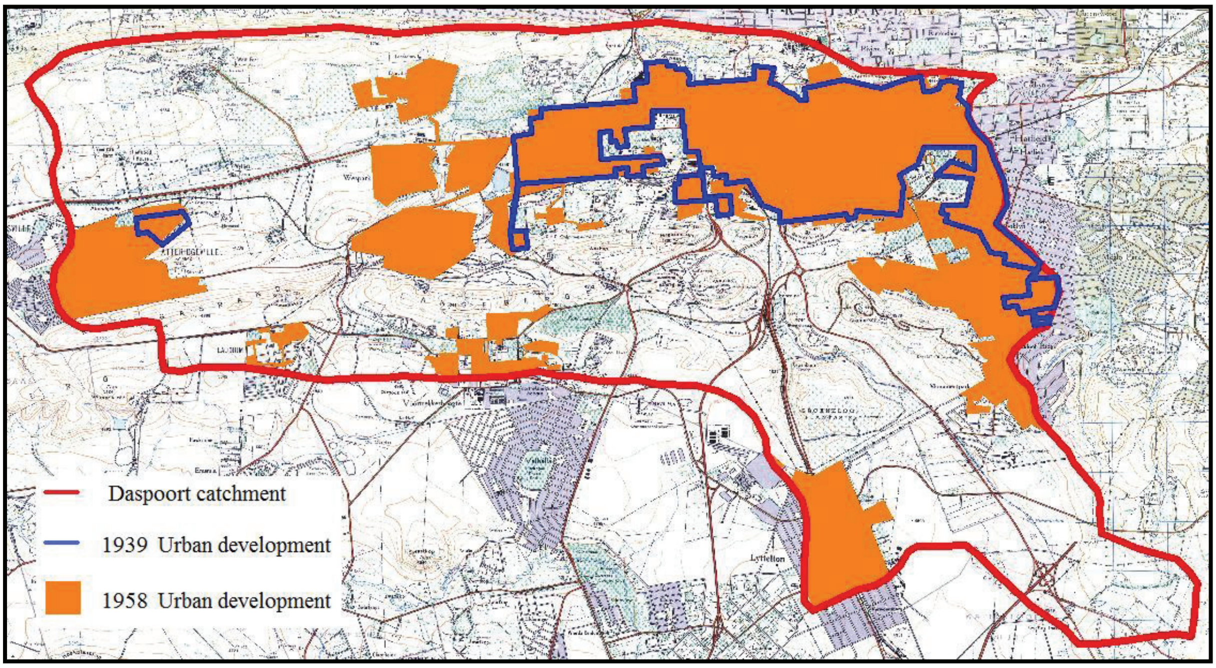


Figure 6-9: Urban development in the Daspoort catchment

Table 6-5: Details of the urban development in the Daspoort catchment

Review of the development in the Daspoort catchment area		
% of Catchment developed	**Year**	
	1939	1958
Urban development	12,62	26,54
Agricultural development	Not determined	
Rural state		

6.2.2 Methodology for the hydrological review of similar hydrological years for the Daspoort catchment

The methodology is similar to what was discussed in **Section 6.1.2**.

Details of the flow records, rainfall stations with recorded data and the return flows are reflected in **Appendix F** to **Appendix H**

The area allocations for each rainfall station are reflected in **Table 6-6**.

The flow data that was used in the comparison was obtained from the Surface Water Resources of South Africa 1990 Volume I (WRC 1990) published by the Water Research Commission . Due care was taken to use minimal sets of patched flow records. The flow records which were reviewed had patched data for the following months: February 1936, November and December 1939, November 1943 and January 1944

The Daspoort catchment area contains one WWTW called Daspoort, which was commissioned before 1920. The monthly WWTW flow volumes were obtained from the BKS report, "Assessment of the Water Availability in the Crocodile (West) River Catchment" (BKS, 2008) as shown in **Appendix H**

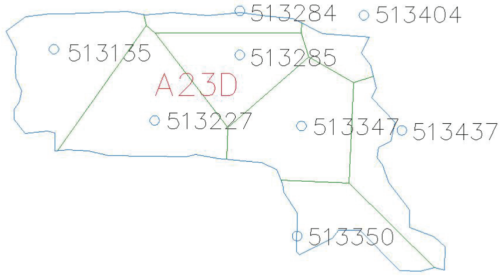
.

Table 6-6: Tabular and visual representations of area allocations of rainfall stations for the Daspoort catchment

Area contribution of rainfall stations for the Daspoort catchment	Visual representation of area allocations for rainfall stations in the Daspoort catchment

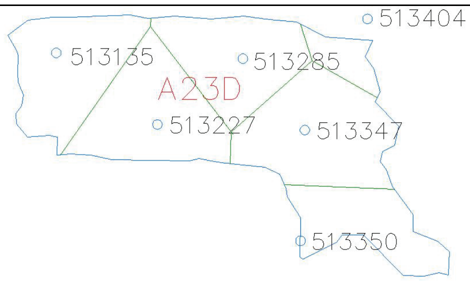
1935

Section	Position	% of catchment area allocated to rainfall station
513	135	19.2
513	227	20.7
513	284	4.6
513	285	13.6
513	347	18.4
513	350	10.1
513	404	4.0
513	437	9.4
		100.0



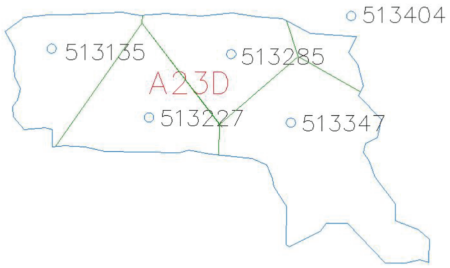
1939 and 1943

Section	Position	% of catchment area allocated to rainfall station
513	135	19.3
513	227	20.7
513	285	18.1
513	347	23.6
513	350	14.0
513	404	4.4
		100.0



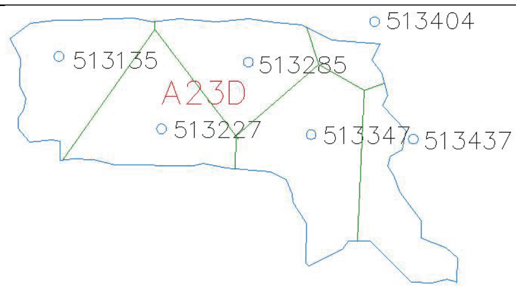
1943

Section	Position	% of catchment area allocated to rainfall station
513	135	19.3
513	227	20.7
513	285	18.1
513	347	37.6
513	404	4.4
		100



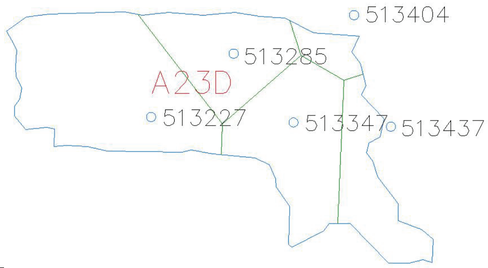
1945

Section	Position	% of catchment area allocated to rainfall station
513	135	19.3
513	227	20.7
513	285	18.1
513	347	24.0
513	404	4.0
513	437	13.9
		100



1947 and 1949

Section	Position	% of catchment area allocated to rainfall station
513	227	40.0
513	285	18.1
513	347	24.0
513	404	4.0
513	437	13.9
		100



The spatial and temporal distribution of rainfall and the antecedent catchment conditions were reviewed in a similar way to that of the Rietvlei catchment as was discussed in **Section 6.1.2**. **Figure 6-10** reflects the antecedent rainfall events and **Figure 6-11** reflects the temporal distribution of the normalised accumulated calculated rainfall for the selected hydrological years.

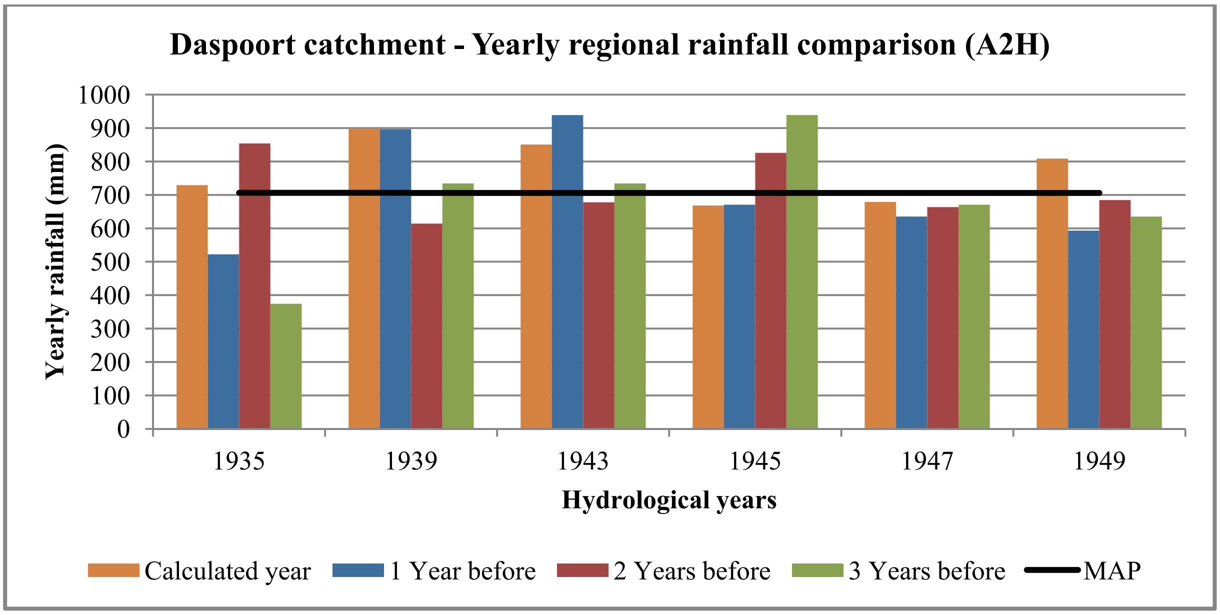


Figure 6-10: Yearly regional rainfall comparison for Daspoort catchment

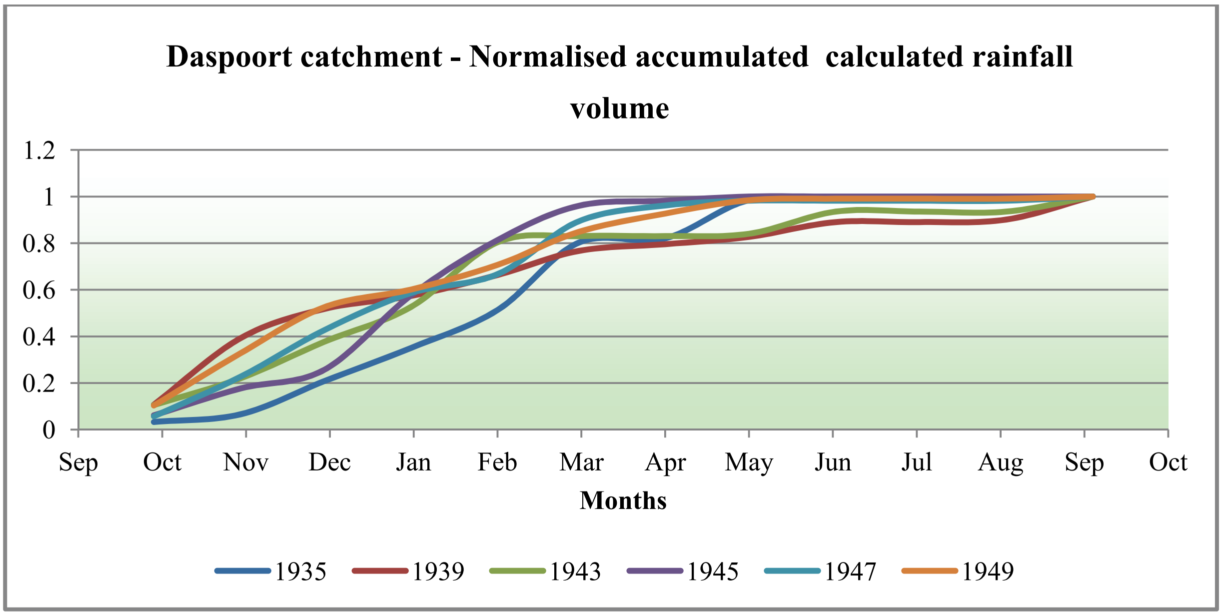


Figure 6-11: Normalised accumulated calculated rainfall volume for Daspoort catchment

Based on the assessment it was concluded that:

- The calculated rainfall for 1939 was higher than that of the other selected hydrological years and was dropped from the comparison.
- The calculated rainfall for 1943 was higher than that of the other selected hydrological years and was dropped from the comparison.
- The runoff for the hydrological year 1949 might be accentuated by the high rainfall in 1949 and 1948. This record was however maintained for the comparison.

The percentage runoff was calculated from the total runoff volume (**Figure 6-12**) and the total rainfall volume (**Figure 6-13**). This calculated percentage runoff is then plotted for the years in which development data is known **(Figure 6-14**).

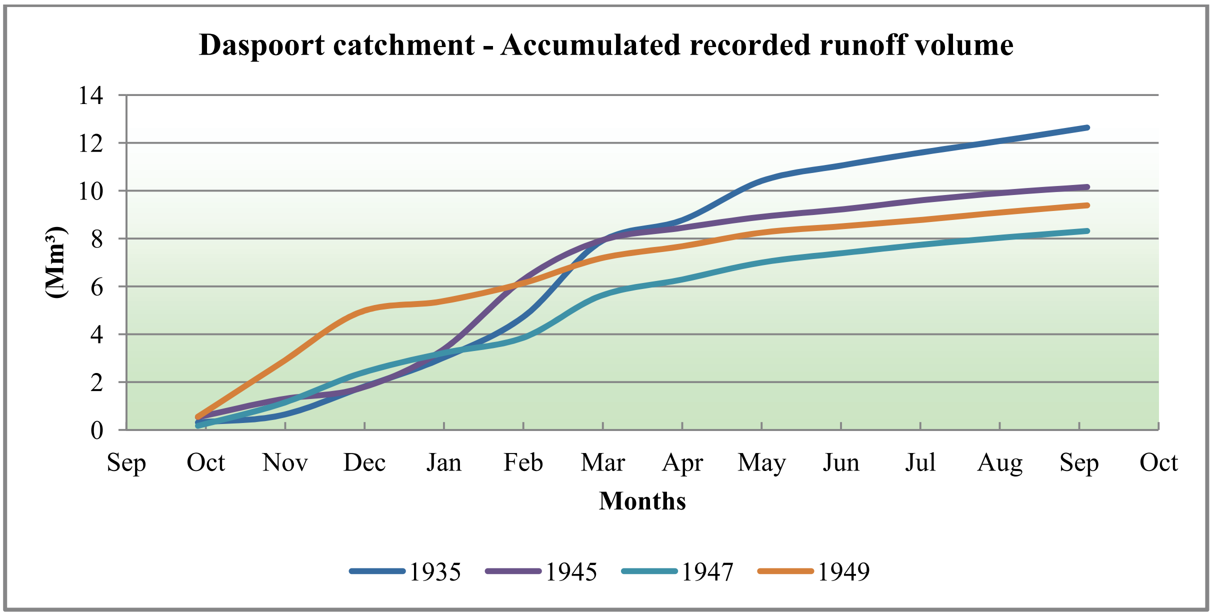


Figure 6-12: Accumulated recorded runoff volumes for Daspoort catchment

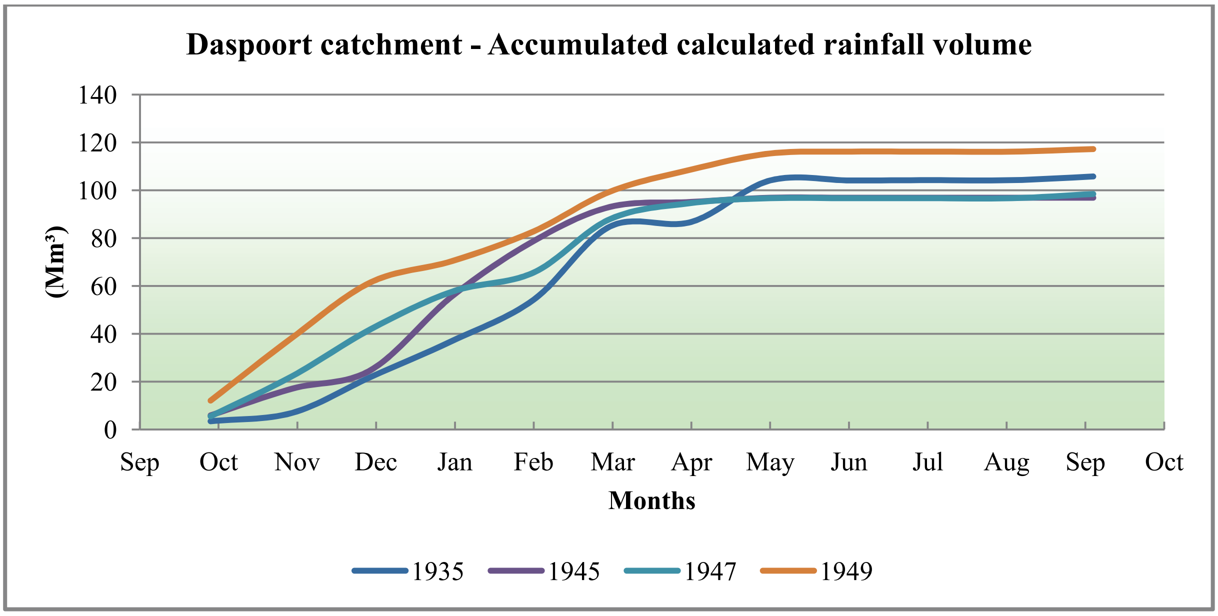


Figure 6-13: Accumulated calculated rainfall volumes for Daspoort catchment

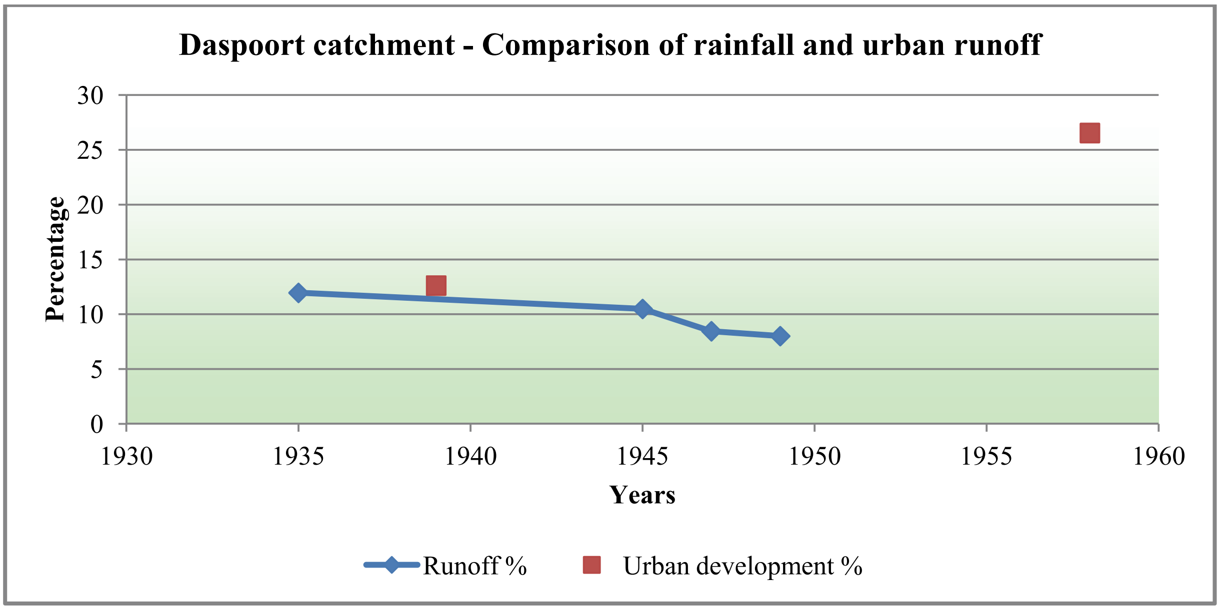


Figure 6-14: Final review of runoff as a percentage of rainfall and urban development percentage for Daspoort catchment

It is evident that the volume of discharge has not significantly changed during the time for which recorded data have been available although there was significant urban development during the period (Unfortunately the flow gauging station was closed in 1949). This decrease in runoff percentage could probably be ascribed to the increase in horizontal surfaces resulting from the development, which in turn causes an increase in infiltration into the ground water.

6.3 Hydrological review of similar hydrological years for the Kameeldrift catchment area (A2H028)

6.3.1 General information

Figure 6-15 indicates the location of Kameeldrift flow gauging station (A2H028) and the contributing catchment.

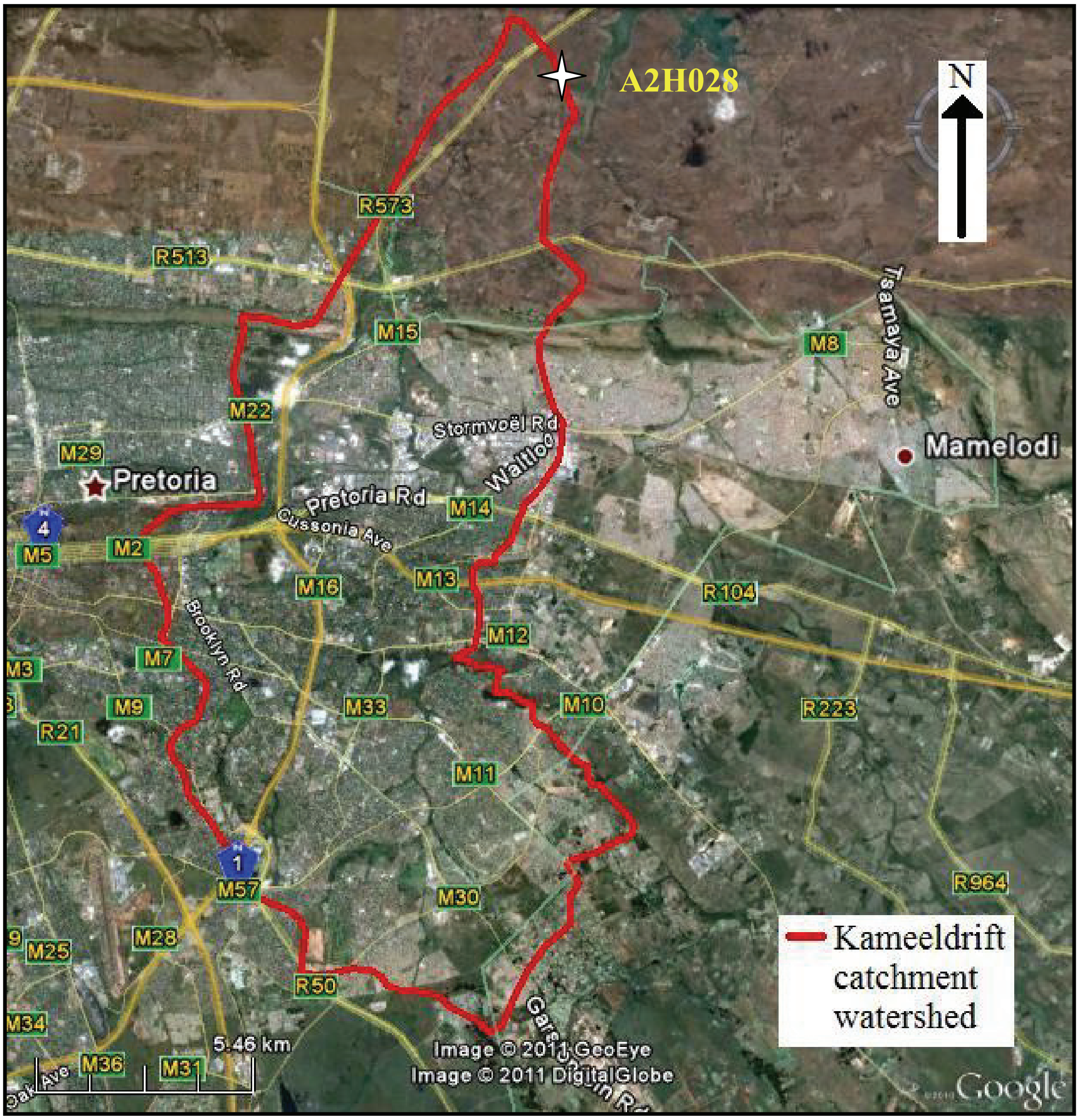


Figure 6-15: Catchment area of Kameeldrift gauging station (A2H028)

Table 6-7 provides some details of the Kameeldrift catchment area.

Table 6-7: Main features of the Kameeldrift catchment area

Parameter	Value	
Name	Kameeldrift	
Area of catchment	165 km2	
River	Hartbeesspruit	
Rainfall area	A23A2	
Gauging station	A2H028	
MAP	698 mm	115,17 Mm^3
MAR	55,3 mm	9,12 Mm^3
Period with recorded flow data	October 1961 to present *	

Note:

* *Data up till 1988 was reviewed*

The percentage urban development in the catchment was determined as discussed **Section 6.1.2**. The urban development levels for 1964 and 1975 are indicated in **Figure 6-16** with the details provided in **Table 6-7.**

The percentage urban development for the Kameeldrift catchment was determined using the following maps: 2528CA. 2528CB, 2528CC and 2528CD. The percentage urban development for 1975 had to be linearly interpolated for map 2528CD, as only the maps for 1958 and 1991 were available. In 1958 the urban development area was 6,7 km^2 and in 1991 the urban development area was 33,5 km^2. It was therefore assumed that the urban development area for 1975 on map 2528CD was 13,8 km^2. Thus the total urban development area was 34,9 km^2, which is 21.1% of the total catchment area.

Table 6-8 reflects the increase in urban development.

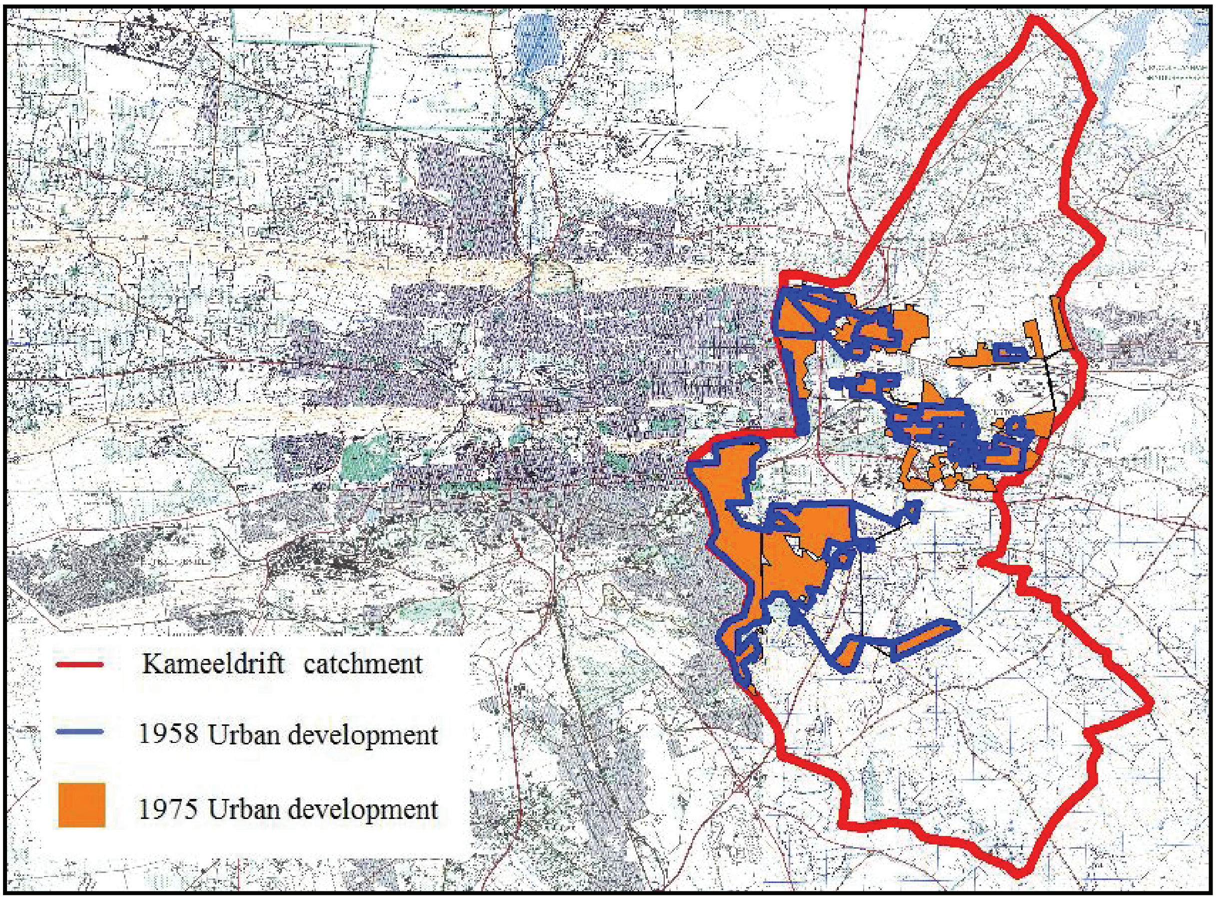


Figure 6-16: Kameeldrift catchment with indications of urban development *

Note:

* The full extent of 1975's urban development could not be shown visually in **Figure 6-16**, this is due to the fact that map 2528CD was not available for 1975

Table 6-8: Summary of Kameeldrift urban development percentages

<table>
<tr><th colspan="3">Urban development details of the Kameeldrift catchment area</th></tr>
<tr><th rowspan="2">% of Catchment developed</th><th colspan="2">Year</th></tr>
<tr><th>1964</th><th>1975</th></tr>
<tr><td>Urban development</td><td>10,68</td><td>21,13</td></tr>
<tr><td>Agricultural development</td><td colspan="2" rowspan="2">Not determined</td></tr>
<tr><td>Rural state</td></tr>
</table>

6.3.2 Hydrological review of similar hydrological years for the Kameeldrift catchment

6.3.2.1 Methodology

The methodology is similar to what was discussed in **Section 6.1.2**.

Details of the flow records, rainfall stations with useful data and return flows are attached in **Appendix F** to **Appendix H**.

6.3.2.2 Hydrological review of the Kameeldrift catchment

The spatial and temporal distribution of rainfall and the antecedent catchment conditions are two main drivers which could distort the relationship between runoff and rainfall. These parameters were evaluated as was indicated in **Section 6.1.2**

Table 6-9 reflects the rainfall stations which were used for the different selected hydrological years.

The flow data that was used in the comparison was obtained from the Surface Water Resources of South Africa 1990 Volume I book published by the Water Research Commission . Although due care was taken to use minimal sets of patched flow records, it was still necessary to use some years with patched data. The flow records which were reviewed had patched data for the following months: March 1969, February 1976, July 1984, August 1984, September 1984, October 1988 and July 1989.

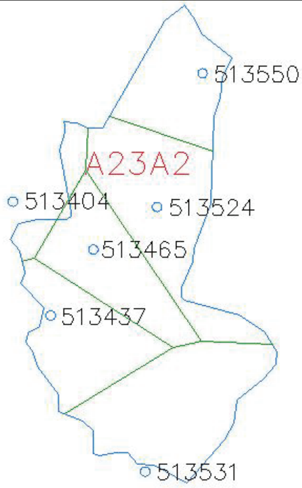
The recorded runoff record for the Kameeldrift catchment (A2H028) is not influence by any return flows from any wastewater treatment works (WWTW).

Table 6-9: Tabular and visual representations of area allocations of rainfall stations for the Kameeldrift catchment

Area contribution of rainfall stations for the Kameeldrift catchment	Visual representation of area allocations for rainfall stations in the Kameeldrift catchment

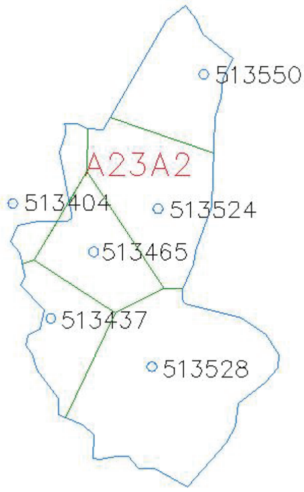
1964, 1968 and 1972

Section	Position	% of catchment area allocated to rainfall station
513	404	4
513	437	17
513	465	17
513	524	24
513	531	24
513	550	14
		100



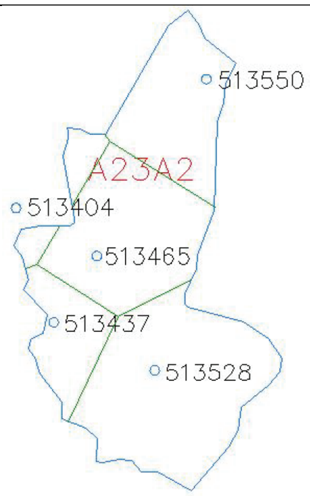
1975

Section	Position	% of catchment area allocated to rainfall station
513	404	4
513	437	11
513	465	13
513	524	20
513	528	38
513	550	14
		100



1983 and 1988

Section	Position	% of catchment area allocated to rainfall station
513	404	5
513	437	11
513	465	26
513	528	38
513	550	20
		100



The spatial and temporal distribution of rainfall and the antecedent catchment conditions are two main drivers which could distort the relationship between runoff and rainfall. These parameters were evaluated as was indicated in **Section 6.1.2**. **Figure 6-17** reflects the antecedent rainfall events and **Figure 6-18** reflects the temporal distribution of the normalised rainfall for the selected hydrological years.

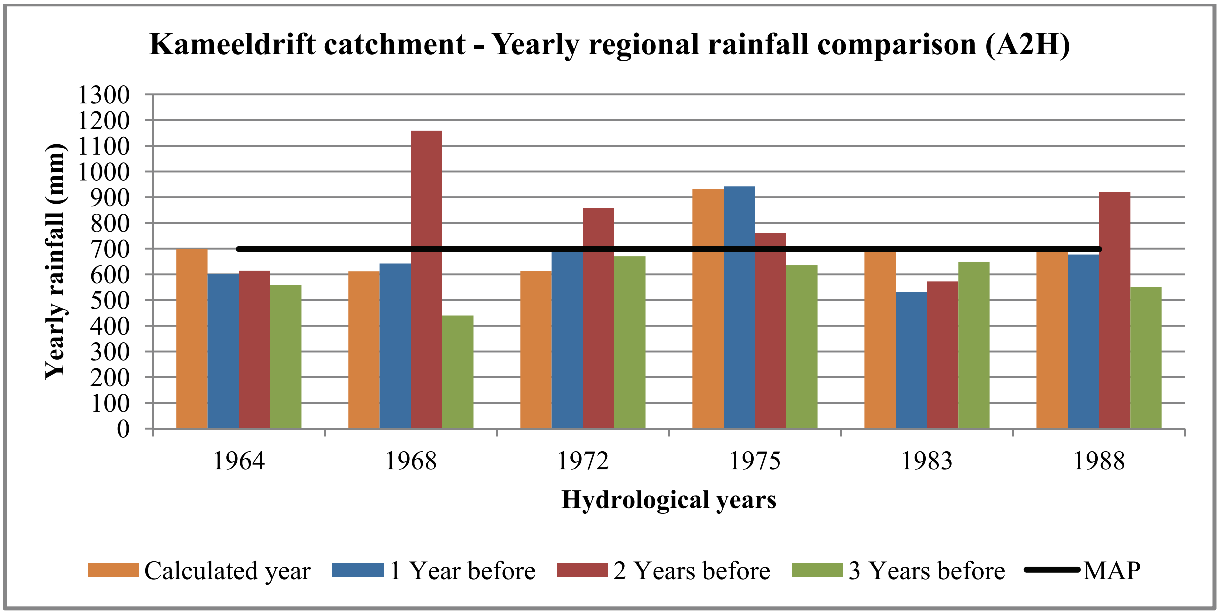


Figure 6-17: Yearly regional rainfall comparison for Kameeldrift catchment

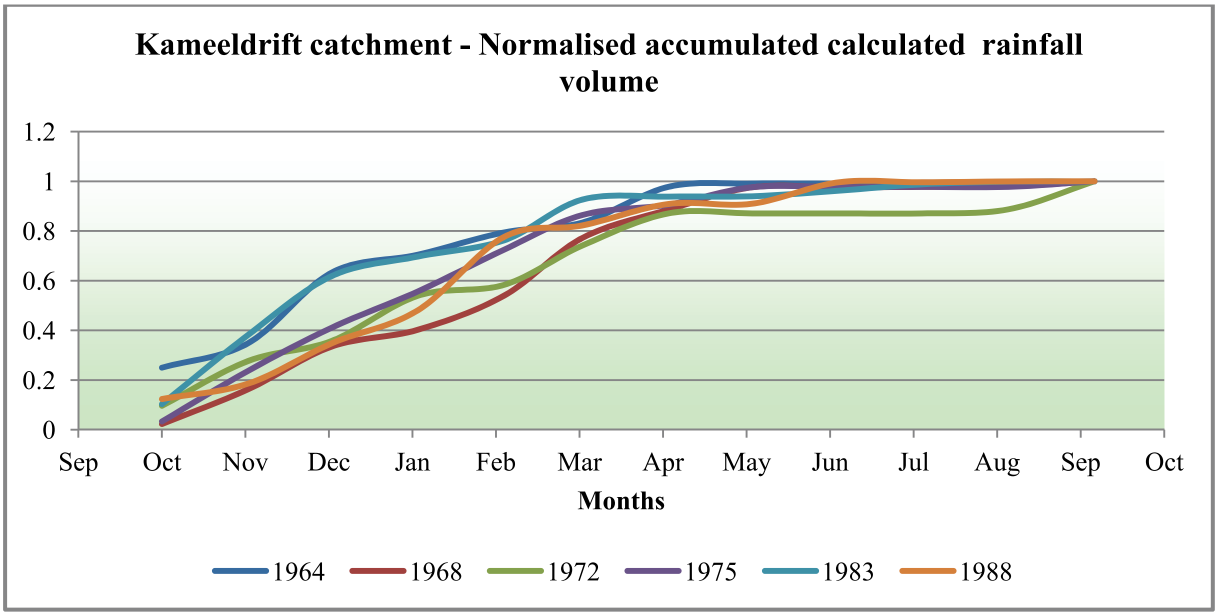


Figure 6-18: Normalised accumulated calculated rainfall volume for Kameeldrift catchment

Based on the assessment it was concluded that:

- The runoff for the hydrological year 1968 might be accentuated by the high rainfall in 1966. This record was dropped from the comparison
- The calculated rainfall for 1975 was higher than that of the other selected hydrological years and was dropped from the comparison.
- The runoff for the hydrological year 1988 might be accentuated by the high rainfall in 1986. This record was however maintained for the comparison.

The percentage runoff was calculated from the total runoff volume (**Figure 6-19**) and the total rainfall volume (**Figure 6-20**). This calculated percentage runoff is then plotted for the years in which development data is known **(Figure 6-21)**.

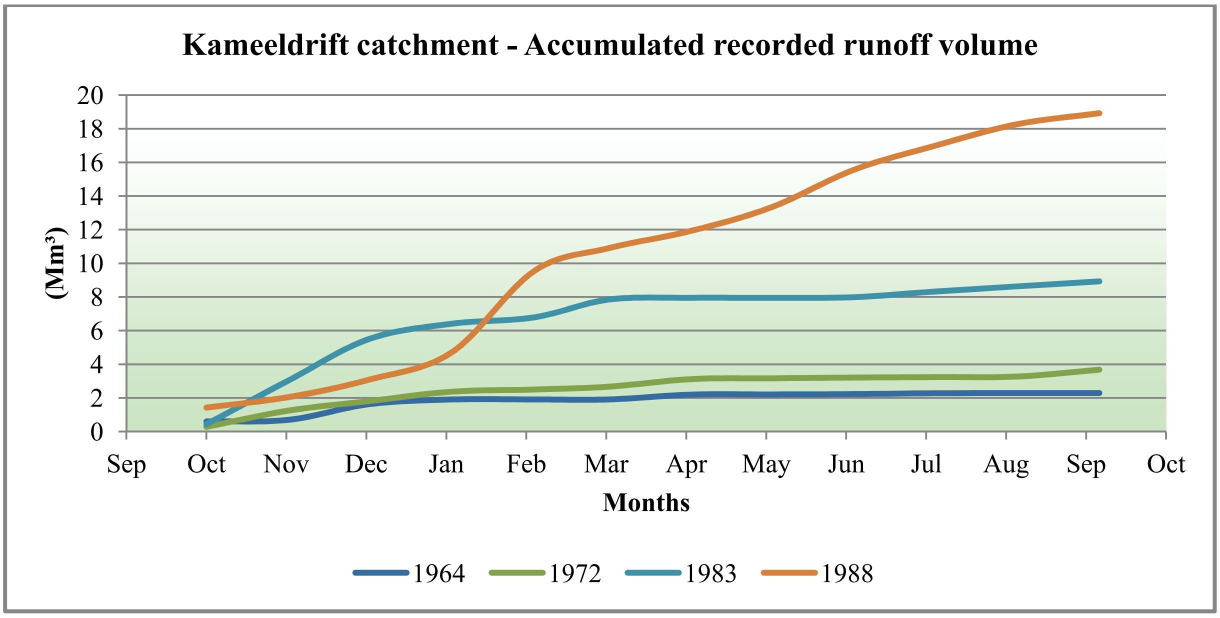


Figure 6-19: Accumulated recorded runoff volumes for Kameeldrift catchment

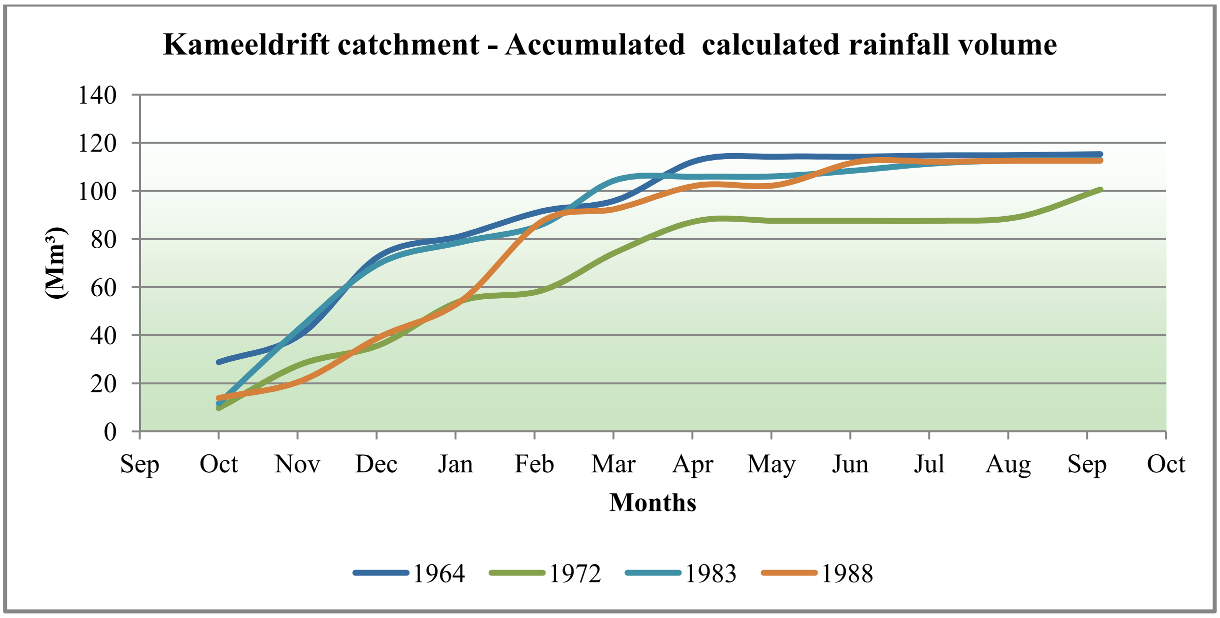


Figure 6-20: Accumulated calculated rainfall volumes for Kameeldrift catchment

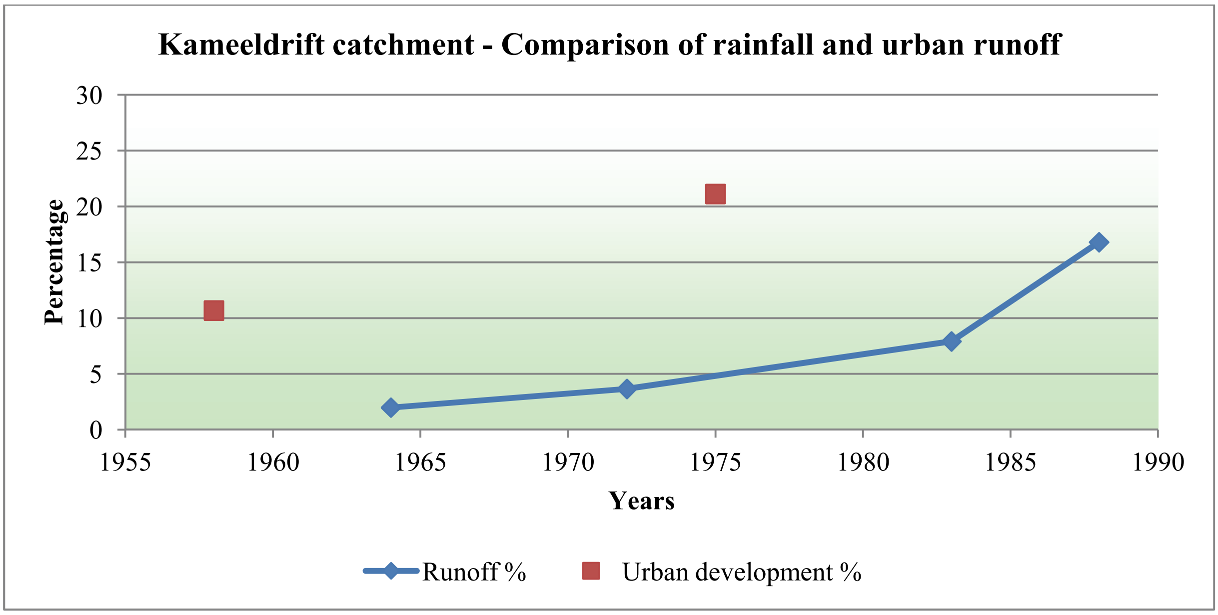


Figure 6-21: Final comparison of runoff as a percentage of rainfall and urban development percentage for Kameeldrift catchment

It is evident that the volume of discharge has increased significantly in this catchment for which the urban development area during 1975 was more than 20 %.

In **Figure 6-22** the influence of the three catchments are combined to provide an overview of the influence of development on runoff.

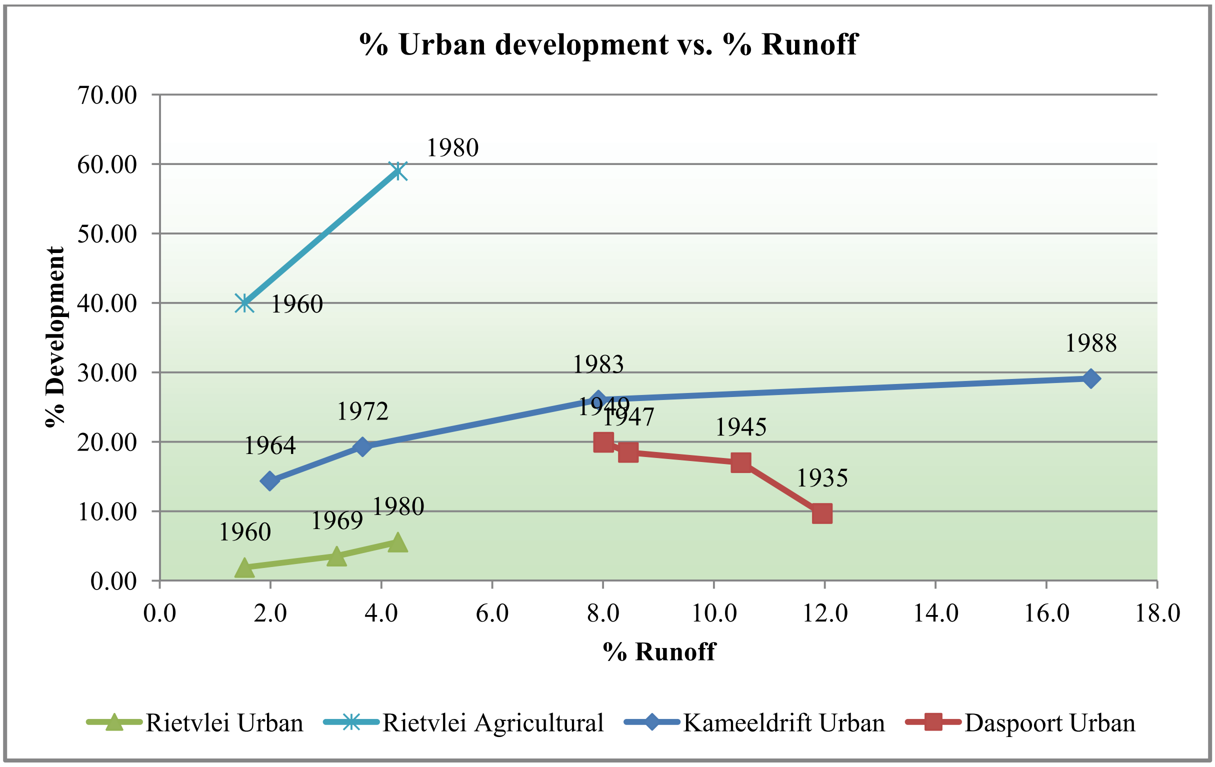


Figure 6-22: Final comparison of runoff as a percentage of rainfall for the different catchments

6.4 Conclusion

All the relationships indicate an increase in the runoff volume from the catchments due to an increase in urban development except in the case of the Daspoort catchment. It can be assumed that the rate of development decreased during the period of World War II (i.e. non-linear urban development), the results however indicate a higher runoff during 1945 in comparison to that experienced in 1947. The reason for this can be contributed by the temporal distribution of the rainfall in these two periods. Proof of this is found in **Figure 6-23** which clearly indicates the shortened period of rainfall which occurred in 1945, resulting in a larger runoff percentage (a runoff percentage increase of more than 2%).

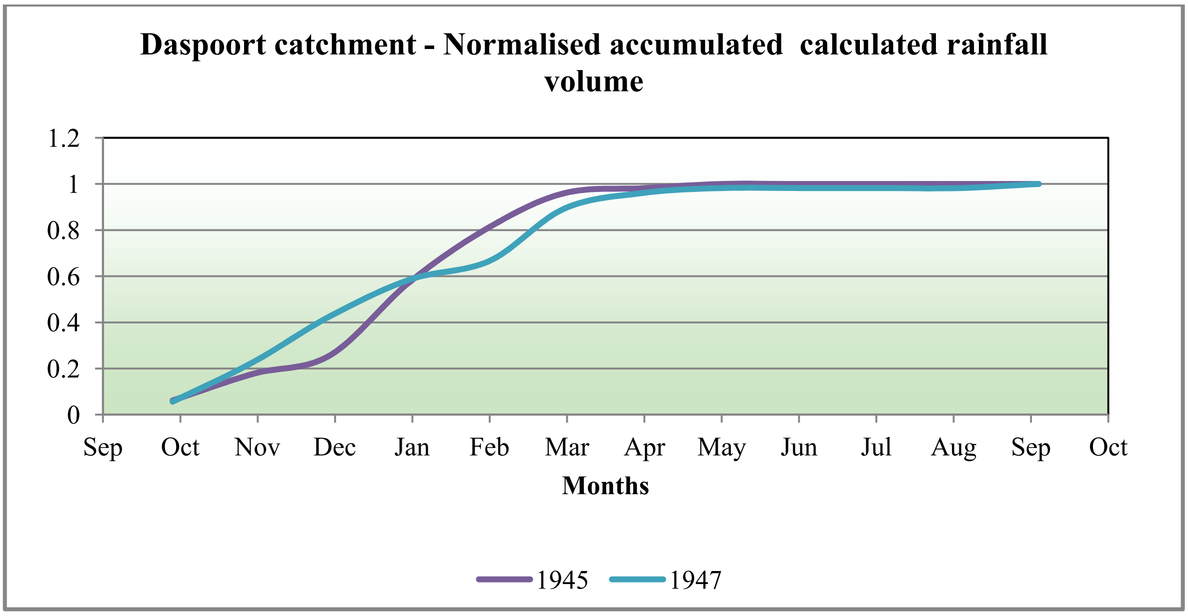


Figure 6-23: Final comparison of runoff as a percentage of rainfall for the different catchments

This is a significant finding, reflecting the importance of the autographic rainfall recordings and the influence of the antecedent conditions in the catchment.

7 Conclusion

This research project has neither proved with certainty nor disproved the hypothesis that an increase in certain specific urban development will decrease the peak runoff response.

The study identified trends which reflect that the relationship between the expected increases in urban peak runoff response resulting from urban development is less than the calculated peak discharge obtained from the use of the deterministic calculation procedures (Rational Method or other deterministic procedures).

The study identified that the effect of antecedent conditions is (rainfall) is not simply related to the influence on discharge in the following years. Possible causes for this could be related to soil properties and other hydrological parameters.

The effect of temporal distribution seems to have a greater effect on total volume of runoff, but the limited data which was reviewed is insufficient to verify the finding.

8 Recommendations

It is recommended that further research should be conducted, identifying and describing the **various types of urban developments**. With a definition of the urban development types, the catchment response should be assessed to be able to reflect:

- The effect of urban development on the runoff coefficient of a catchment;
- The effect of spatial and temporal distribution of the rainfall on the runoff response; and
- The effect of hydraulic structures, resulting in hydraulic routing (culverts, bridges and detention facilities), on the peak discharge in urban areas.

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List of Appendices on the enclosed CD

Appendix A: Additional literature of studies that have been conducted on the effect of urban development on runoff response

Appendix B: Details of hydraulic structures in the Willowspruit catchment in 2008

Appendix C: Computer coding (Visual Basic)

Appendix D: Rainfall data at Ramkie-oor-Willows, Irene WO and Pretoria Eendracht

Appendix E: Rainfall data at Robert's Place

Appendix F: Selection of analysis years based on regional rainfall distributions for assessment of Rietvlei, Daspoort and Kameeldrift

Appendix G: Recorded data periods for rainfall stations in Rietvlei, Daspoort and Kameeldrift catchments

Appendix H: Return flow data for the WWTW in the Rietvlei and Daspoort catchments