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Review of TP10 Water Quality Volume Estimation

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Reviewed by:	Approved for ARC publication by:
	
Name: Bodo Hellberg	Name: Judy-Ann Ansen
Position: Technical Specialist Stormwater Technical Services	Position: Acting Team Leader Stormwater Action Team
Organisation: Auckland Regional Council	Organisation: Auckland Regional Council
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Review of TP10 Water Quality Volume Estimation

Asaad Y. Shamseldin

Prepared for

Auckland Regional Council

Department of Civil and Environmental Engineering
University of Auckland
Private Bag 92019
Auckland Mail Centre
Auckland, 1142
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List of Symbols and Abbreviations

ARC	Auckland Regional Council
ATC	Atlanta Regional Commission
C	Runoff Coefficient
CCC	Christchurch City Council
CDF	Cumulative Probability Density Function
CEP	Centre of Watershed protection
CM	City of Maryville
IA	Fraction of impervious area
N	Total number of rainfall storm events
NIWA	National Institute for Water and Atmospheric science
No	Total number of overflow events
NSWG-	New South Wales Government
NT	Northeast Tennessee
NZWRF	New Zealand Water Environment Research Foundation
P	Non-exceedance Probability
R	Runoff capture ratio
SCS	Soil Conservation Service
SMRC	Stormwater Manager's Resource Center
TP	Technical Publication
TSS	Total Suspended Solids
WQSD	Water Quality Storm Depth
WQV	Water Quality Volume

Executive Summary

In broad terms, this report is concerned with reviewing the basis of the water quality design of stormwater treatment devices outlined in the Auckland Regional Council (ARC) technical publication TP10 (ARC, 2003). TP10 provides general guidance about the design approaches required to deliver both water quantity and water quality benefits.

In TP10, the water quality sizing of stormwater treatment devices is based on the Water Quality Volume (WQV) criterion required to achieve a certain level of suspended solid removal.

The objective of this report is to update the TP10 WQV criterion. It is intended that the outcomes of this report will contribute to robust, defensible computational procedures to enable better stormwater management design.

The report is divided into two parts. The first part provides a literature review of the various methods which have been used to estimate WQV. The objective of this part is to recommend to the ARC potential WQV estimation methods. The second part of the report provides the results of a case study obtained from the application of the potential methods to Auckland data.

1 Review of the Water Quality Volume Estimation Methods

1.1 Introduction

This section provides a literature review of six WQV estimation methods; outlining their advantages and disadvantages. These methods are:

- TP10 WQV method,
- Half-inch (first-flush) rule,
- One-inch rule,
- Rainfall capture rule,
- Maximized detention volume and
- Pitt method.

The main objective of the review is to recommend to ARC potential methods which can be used for WQV estimation.

1.2 The TP10 procedure for estimation of the Water Quality Volume (WQV)

The current TP10's WQV procedure was originally proposed in the ARC Technical Publication, TP4 (ARC, 1992) on the basis of a study conducted using the data from a rainfall gauging station located in the Botanic Gardens. It is based on the specification of the Water Quality Storm Depth (WQSD) which is taken as 1/3 of the 2-year 24- hour rainfall depth as determined from the rainfall maps of TP108 (see ARC, 2003, 1999, 1992). The corresponding stormwater runoff volume is calculated using a locally adapted version of the Soil Conservation Service (SCS) curve number approach (ARC, 1999). This volume is known as the Water Quality Volume (WQV). TP10 stipulates that designing stormwater management devices based on WQSD enables 80% of the long term runoff volume from all the storms to be captured and treated. This gives a water quality control goal of 75% removal of Total Suspended Solids (TSS). The choice of this goal is also based on the Botanic Gardens study which found that the removal of 75% TSS is at the marginal point of diminishing return for sediment removal versus device size, i.e. aiming for a higher degree of removal would require an undue increase in treatment device size and therefore cost. In TP10, TSS is used as an "indicator" pollutant and such a use is very well established in many storm water management manuals (ATC, 2000; NT 2008; CM, 2008). The implied assumption here is that the control of TSS leads to indirect control of other stormwater runoff pollutants (NT, 2008).

1.3 Half-inch (First-Flush) Rule

This rule is based on the first flush concept aimed at capturing the first portion of stormwater runoff which is assumed to contain the majority of pollutants. The essence of the first flush concept is that pollutants deposited on exposed surfaces can be entrained by rainfall-runoff processes, with the first part of stormwater runoff being the most polluted containing disproportionately high concentration or pollutant mass (NSWG-Australia, 2009). The concept of first flush is not new and its origin can be traced back to 1910s (Metcalf and Eddy, 1916). However, the existence of the first flush concept is very controversial. As noted by Bertrand-Karjewiski et al. (1998) it has been the subject of hot debate between "those who have seen" and "those who do not believe in it". While there are many studies which confirm the existence of the first flush (Li-qing et al. 2007; Kang et al., 2008; Kim et al., 2005; Line et al., 1997) there are also many studies which confirm its non existence (Suarez and Puertas 2005; Saget et al., 1995; Pratt and Adams, 1984). Maestre et al. (2004) found that the first flush may not exist in all landuse types. NSWG-Australia (2009) has outlined some of the reasons why the first flush may not be observed. These reasons are;

- "*The drainage characteristics of the catchment may prevent it. Particularly in large catchments, initial runoff from the most distant parts of the catchment may not reach the catchment outlet for some time after a storm starts. This time lag is rarely an issue for smaller, individual premises.*
- *The pollutants may not be very mobile. Rainfall does not remove some pollutants, like oils and greases, as easily or as quickly as soluble materials and fine dusts. Bare soils or vegetated surfaces are generally not 'cleansed' as easily or effectively as sealed surfaces.*
- *Pollutant sources that are effectively continuous may exist within the catchment. First flush is generally seen only where the supply of pollutants is limited. Sediment generated from soil erosion, for example, will not give a first flush because the supply of soil particles is (for all practical purposes) unlimited. In cases like this, on-line, flow-through pollution controls will be needed.*
- *In urban catchments during large storms, continuous discharges from sewer overflows may mask any first flush associated with stormwater runoff*".

One of the main conceptual difficulties with the first flush concept is the ambiguity about how to define and characterise what constitutes a first flush. There are various ways of defining the first flush (Deletic, 1998) with no universal consensus about the best definition. A first flush can be identified when a significant concentration peak occurs at the beginning of storm events (Thornton and Saul 1986; Deletic 1998; Deletic and Maksumovic 1998; Gupta and Saul 1996). Drapper et al. (2000) consider a first flush has occurred when the concentration of the first 20-L of stormwater runoff is higher than the event mean concentration. However, such a definition can be location specific and many not be valid for other locations. Sansalone and Buchberger (1997) noted the first flush exists when the cumulative pollutant mass curve is above the runoff volume curve. Saget et al. (1995) defines the first flush as occurring when at least 80% of the total pollutant load is transported in the first 30% of the stormwater runoff volume. Wanielista and Yousef (1993) define the first flush as the transporting of at least 50% of the constituent mass in the first 25% of the runoff volume. Vorreiter and Hicky (1994) define the first flush as the pollutant load in the first 25% of the runoff volume. Likewise, Deletic (1998) defines the first flush as the percentage of pollution contained in the first 20% of the runoff volume. From the above definitions, it can be seen that the first portion of the stormwater runoff containing the significant fraction of the pollution is ill-defined. This gives rise to difficulties in assimilating the results of different research studies (Deletic, 1998).

The first flush is a very complex phenomenon and depends on the rainfall and runoff characteristics which include (Deletic, 1998);

- Climate characteristics, specifically, antecedent dry weather conditions
- Rainfall characteristics such as depth, duration and maximum intensity
- Runoff quantity characteristics, namely, event volume and maximum runoff rate
- Runoff quality characteristics, namely, pH, conductivity, suspend loading rate

In New Zealand, Christchcurch City Council uses the first flush concept to size stormwater devices. The Christchurch City Council's Waterways Wetlands and Drainage Guide (CCC, 2003) recommends the capture of runoff from the first 25 mm of storm rainfall depth, but not less than 15 mm from hardstand areas with its use being limited to the design of ponds and wetlands (NZWRF, 2004). However, there are variations to the rainfall depth to be captured and treated. For example, "*Environment Canterbury consent CR C000315 granted to the Christchurch City Council for green field development in the Upper Heathcote/Wigram area requires the capture and treatment of the first 12.5 mm of all rainfall events prior to discharge to ground. This first flush interception will achieve treatment of 58% of the Christchurch average annual rainfall depth falling on the recipient catchment*" (NZWRF, 2004). Recent studies carried by Zollhoefer (2009) on a 10-year old, 6.1 ha residential development in Christchurch, New Zealand indicated that capturing the 25 mm first flush depth may be very conservative. However, there is a need for further investigation at different locations and land uses in New Zealand to confirm the generality of the results obtained by Zollhoefer (2009). In Taiwan, Chang et al. (2008) found the first 6 mm and 10 mm of runoff contains 60% and 80% of non-point source loads in two industrial parks. This clearly highlights the need for defining the first flush depth in conjunction with the fraction of pollution load it carries.

There are also variations to the half-inch rule depending on how the stormwater runoff volume is calculated. For example, CWP (2005) noted the water quality volume based on the first flush concepts can be calculated as one-half inch times the impervious area of the site. The WQV calculated in this way provides an incentive to reduce impervious cover although it may not be adequate to provide adequate treatment for a significant fraction of the annual pollutant load (CWP, 2005).

1.4 One-Inch Rule

This is a very arbitrary rule without any theoretical background (CWP, 2005). According to this rule, WQV is calculated as the storage capacity needed to capture and treat a portion of stormwater runoff from the developed areas of the site produced from 1 inch of rainfall. The WQV is calculated by multiplying the rainfall depth by a runoff coefficient (C) depending of the fraction of impervious area (IA) according to the following equation (DCP, 2005)

$$C=0.05+0.9\ IA \qquad (1)$$

Thus, if the catchment is totally impervious then the runoff coefficient would be equal to 0.95. This equation provides incentive to reduce the impervious area as increase in the IA value results in high WQV values and hence larger storage facilities. Although this method is arbitrary and produces "one size fits all", it has the advantage that it simplifies analysis and reduces potential computational errors arising for the selection of the corresponding rainfall depth (CWP, 2005).

1.5 The Rainfall Capture Rule

This is one of the popular methods for estimating WQV. It is based on the long term spectral frequency analysis of storm event rainfall depth. This spectral analysis is very similar to that conducted when developing a flow duration curve. In the spectral analysis, the rainfall storm events which do not produce runoff are eliminated from the analysis. A non-parametric frequency analysis is then used to construct a relationship between the non-exceedance probability and the storm event rainfall depth. The curve typically shows a sharp curvature (knee/inflection point). A rainfall depth at a point on the knee typically between the 85th percentile and the 90th percentile rainfall depth is normally selected as WQSD and used to calculate WQV. Figure (1) shows an example of a probability plot of the event rainfall depth. In this figure, the 85th percentile rainfall depth has been chosen as WSQD to be used for WQV calculations "*because, it represents the "knee in the curve" volume that captures a significant number of storms without attempting to treat the small percentage of much larger storms that result in large volumes of runoff. Such storms would be expensive to treat, are rare in occurrence, and typically diluted in pollution concentration*" (NT, 2008). Hence, optimization of the device size is implicitly taken into consideration. The main difficulty in applying this method is that there is no agreed definition of what constitutes a storm. In the case where there are a number of rainfall events occurring very close to each other, there is a lot of subjectively in determining whether or not these events should be treated as a single storm event or as a succession of individual storm events. In the USA, Driscoll et al. (1989) found that a 6-hour separation time between storms produced the most consistent results. It is worth noting that WSQD values have also been determined using the daily rainfall which would eliminate some of the subjectivity associated with the determination of what constitutes a storm (SMRC, 2009).

One of the major concerns raised regarding this method is that the percentile of the rainfall depth will not automatically translate to an equivalent percentage of annual runoff volume capture (SMRC, 2009). For example, WSQD based on the 90th percentile rainfall depth does not guarantee that 90% of the annual runoff volume will be captured and treated. This is mainly due to the non-linearity in both the rainfall-runoff transformation process and the relationship between the event magnitude and the probability of non-exceedance as shown in

Figure 1. However, studies carried out in the USA by the Centre of Watershed Protection using rainfall data from New York, Vermont and Georgia, indicated that the percentile of the rainfall depth gives a close approximation to the percentage of annual runoff capture (SMRC, 2009). This close approximate is not a universal rule and its validity should be checked for different climates.

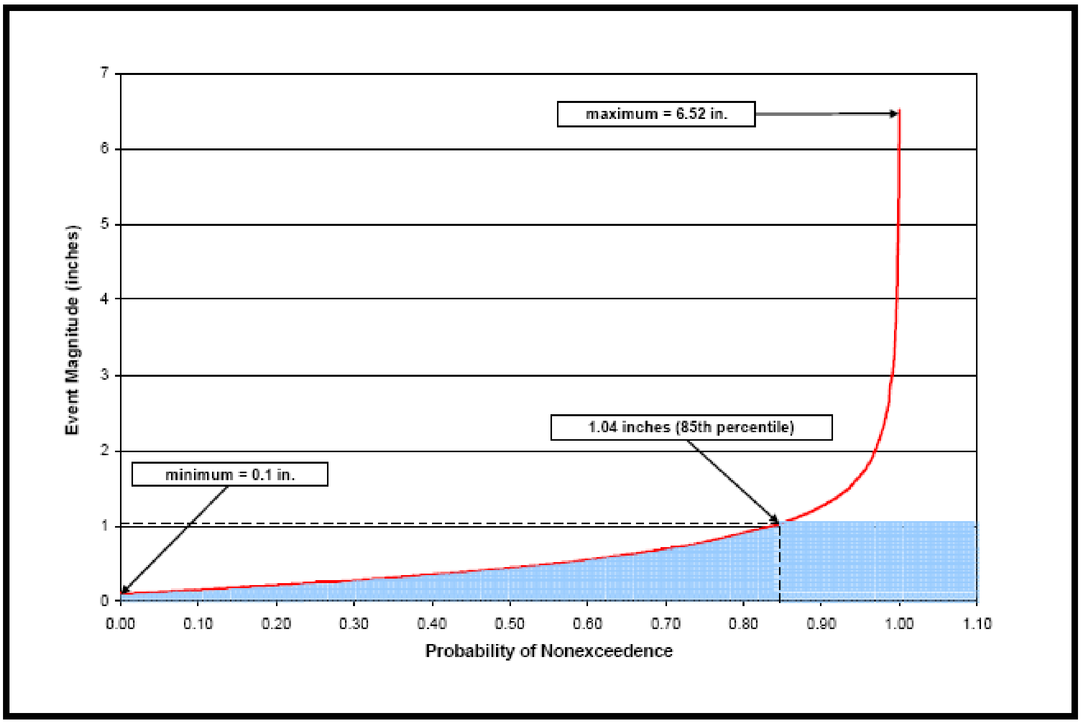


Figure (1): Rainfall frequency curve (after NT, 2008).

1.6 Maximized Detention Volume

This procedure was originally introduced to determine the stormwater quality detention volume (Urbonas et al., 1989; Guo and Urbonas, 1996). This volume is estimated based on finding the point of diminishing return at which further increase in the detention volume does not produce a significant increase in the number of events or the runoff volume captured. This volume is often referred to as the maximized detention volume. This procedure is very similar to that used in the current TP10 (ARC, 2003).

Guo and Urbonas (1996) noted that stormwater quality management design criteria based on traditional flood control procedures may not provide optimal quality treatment of storm water runoff. Designing a stormwater quality control facility to accommodate the more rare and less frequent storms does not ensure that the facility will provide the appropriate detention time storage required to effectively mitigate the impact of the stormwater runoff on the receiving waters (Guo, 1993). Guo and Urbonas (1996) noted that the 2-year storm, which is often used in flood and drainage management, produces runoff larger than 95% of the events that may take place over an urban catchment.

Furthermore, they argued that if the detention volume is too small, a large number of storms may exceed the facility capacity. However, if the detention volume is too large, smaller events may flow faster through the facility than required for adequate removal of pollutant. Hence, there is a detention volume threshold after which the removal of sedimentation becomes negligible. They also argued that this threshold is dependent on the smaller and more frequent storms which constitute the majority of the storms at a given location. The operation of this procedure is based on identifying the runoff producing storms from a continuous record of rainfall data based on a chosen measure of storm separation time. The runoff producing storms are those which have rainfall depths greater than the incipient rainfall (i.e. initial abstraction). The runoff volume is obtained by multiplying the excess rainfall depth by a runoff coefficient. The runoff volume in conjunction with facility operation rules are used to identify the overflow events. The effective/performance of the facility is determined by calculating the runoff capture ratio R according the following equation;

$$R = 1 - \frac{No}{N} \qquad (2)$$

where N is the total number of events and No is the total number of overflow events exceeding the design capacity of the facility. The runoff capture is repeated for a range of detention volumes. The detention volumes are then normalized by dividing the detention volume by the maximum runoff depth. Figure (2) shows a plot of runoff capture ratio versus the normalized detention volumes which enables the identification of the point of maximization (diminishing return) where further increase in the facility of the device doesn't produce significant increase the performance of the facility.

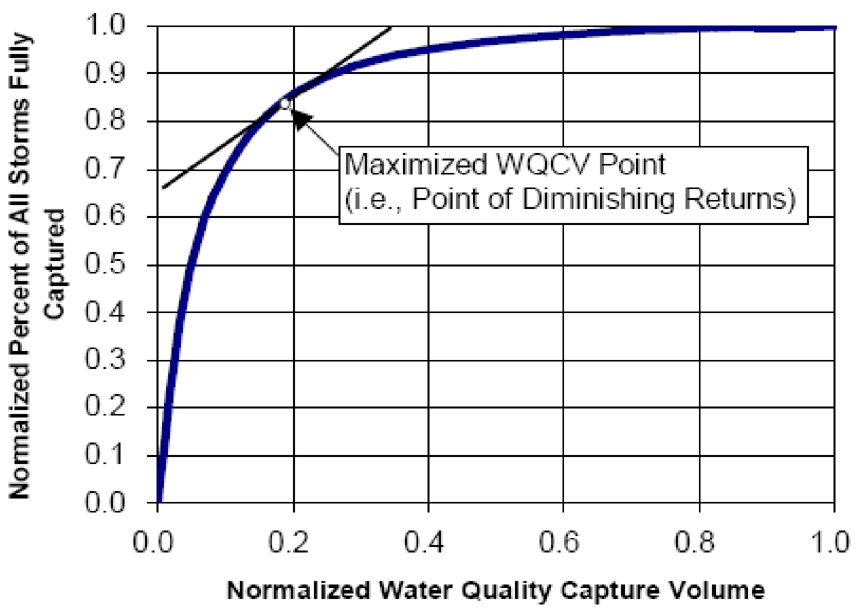


Figure (2): Maximize detention volume by runoff capture ratio (after Guo and Urbonas, 1996).

1.7 The Pitt Method

This method can be regarded as a further extension and rational progression to the rainfall capture method. It is based on plotting the cumulative probability density functions (CDFs) of rainfall, runoff and pollutant loads. The joint inspection of these CDFs will enable determining the rainfall depth responsible for most of runoff events and pollutant loads. It appears this method was originally developed by Pitt (1999) using the data of the Milwaukee catchment in south-eastern Wisconsin, USA.

Figures (3) and (4) show the CDFs of rainfall, runoff and a range of pollutants for Milwaukee catchment for a medium residential area.

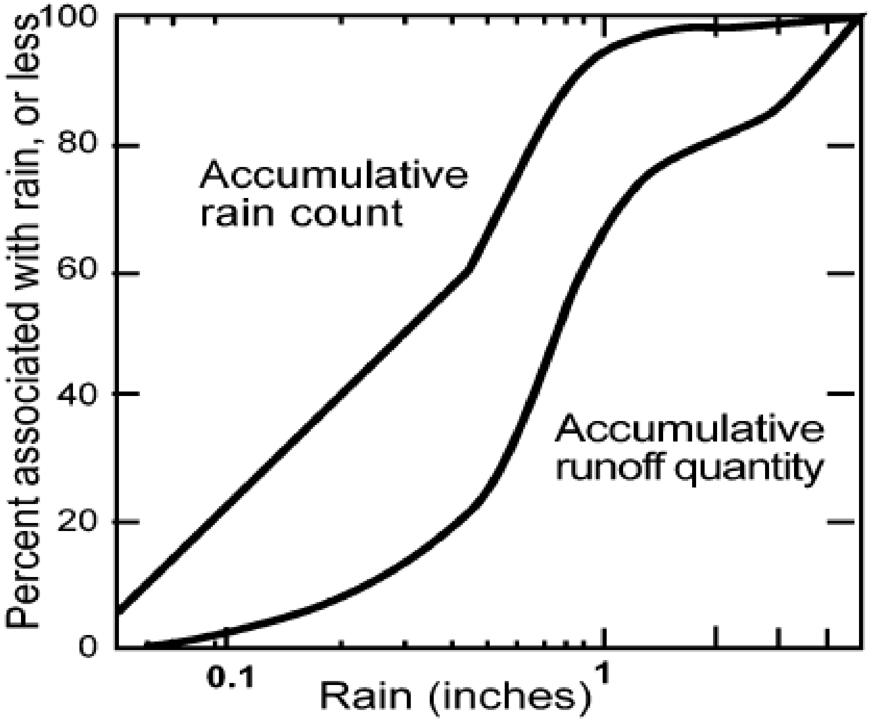


Figure (3): CDFs of rainfall and runoff (after Pitt, 1999).

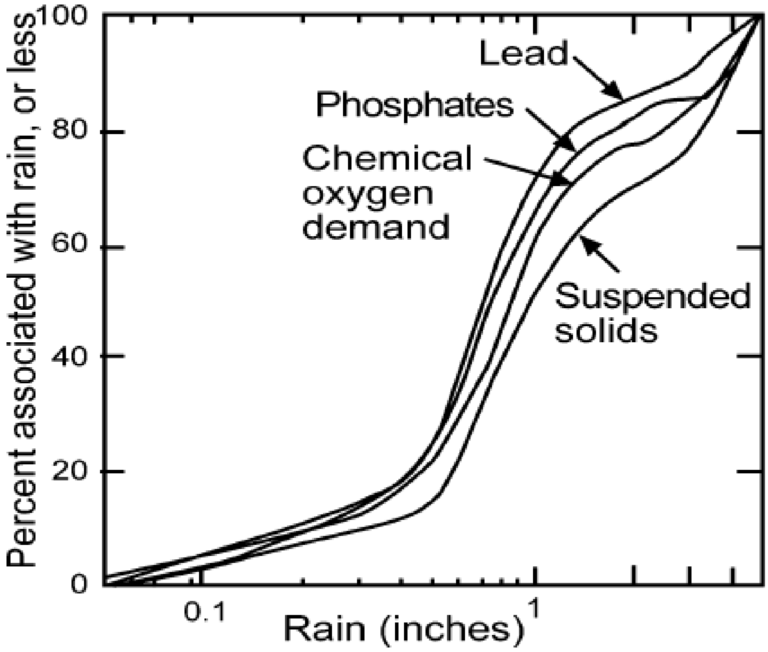


Figure (4): CDFs of different pollutant (after Pitt, 1999).

Based on the analysis of these figures and other data from this catchment Pitt (1999) noted that rainfall depths less than 0.5 inch (12.7 mm) which are very common occurring once or twice a week "*account for most of the events, but little of the runoff volume, and are therefore easiest to control. They produce much less pollutant mass discharges and probably have less receiving water effects than other rains. However, the runoff pollutant concentrations likely exceed regulatory standards for several categories of critical pollutants, especially bacteria and some total recoverable metals*". However, rainfall depths in the range 0.5 inch (12.7mm) to 1.5 inch (38.1 mm) which occur on average every two weeks account for the majority of the annual runoff volume and subject the receiving environmental to frequent high pollutant loads and moderate to high flows. Furthermore, rainfall depths in the range 0.5 inch (12.7 mm) to 1.5 inch (38.1 mm), which produce high flow events occurring once or twice a year are the most damaging to habitat. However, they only account for 10% of annual pollutant load. The less frequent rainfall depths in excess of three inches, which are typically the design of the drainage system of the Milwaukee, produce around 15 % of the annual runoff and pollutant discharges. These rainfall depths are associated with flooding and possible loss of life and damage to properties. However, Pitt (1999) noted that storms associated with this rainfall depth range, while very destructive, are sufficiently rare that the resulting environmental problems do not justify the massive stormwater quality controls that would be necessary for their reduction. Flood detention storage is maximised to prevent property damage and possible loss of life. That is, for such a purpose, large storage facilities are often required, whereas water quality facilities usually require less storage. It is clear that both objectives have to be incorporated where possible".

The above noted analysis of the Milwaukee catchment data highlights the importance of joint analysis of CDFs of rainfall, runoff and pollutant in setting different criteria to achieve a range of stormwater management goals. The analysis also demonstrates that the design basis for stormwater quality management devices should be based on the small and more frequent storms where the understanding of the hydrology of these storms is paramount.

1.8 Discussion and Conclusions

In this part of the report six WQV estimation methods have been discussed. These methods are:

- TP10 WQV method,
- Half-inch (first-flush) rule,
- One-inch rule,
- Rainfall capture rule,
- Maximized detention volume and
- Pitt method.

In TP10, at a particular site, the WQV is calculated as 1/3 of the design rainfall depth defined as the at-site 2-year ARI 24-hour rainfall. This 1/3 rule was derived using the data from a single site (Botanic Gardens) which showed that there is a point of diminishing return where increasing the system's capture volume does not yield a corresponding increase in potential for treating annual pollutant. In the Botanic Garden, the point of diminishing return corresponds to a rainfall depth of 25 mm which is loosely approximated by a storm that is 1/3 of the 2-yr 24-hour rainfall depth. From a statistical point of view, the generalization of the 1/3 rule on the basis of a single site may not be adequate. In principle, the method used to derive the WQV in TP10 is very similar to the maximized detention volume proposed by Urbonas et al. (1989). However, estimating WQV in this way may be device dependent and it is possible that different devices may have different points of diminishing return.

A common feature of the six WQV estimation method discussed in this chapter is that they are based on capturing runoff volume from the smaller more frequent storm events. This highlights that fundamental understanding of small storms hydrology is the key for effective stormwater quality management. The six methods discussed in this report may differ in the way in which the WQV is estimated. Only two methods, namely, TP10 WQV method and maximized detention volume; explicitly take into consideration optimization of the device size when determining the appropriate WQV value. However, the rainfall capture rule and the Pitt method implicitly take into consideration optimization of the device size by determining the rainfall depth that captures the majority of runoff volume.

In terms of data requirements, all the methods discussed in this part of the report require rainfall data. The Pitt method and the first-flush rule require additional information about pollutants. However, the TP10 and maximized detention volume require a device to be selected in order to determine WQV.

In terms of scientific soundness, the one-inch rule stands at the low end of the scale as capturing the runoff volume from one inch rainfall depth is very arbitrary for a particular location. As there is no actual methodology in this rule, it cannot be transferred to other regions having climate and hydrologic conditions significantly different from the location in which it was originally applied. The first-flush rule is very controversial as there is a considerable debate on whether or not the first-flush effects exist and how the first flush is defined. Thus, prior to the use of this rule in the estimation of WQV, there is a need to confirm the existence of the first flush so that an appropriate value of WQV can be determined. This requires pollutograph data, however, the availability of such data in Auckland may be very limited. The first flush rule is useful in device design. It provides design elements for capture of initially mobilised material such as suspended sediment. If there is strong base flow then the rule may need some adjustment, but general first flush indicators such as total suspended solids should feature in design. The rainfall capture rule, maximized detention volume and Pitt methods are scientifically more elegant. They offer transparent and defensible WQV estimation methodologies which can be universally applied to different climatic and hydrologic regions. The Pitt method also requires pollutograph data which are of limited availability in Auckland. Likewise, the maximized detention volume required a device to be used in the analysis. Furthermore, the uniqueness of the WQV estimate is questionable as it may depend on the chosen device. For the aforementioned reasons, the author's recommendation to ARC is to use the rainfall capture rule for WQV estimation. This rule is less complex and data demanding than that the maximized detention volume and the Pitt method and its results can be further refined by these two methods.

2 Application of the Rainfall Capture Rule

2.1 Introduction

This section of the report deals with applying the rainfall capture to the data of 31 automatic rainfall stations in the Auckland region. The main purpose of applying this rule to 31 stations is to investigate whether or not there is a significant spatial variability in the WQV estimates in the Auckland region. This part of the report is organized in the following manner. Firstly, the rationale behind the selection of the 31 automatic rainfall stations in the Auckland region is discussed. Secondly, the steps involved in developing the rainfall spectrum required by the rainfall capture rule to determine the appropriate WQV value is given. Thirdly, the results of applying the rainfall capture to the Auckland data are presented. Finally, summary and conclusions are given.

2.2 Selection of Automatic Rainfall Stations

The 31 automatic rainfall stations used in WQV estimation using the rainfall capture are the same as those used by Shamseldin (2008) when updating the rainfall statistic data for the Auckland region. The rainfall data was obtained from a variety of sources including Watercare, National Institute for Water and Atmospheric science (NIWA) and ARC providing data held by Auckland local councils. The data of these stations were previous extensively checked by Shamseldin (2008) for trends and errors. The criteria used for selecting the automatic stations is very similar to that of Shamseldin (2008) and can be summarized as follows:

• Record length should be greater than 13 years which should long enough to produce reliable results using the rainfall capture rule

• Location to ensure geographical spread.

• Near complete record in the case of the rainfall stations available in the NIWA national database.

Figure (5) shows the station locations while Table (1) provides summary information about the station including the record length, latitude and longitude.

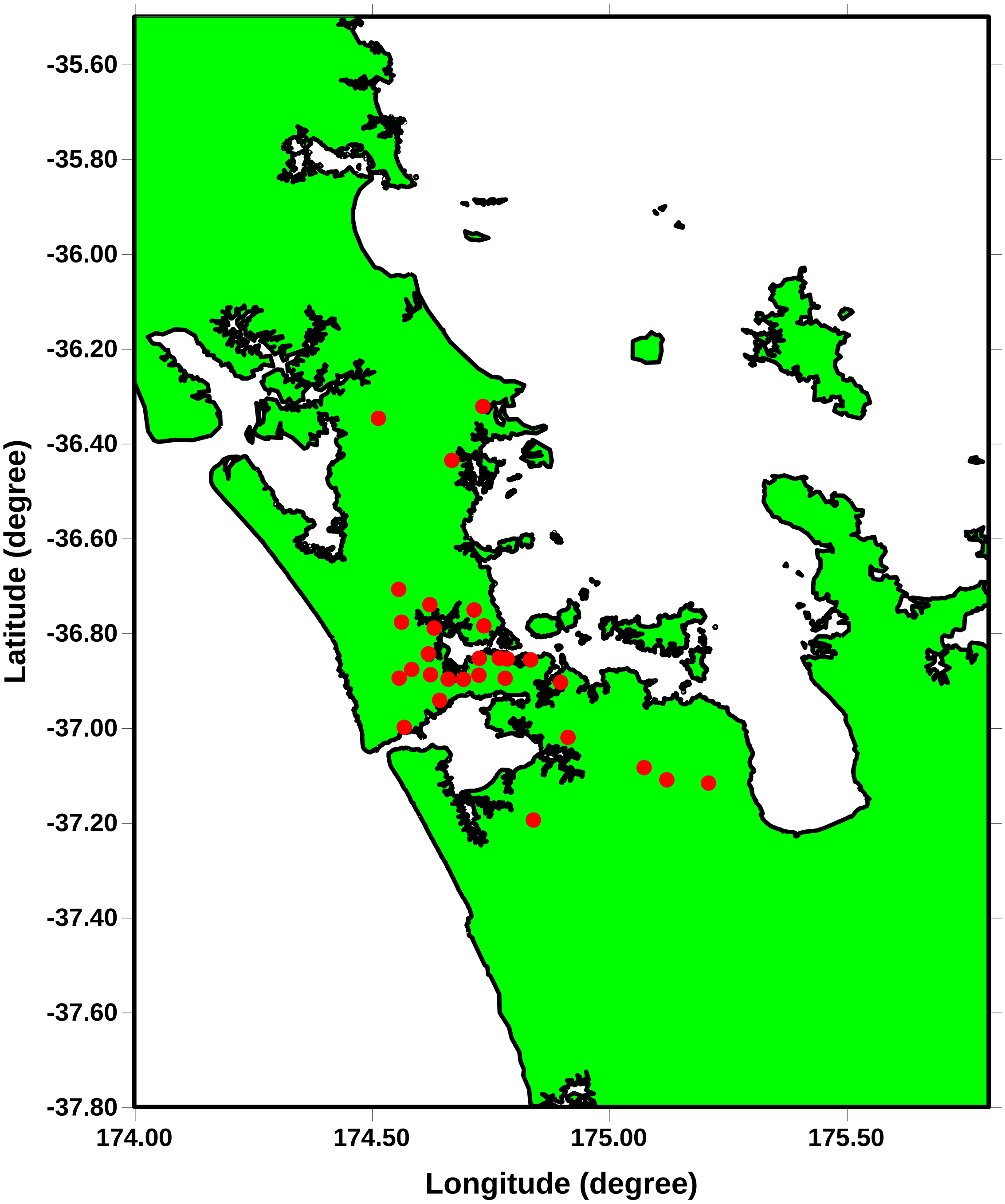


Figure (5): Location of the 31 selected automatic rainfall stations in the Auckland Region (after Shamseldin, 2008).

Table (1): Summary Information on the 31 selected automatic rainfall stations in the Auckland Region.

Station	Latitude (Degrees)	Longitude (Degrees)	Period
643510	-36.3457	174.5125	1978-2008
643713	-36.3207	174.7325	1992-2008
647510	-36.7065	174.5552	1978-2008
647601	-36.7883	174.6299	1945-2008
647614	-36.7388	174.6209	1978-2008
648510	-36.8755	174.5826	1988-2008
648612	-36.8866	174.6222	1990-2008
648613	-36.8425	174.6178	1990-2008
648614	-36.8960	174.6594	1991-2008
648719	-36.8519	174.7671	1962-2003
649625	-36.9404	174.6414	1990-2008
649820	-36.9028	174.8964	1988-2008
740815	-37.0188	174.9119	1983-2008
741813	-37.1932	174.8390	1992-2008
750010	-37.0826	175.0723	1979-2008
nSCC07	-36.7835	174.7347	1973-2003
Wairoa	-37.1083	175.1201	1992-2007
Lower Huia	-36.9975	174.5668	1992-2007
Mangatangi	-37.1152	175.2080	1993-2007
644616	-36.4344	174.6673	1993-2007
647513	-36.7758	174.5613	1993-2007
647727	-36.7500	174.7141	1994-2007
648513	-36.8939	174.5561	1993-2007
648615	-36.8963	174.6920	1992-2007
648717	-36.8877	174.7243	1991-2007
648718	-36.8940	174.7794	1992-2007
649713	-36.8519	174.7250	1991-2007
649714	-36.8533	174.7838	1992-2007
649818	-36.8554	174.8333	1992-2007
742736	-36.4344	174.6673	1993-2007
742914	-36.7500	174.7141	1993-2007

2.3 Development of the Rainfall Frequency Spectrum

In this study, the development of the analysis of the rainfall frequency spectrum is conducted based on the total storm rainfall depth as well as the daily rainfall depth. The development of the rainfall frequency spectrum curve is similar to that used in developing flow durations. As mentioned in Section 2 of this report, the development of the rainfall frequency spectrum curve is a process of establishing a non-parametric relationship between non-exceedance probability and the storm event/daily rainfall depth. This curve gives the percentage of time that a given rainfall depth is equalled or exceeded. The curve provides useful information about the rainfall depth variability. In general, steep curves indicate high variability. The curve typically shows a sharp curvature (knee/inflection) point. Beyond this point the curve can be regarded as containing the large storms. The steps involved in the development of the rainfall frequency spectrum can be summarized as follows:

- Step 1: Identify rainfall storms or days with a total depth greater than 5 mm. It is assumed that a total rainfall depth less than or equal to 5 mm does not produce significant runoff.

- Step 2: Rank the rainfall depths in ascending order.

- Step 3: Determine the non-exceedance probability (P %) according the following equation which is often used in the case of flow duration curves (Watson and Burnett, 1995)

$$P= (100\, j)/(N+1) \qquad (2)$$

where j is the rank and N is the total number of runoff-producing storms.

2.3.1 Rainfall Storm Event Identification

Defining what constitutes a rainfall event is highly uncertain due to the variability and randomness of the rainfall phenomena (Veneziano and Villani, 1999), and is also user specific. In the Auckland region, it is very difficult to define clear cut rainfall storm events especially when the separation time between subsequent rainfall bursts is short. The identification of rainfall events from a continuous rainfall record is based on specifying the event-separation time. If the inter-event time between two subsequent events is less than the separation time then the two events are combined into a single event. There is no universal consensus on the value of separation time. Driscoll et al. (1989) found that a 6-hour event-separation time produced the most consistent results. In the context of small storm hydrology, Pitt (1999) found that the definition of the event-separation time makes little difference to the conclusions. The underlying principle is basically to examine the sensitivity of the results to the value of the event-separation time. This principle is adopted in this study through examining the sensitivity of the WQSD estimates to seven separation times, namely, 0.5, 1, 2, 3, 6, 12 and 24 hour.

2.4 Analysis of Rainfall Frequency Spectrum in the Auckland Region

2.4.1 Rainfall Spectrum Analysis based on Storm Events

The rainfall spectrum analysis procedure outlined in Section 3.3 is applied to the 31 stations using different event separation times. Figure (6) shows the rainfall frequency curves for six selected stations for the two hour inter-event time. These stations are; Kumeu (647513), Mahurangi (644614), Albert Park (648719), Whenuapai (647601), Pakarangua (649820) and Lower Huia. Examination of the figure shows that the majority of rainfall events are small with rainfall depths less than 30 mm. Further inspection of the figure reveals that the 90th percentile rainfall depth can be regarded as a good approximation of the knee/inflection point of the frequency curve. Table (A1) in Appendix-A provides further information about rainfall event depth estimates and summary statistics for different event-separation times and percentiles.

Figure (7) shows the variation of the spatially averaged event rainfall depths for the 80th, 85th, 90th and 95th percentiles with the event separation time. The figure shows that the relationship between the event rainfall depth and the separation time has a sharp curvature around the one hour inter-event time. This may signify changes in the rainfall generating mechanisms.

Further examination of Figure (7) shows that for a given percentile, the event rainfall depth increases significantly with the increase in the event separation time. This is not surprising given the Auckland climate which can have extended wet periods with consecutive rainfall events having short separation times. Hence, as a result of increasing the event separation time, many of these closely occurring events are amalgamated into a single event with a higher rainfall depth. For a given percentile, the figure also indicates that the WQSD values vary spatially across the Auckland region.

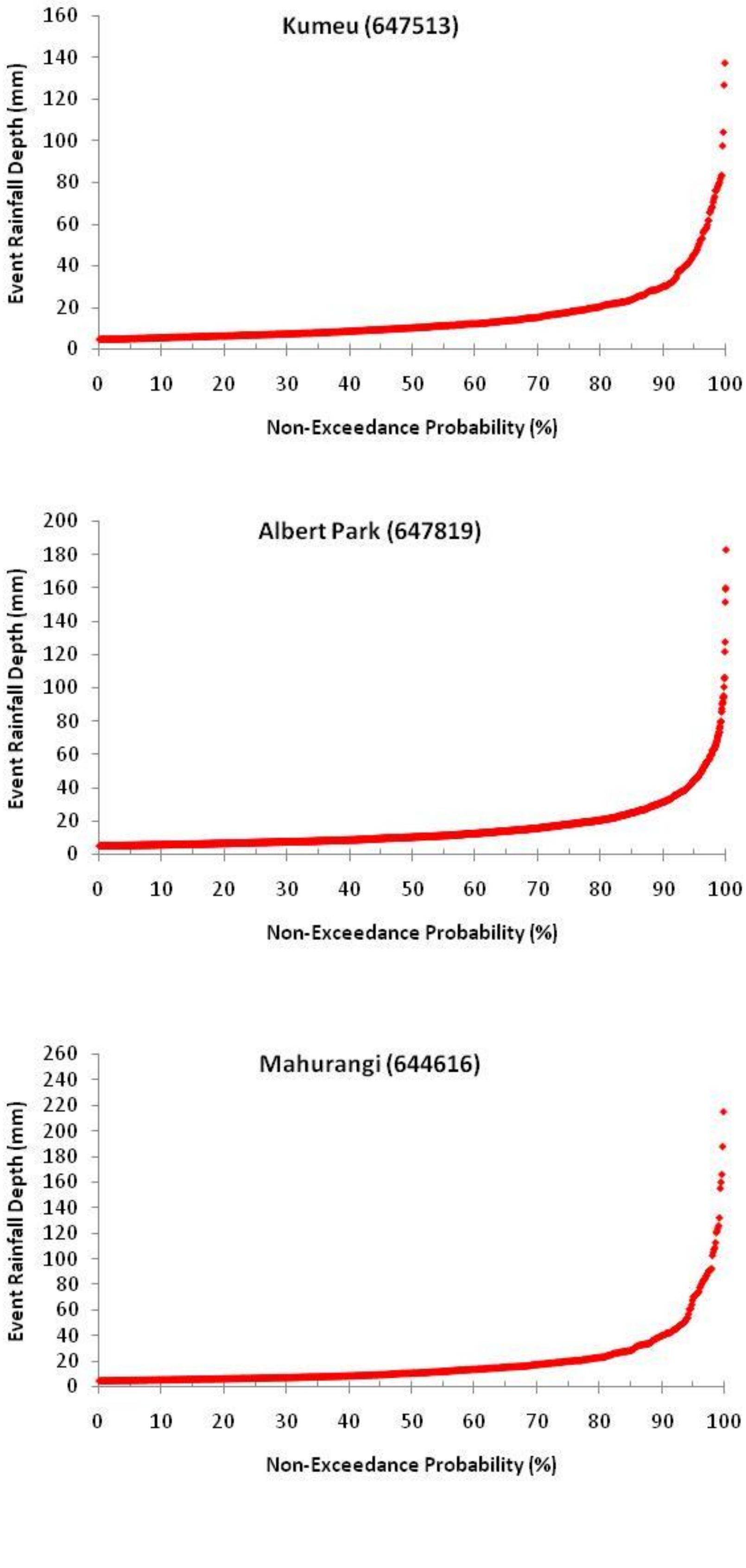
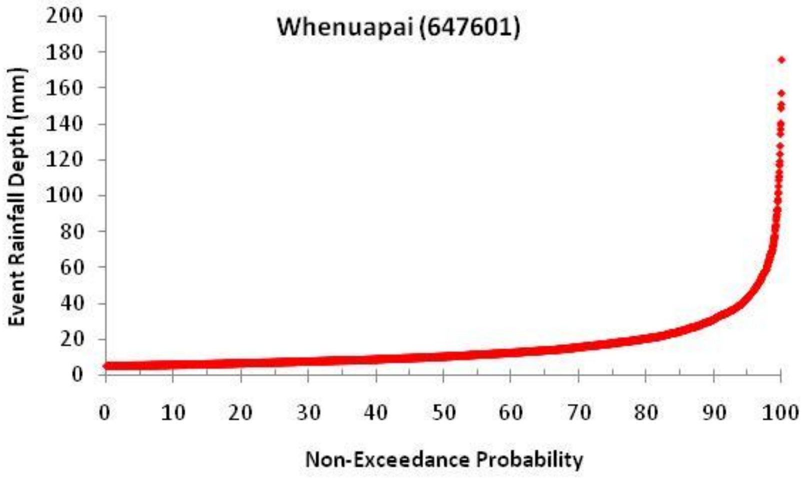
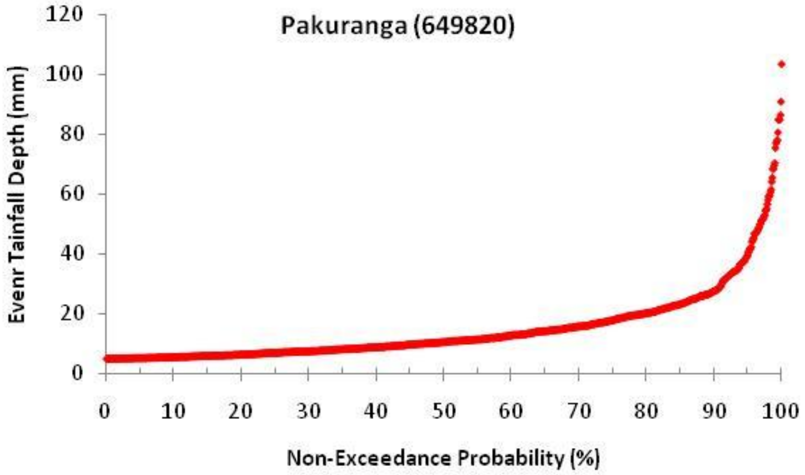


Figure (6): Rainfall frequency curves for the 2 hour event separation time in the Auckland Region.





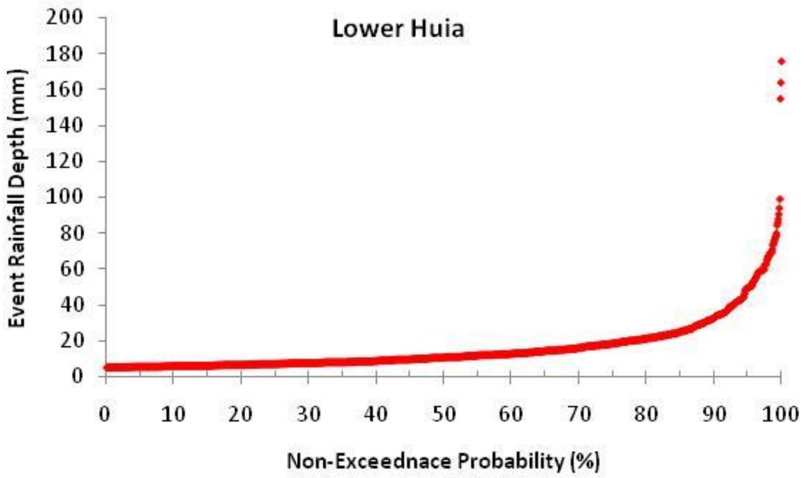


Figure (6): cont

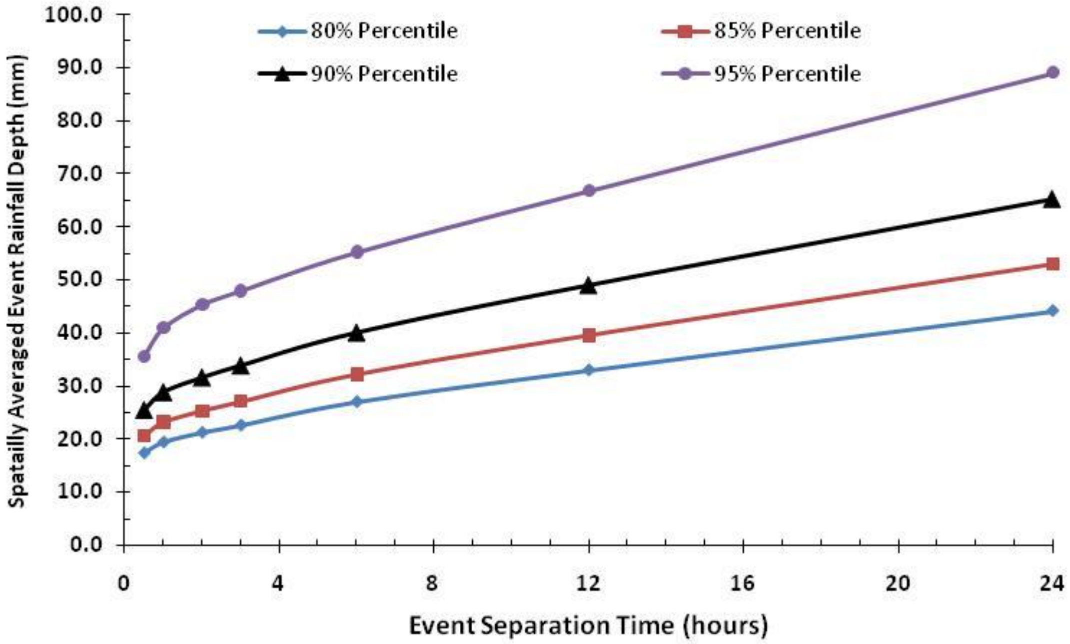


Figure (7): Spatially averaged event rainfall depths for different percentiles and event separation times.

In the rainfall frequency curves such as those shown in Figure (6), the rainfall event depths are arranged without regard to the chronological sequence of occurrence of rainfall events which is vital in determining the device real-time performance as they can significantly influence runoff volumes and pollutant load. Thus, it is recommended that WQV design values obtained on the basis of such curves be refined by continuous simulation which takes into consideration the chorological sequence of rain events and enables the evolution of the long term operational device runoff performance in volume capture and pollutant removal.

Based on the work of Shamseldin (2008), the Water Quality Storm Depth (WQSD) (1/3 of the 2 year 24 hour annual maximum rainfall depth) for these stations can vary between 45.9 mm and 24.44 mm (see Table A.2 for further details). Figure (8) shows the relationship between the 2 year 24 hour annual maximum rainfall depth and the 90th percentile rainfall depth for the 2 hour separation time. Examination of Figure (8) shows that the 90th percentile rainfall depth of the 2 hour event separation time is approximately equal to 1/3 of the 2-year 24 hour annual maximum rainfall depth.

Figure (9) provides information about the spatial variability of the event rainfall depth expressed in term of the spatial range (i.e. difference between the maximum and the minimum rainfall depth for the 31 stations) for different percentiles and event separation times. Examination of the figure shows that in general the rainfall depth range increases with the increase in the event separation time. The figure also indicates that the rainfall depth range exhibits spatial variation and the degree of its spatial variation is dependent of both the percentile and the event.

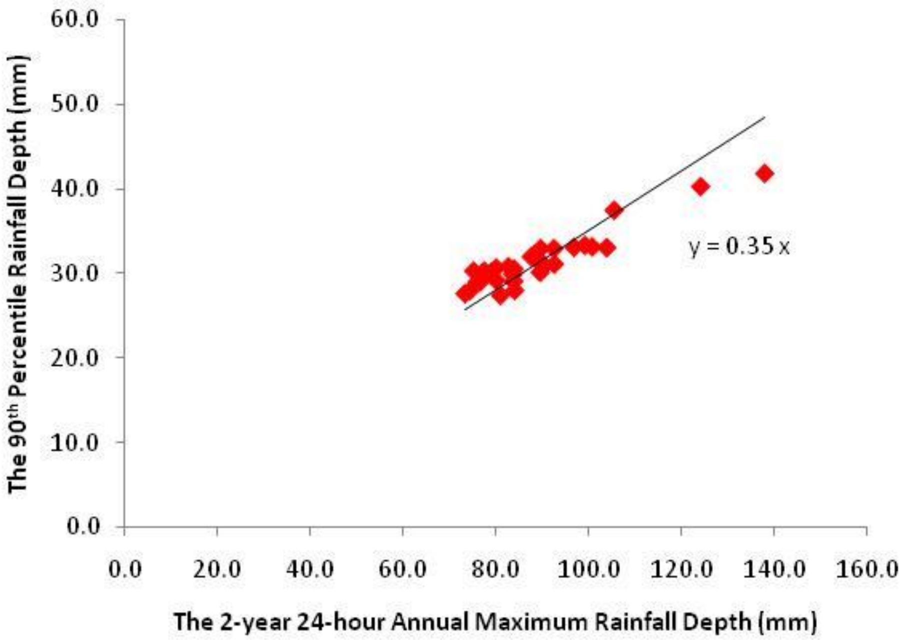


Figure (8): Relationship between the 2-year 24-hour annual maximum rainfall depth and the 90th percentile rainfall depth of the 2-hour event separation time in the Auckland Region.

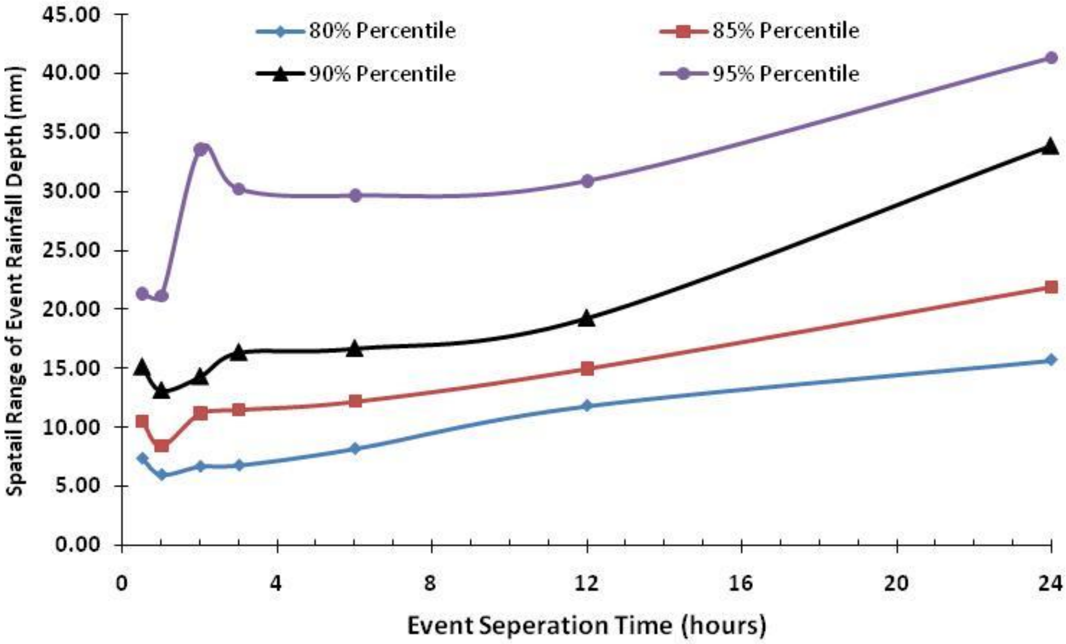


Figure (9): Spatial range of the event rainfall depth in the Auckland Region.

2.4.2 Rainfall Spectrum Analysis based on Daily Rainfall Values

A method to avoid the uncertainties with the definition of the inter-event time is the usage of daily reading rainfall records. For this analysis the same 31 stations as described in Section 3.2 were used. The rainfall depth was accumulated over the period from 9am to 9am the next morning. Figure (10) shows the rainfall frequency curves for six selected stations obtained using the daily rainfall values. These stations are the same used in Section 3.4.1 dealing with the rainfall spectrum analysis using storm events. Examination of Figure (10) shows that the 90th percentile rainfall depth can be regarded as a good approximation of the knee/inflection point of the frequency curve. Hence, the 90th percentile daily rainfall depth should be used as the Water Quality Storm depth (WQSD). Table (A3) in Appendix-A provides further information about rainfall depth estimates for the 80th, 85th, 90th and 95th percentiles.

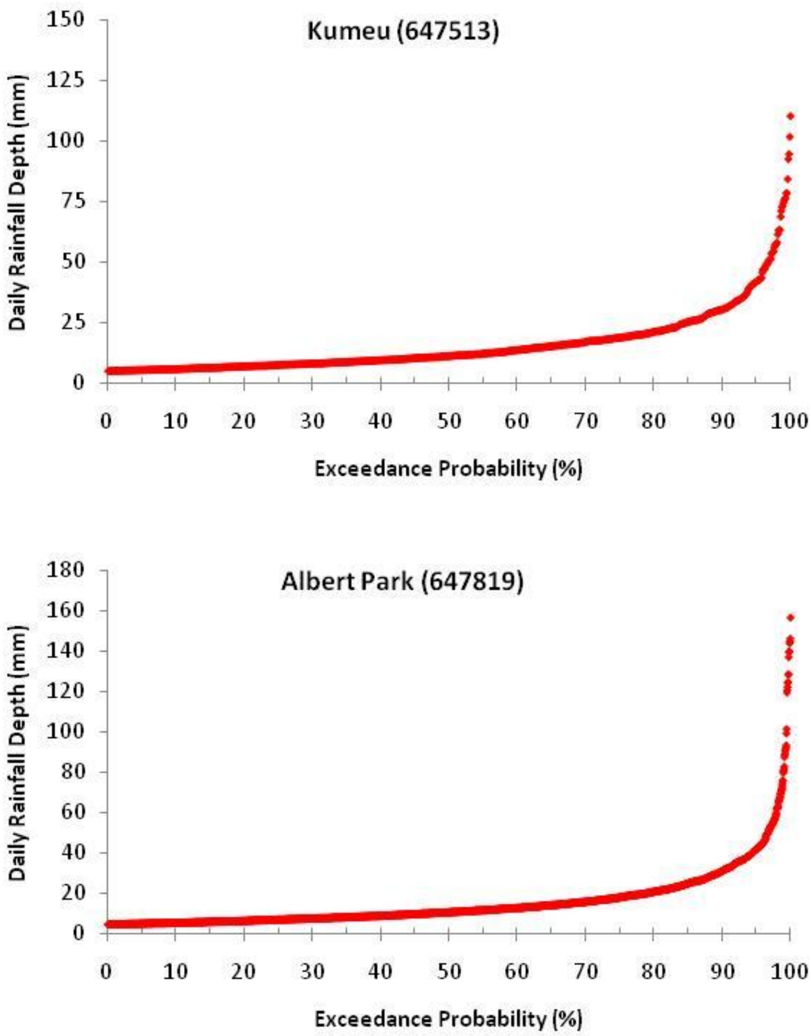
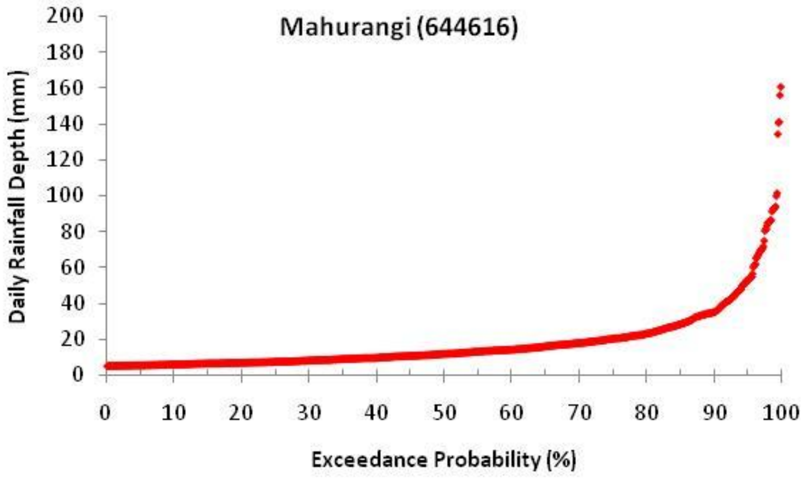
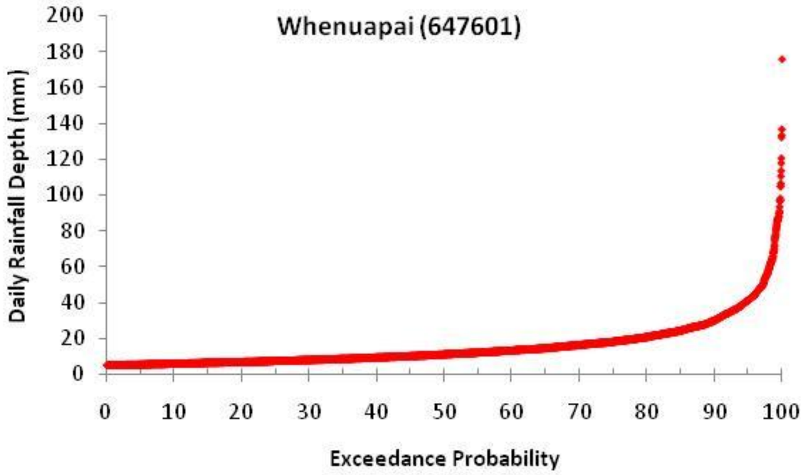


Figure (10): Frequency curves for the daily rainfall depth in the Auckland Region





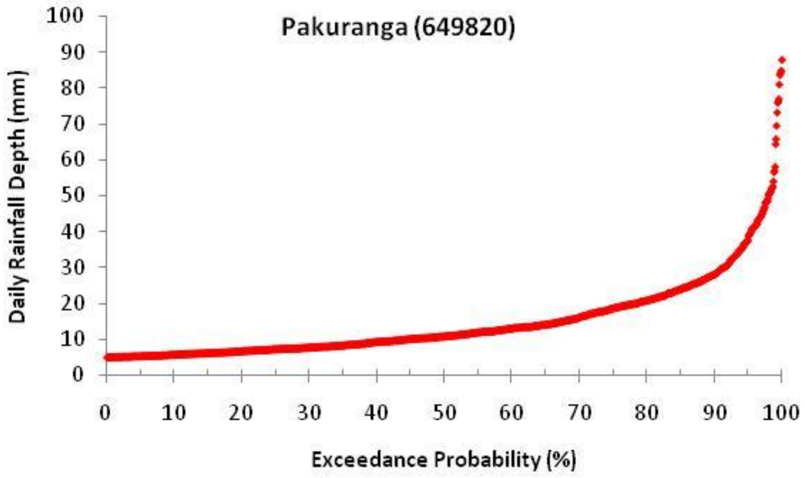


Figure (10):Cont.

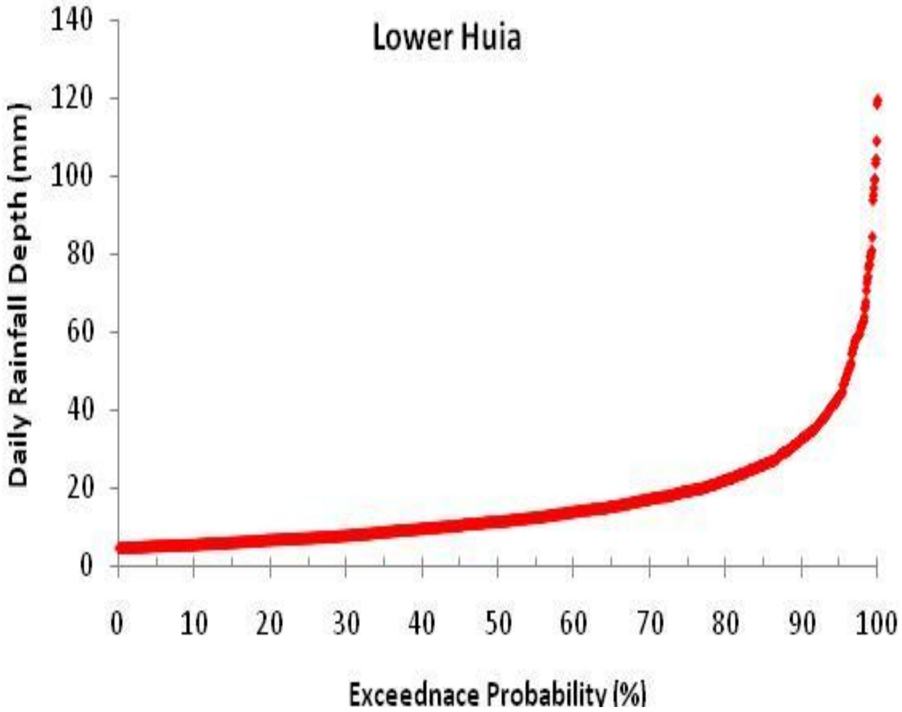


Figure (10):Cont.

Figure (11) shows the relationship between the 2-year 24-hour annual maximum rainfall depth and the 90th percentile daily rainfall depth. The figure shows that the 90th percentile daily rainfall depth is approximately equal to 1/3 of the 2-year 24-hour annual maximum rainfall depth. The result of this figure confirms the adequacy of the WQV estimation procedure in TP10 in which WQSD is obtained as 1/3 of the 2 year 24 hour annual maximum rainfall depth.

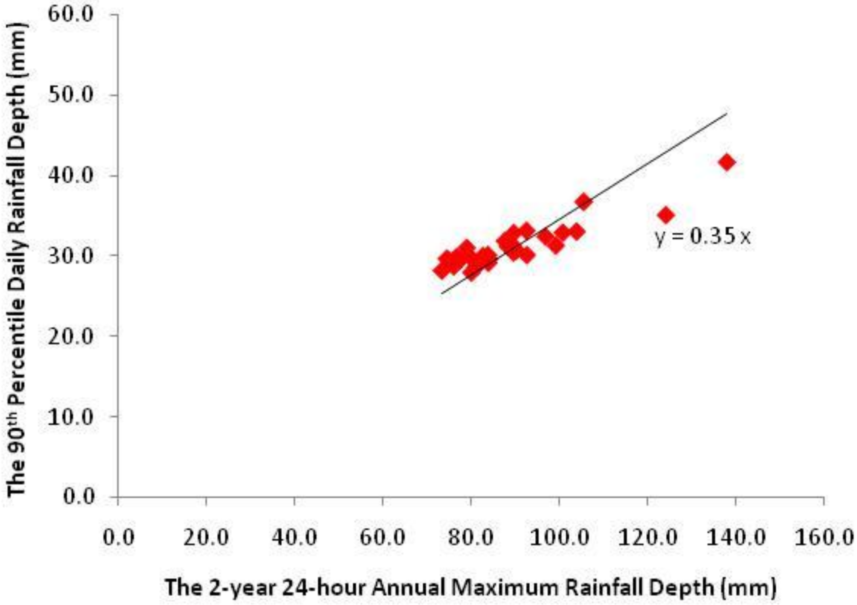


Figure (11): Relationship between the 2-year 24-hour annual maximum rainfall depth and the 90th percentile daily rainfall depth in the Auckland Region

2.5 Conclusions

In this part of the report, the rainfall capture for estimating the water quality storm water depth (WQSD) is applied to the data from 31 automatic rainfall stations in the Auckland region. This depth is used in calculating what is known as the Water Quality Volume (WQV). The sensitivity of the WQSD estimates to the event separation time is examined using seven separation times, namely, 0.5, 1, 2, 3, 6, 12 and 24 hour. The spatial variability of WQSD estimates is also investigated in this part of the report. WQSD estimates are also obtained using daily rainfall data. The main conclusions of the study conducted in this part of the report are:

- The WQSD estimates exhibits spatial variation dependent on both the percentile and the event separation time.

- The WQSD estimates are sensitive to the event separation time and the percentile of the rainfall depth. The sensitivity of WQSD to the event separation time has also been reported in previous studies (e.g. Balistrocchi et al., 2005). The WQSD estimates based on the 90th percentile rainfall depth of the two hour event separation time provides a close approximation to the TP10 1/3 rule in which WQSD is approximately equal to 1/3 of the 2-year 24 hour annual maximum rainfall depth.

- The WQSD estimates based on the 90th percentile daily rainfall depth provides a close approximation to the TP10 1/3 rule of the 2-year 24 hour annual maximum rainfall depth.

- The TP10 1/3 rule for estimating the water quality storm water depth is very adequate and therefore provides a sound base for the WQSD definition.

Acknowledgements

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Appendix A

Table (A1): Rainfall Event Depth Estimates and summary statistics for different event separation times and percentiles.

Station	Event separation time															
	0.5 hour				1 hour				2 hour				3 hour			
	Percentile															
	80 %	**85 %**	**90 %**	**95%**	**80 %**	**85 %**	**90 %**	**95%**	**80 %**	**85 %**	**90 %**	**95%**	**80 %**	**85 %**	**90 %**	**95%**
643510	17.6	21.0	26.1	36.0	20.6	24.5	31.0	42.5	22.5	26.9	33.2	47.8	24.0	28.5	35.5	52.5
643713	20.9	26.3	34.7	46.4	23.0	29.1	38.5	55.9	25.9	33.5	41.8	63.0	27.0	35.4	45.0	65.8
647510	17.6	20.4	24.5	35.9	19.4	23.1	29.1	41.3	20.5	24.4	32.1	44.4	21.8	26.3	33.3	45.2
647601	13.6	15.8	19.6	27.0	17.7	21.0	26.2	35.6	20.3	24.4	31.2	43.4	21.6	26.1	33.0	45.9
647614	17.5	20.6	26.0	36.9	20.7	24.9	30.6	44.0	21.4	27.0	33.4	48.0	22.0	27.6	36.0	51.5
648510	17.5	20.3	24.4	35.2	19.1	22.8	29.1	40.5	21.0	23.8	30.9	42.5	22.8	26.5	33.6	46.0
648612	16.6	20.2	24.9	33.8	19.2	21.8	28.0	39.0	20.1	23.0	29.2	42.5	21.5	24.9	31.3	44.0
648613	17.0	19.9	24.0	32.2	19.2	22.5	26.4	38.5	20.0	23.8	27.5	40.0	20.9	24.5	29.7	41.3
648614	16.4	20.3	24.5	33.2	18.5	22.0	26.9	36.9	19.2	23.0	28.1	41.0	21.2	24.4	30.9	42.7
648719	15.7	18.0	22.4	31.7	19.0	22.3	28.0	38.9	20.6	25.0	31.6	45.1	21.8	26.3	33.3	47.0
649625	18.1	21.0	25.9	37.0	19.4	22.8	27.6	39.8	20.8	24.9	30.9	43.2	22.6	27.5	33.7	45.3
649820	17.1	19.7	23.4	31.5	19.2	21.7	26.0	36.5	20.3	23.3	27.7	40.5	21.1	24.5	30.5	44.4
740815	16.5	19.0	23.5	32.5	18.9	22.4	28.4	38.0	20.7	24.8	30.7	42.6	21.8	25.9	32.5	45.1
741813	15.9	18.6	22.4	29.6	18.4	21.7	26.0	34.7	20.7	24.2	29.1	37.1	21.8	25.9	30.2	40.5
750010	17.2	20.0	24.8	32.0	20.0	23.4	28.1	38.2	22.3	26.0	32.3	44.0	23.7	28.1	34.1	47.0
nSCC07	17.2	20.3	25.1	36.2	19.8	23.4	28.3	42.5	21.5	25.4	32.0	45.2	22.9	27.1	34.5	49.2
Wairoa	17.5	20.5	26.0	36.0	19.5	23.0	27.6	37.5	22.0	26.0	33.0	45.2	24.3	28.4	35.5	50.0
Lower Huia	17.5	21.5	29.0	39.6	19.0	23.5	31.3	45.5	21.2	25.0	33.1	49.8	23.5	28.0	35.6	50.5
Mangatan gi	20.1	24.0	32.0	43.5	21.5	27.0	34.5	48.0	25.0	30.0	37.5	55.5	27.5	31.2	43.5	62.0
644616	19.2	22.4	31.6	48.5	21.0	26.5	36.8	53.6	23.2	29.0	40.3	70.7	24.0	31.7	42.3	70.7
647513	16.9	20.2	24.9	34.1	19.6	22.5	28.6	43.7	20.7	24.0	30.2	46.2	22.6	26.4	31.7	47.8
647727	18.6	21.8	26.0	37.5	20.0	22.9	27.6	40.2	22.0	25.0	33.0	46.5	22.6	25.9	34.2	50.7
648513	19.0	22.6	29.0	41.9	20.2	24.2	30.7	46.3	22.9	27.0	33.2	46.4	24.2	29.3	37.3	50.6
648615	17.5	20.4	25.0	34.8	19.2	21.9	27.6	38.3	19.7	22.3	29.3	40.1	21.1	24.2	31.7	40.8
648717	17.8	20.9	24.5	36.0	18.7	22.5	27.7	40.0	20.7	24.0	30.4	41.0	22.1	26.0	32.5	43.5
648718	18.6	21.5	26.0	36.2	19.2	22.5	29.4	40.3	20.8	24.4	30.0	45.5	22.1	26.2	32.5	48.5
649713	18.8	21.5	26.5	37.8	19.9	22.5	28.9	40.3	21.1	24.5	30.6	42.5	22.5	25.9	32.7	47.1
649714	16.4	19.1	23.5	32.8	18.6	21.3	26.2	40.8	19.9	23.0	29.1	42.7	21.0	25.1	30.8	42.9
649818	17.9	21.0	25.6	36.5	19.4	22.4	27.6	41.3	21.7	25.7	30.4	46.8	22.8	27.0	31.7	46.5
742736	15.4	18.8	22.1	31.7	17.1	20.7	25.4	35.8	19.5	23.2	28.1	37.6	20.7	23.9	28.6	40.6
742914	15.9	18.9	23.0	32.0	18.5	22.3	26.4	36.5	21.0	25.0	30.2	40.6	22.5	26.3	32.0	42.6

Mean	17.4	20.5	25.5	35.7	19.5	23.1	28.8	41.0	21.3	25.2	31.6	45.4	22.6	26.9	33.9	48.0
StD	1.4	1.9	3.1	4.6	1.1	1.8	3.0	4.9	1.5	2.3	3.3	6.9	1.6	2.4	3.8	7.0
Cv	8.2 %	9.0 %	12.1 %	12.9 %	5.8 %	7.7 %	10.5 %	11.8 %	7.0 %	9.1 %	10.4 %	15.1 %	7.0 %	9.0 %	11.2 %	14.6 %
Max	20.9	26.3	34.7	48.5	23.0	29.1	38.5	55.9	25.9	33.5	41.8	70.7	27.5	35.4	45.0	70.7
Min	13.6	15.8	19.6	27.0	17.1	20.7	25.4	34.7	19.2	22.3	27.5	37.1	20.7	23.9	28.6	40.5
Range	7.4	10.6	15.1	21.4	6.0	8.5	13.1	21.2	6.7	11.2	14.3	33.6	6.8	11.5	16.4	30.2

Table (A1): cont.

Station	Inter-event time											
	6 hour				12 hour				24 hour			
	Percentile											
	80 %	**85 %**	**90 %**	**95%**	**80 %**	**85 %**	**90 %**	**95%**	**80 %**	**85 %**	**90 %**	**95%**
643510	28.1	33.1	40.8	57.2	32.3	38.0	48.5	67.0	41.1	51.2	61.5	28.1
643713	31.4	39.7	52.5	75.1	39.8	46.8	61.5	84.2	52.4	63.1	77.5	31.4
647510	26.5	31.3	38.7	53.5	32.5	38.9	47.3	63.0	41.6	48.8	61.7	26.5
647601	26.0	31.0	38.3	53.4	32.2	37.8	47.1	63.8	42.0	50.6	63.3	26.0
647614	27.9	33.2	42.9	59.0	34.3	41.3	52.5	70.0	44.0	54.4	65.9	27.9
648510	28.7	34.7	40.8	54.3	36.6	41.9	51.5	69.2	47.0	57.3	68.9	28.7
648612	26.7	31.9	39.4	56.5	31.3	37.5	45.4	64.0	40.5	49.3	59.9	26.7
648613	26.1	31.2	38.6	50.5	31.2	37.1	43.7	62.0	40.5	46.4	58.7	26.1
648614	26.5	31.9	39.1	51.5	31.8	37.5	46.3	62.9	39.9	48.4	56.5	26.5
648719	25.5	30.2	37.9	53.0	30.3	36.4	44.9	62.7	42.2	49.8	62.2	25.5
649625	30.6	35.2	43.9	59.6	35.2	41.5	53.2	69.3	47.6	56.5	66.4	30.6
649820	24.8	28.9	35.9	50.0	28.5	34.0	42.6	56.2	37.7	43.1	52.0	24.8
740815	26.0	30.2	37.5	51.5	29.6	35.8	43.7	60.5	38.0	44.7	58.5	26.0
741813	25.9	29.0	36.2	45.8	30.2	35.4	44.1	60.4	40.8	47.6	58.8	25.9
750010	28.2	32.5	39.6	53.4	33.4	39.1	47.5	63.9	43.5	51.5	62.7	28.2
nSCC07	26.3	31.2	38.5	54.8	31.8	38.8	47.6	63.9	39.3	46.0	55.7	26.3
Wairoa	28.6	33.5	40.5	56.0	36.0	41.8	50.5	72.5	45.5	53.6	68.5	28.6
Lower Huia	27.0	33.5	42.2	58.6	36.0	42.5	54.0	73.8	51.5	62.0	75.6	27.0
Mangatangi	31.0	37.5	48.5	71.5	40.3	49.0	61.9	85.0	53.4	65.0	85.9	31.0
644616	29.6	36.8	47.0	75.5	36.6	45.1	55.0	81.5	48.9	59.3	74.4	29.6
647513	26.4	31.0	38.8	52.5	32.2	38.5	46.1	66.6	43.0	48.6	63.1	26.4
647727	25.0	29.5	38.8	54.5	30.7	39.0	50.0	67.3	40.2	49.4	60.2	25.0
648513	32.0	38.1	46.4	67.4	39.2	46.0	55.4	76.7	53.0	63.7	76.5	32.0
648615	25.4	30.5	37.0	50.0	31.3	38.7	46.8	62.6	44.3	54.0	64.2	25.4
648717	25.4	31.4	37.8	51.4	31.5	37.9	48.5	64.5	45.6	54.2	66.7	25.4
648718	25.5	29.8	37.4	50.5	30.7	37.7	48.9	64.7	44.2	55.0	66.7	25.5
649713	26.0	31.0	38.5	55.4	32.2	40.0	49.6	67.6	46.6	58.5	72.0	26.0
649714	23.9	29.1	36.6	46.9	30.1	36.7	45.2	60.3	41.1	51.6	66.6	23.9
649818	25.7	29.1	38.3	49.4	31.4	38.2	46.9	65.6	44.4	53.5	67.6	25.7
742736	24.2	27.5	35.8	45.8	30.2	36.0	43.9	54.1	42.5	48.8	56.2	24.2
742914	27.0	31.3	37.7	48.6	32.9	38.3	47.9	65.7	45.4	53.0	65.3	27.0
Mean	27.0	32.1	40.1	55.3	33.0	39.5	49.0	66.8	44.1	52.9	65.1	27.0
StD	2.1	2.9	3.9	7.7	3.1	3.5	4.8	7.3	4.3	5.7	7.4	2.1
Cv	7.8%	9.1%	9.8%	13.9%	9.3%	8.8%	9.8%	10.9%	9.7%	10.7%	11.3%	7.8%
Max	32.0	39.7	52.5	75.5	40.3	49.0	61.9	85.0	53.4	65.0	85.9	32.0
Min	23.9	27.5	35.8	45.8	28.5	34.0	42.6	54.1	37.7	43.1	52.0	23.9
Range	8.2	12.2	16.7	29.7	11.8	15.0	19.3	30.9	15.7	21.9	33.9	8.2

Table (A2): The 2-year 24 hour Annual Maximum rainfall depth and WQSD values.

Station ID	Rainfall Depth (mm)	WQSD (mm)
643510	96.81	32.27
643713	137.91	45.97
647510	87.58	29.19
647601	92.56	30.85
647614	99.16	33.05
648510	82.65	27.55
648612	83.78	27.93
648613	80.96	26.99
648614	84.03	28.01
648719	89.38	29.79
649625	90.24	30.08
649820	73.31	24.44
740815	80.06	26.69
741813	75.98	25.33
750010	88.27	29.42
NSCC07	88.45	29.48
Wairoa	92.48	30.83
Lower Huia	103.84	34.61
Mangatangi	105.48	35.16
644616	124.09	41.36
647513	89.54	29.85
647727	89.63	29.88
648513	100.74	33.58
648615	79.99	26.66
648717	77.45	25.82
648718	83.62	27.87
649713	83.86	27.95
649714	76.62	25.54
649818	75.17	25.06
742736	74.44	24.81
742914	78.95	26.32
Maximum	137.9	45.97
Minimum	73.3	24.44

Table (A3): Rainfall Event Depth Estimates and summary statistics for different percentiles.

Station	Percentile			
	80 %	**85 %**	**90 %**	**95%**
643510	22.3	26.0	32.4	44.2
643713	26.9	33.4	41.7	59.5
647510	21.2	25.6	31.9	43.1
647601	20.8	24.4	30.1	41.5
647614	21.4	25.8	31.3	42.8
648510	21.9	25.0	30.0	41.9
648612	21.5	23.8	30.1	39.8
648613	21.1	24.5	28.6	39.5
648614	21.1	24.6	29.6	40.1
648719	21.2	25.3	31.3	42.1
649625	22.0	25.8	30.7	42.2
649820	20.9	24.0	28.2	38.8
740815	21.2	24.9	29.7	41.2
741813	21.5	24.5	28.7	36.7
750010	23.3	26.8	31.2	41.7
nSCC07	21.2	24.9	31.3	42.9
Wairoa	23.8	26.9	33.2	42.0
Lower Huia	22.5	26.3	33.1	44.1
Mangatangi	25.2	30.6	36.8	51.3
644616	23.0	28.3	35.1	53.1
647513	21.4	25.5	30.4	42.3
647727	22.3	25.9	32.9	43.5
648513	23.6	27.4	32.9	44.4
648615	20.1	23.5	27.9	38.8
648717	20.4	24.3	29.5	38.7
648718	20.8	24.4	29.7	39.9
649713	21.3	24.2	29.1	41.4
649714	20.3	23.4	29.9	38.3
649818	20.4	25.3	29.3	40.7
742736	21.1	24.4	29.7	39.3
742914	22.0	25.6	31.0	39.1

Mean	21.8	25.6	31.2	42.4
StD	1.5	2.1	2.8	4.6
CV	6.7%	8.1%	8.9%	10.9%
Max	26.9	33.4	41.7	59.5
Min	20.1	23.4	27.9	36.7
Range	6.8	10.0	13.8	22.9