Analyses of suspended solid and nutrient loading in catchments with mixed landuse in Kranji, Singapore

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TECHNOLOGICAL
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**Analyses of Suspended Solid and Nutrient Loading in
Catchments with Mixed Landuse in Kranji, Singapore**

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SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING

2009

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ABSTRACT

Urbanization has occurred in Singapore over the recent decades concurrent with the growth of Singapore's population and economy. The process of urbanization has significantly impacted both the storm runoff volume and the timing and magnitude of the peak runoff rate. Urbanization also increases the variety and amount of pollutants transported to receiving waters, causing surface water quality deterioration. Studies on stormwater quantity and quality are hence vital for planning and managing water resources for catchments subjected to human perturbations. The objective of this study is to develop an approach for estimating long term runoffs and pollutant loadings for the Kranji catchment in Singapore, particularly as functions of land use.

The study first established the rainfall-total runoff relationships and total runoff-pollutant loading rate relationships in Kranji Catchment, Singapore. Storm water data were measured and used for calibration and verification of the XP-SWMM model. The calibrated XP-SWMM model was then applied for continuous simulation of catchment runoffs over the 2005-2007 period. The results from XP-SWMM simulations showed that the model is capable of providing good results for continuous flow simulations, and is highly efficient for the estimation of urban storm water direct runoff volumes.

The relationship between rainfall and runoff for the gauged period at each study site shows good correlations. The runoff coefficient (total flow/rainfall ratio) is found to be a function of the total rainfall and land use. In comparing gauging stations CP2 with CP4, the average runoff coefficient is about 3 times higher for CP2, which has the largest proportional area which is developed, around 68%, comprising mainly residential land use with high impervious land cover. In contrast, CP4, which has the largest proportional previous areas, has the lowest runoff coefficient of 0.13.

This study covered thirteen water quality parameters which are considered relevant for water quality management: ammonium-equivalent nitrogen ($\text{NH}_3\text{-N}$), dissolved organic carbon (DOC), particulate organic carbon (POC), total nitrogen (TN), total dissolved nitrogen (TDN), nitrate+nitrite (NO_x), dissolved organic nitrogen (DON), total phosphate (TP), total dissolved phosphate (TDP), ortho-phosphate (OP), dissolved organic phosphorus (DOP), silica (SiO_2), and total suspended solids (TSS). A good

knowledge of the relative pollutant load contributions from dry-weather flow (DWF) and wet-weather flow (WWF) could provide useful guides for implementing effective and efficient water quality management measures for the sub-catchments. This study uses a regression approach to estimate the WWF loads, and uses the monitored data to estimate the DWF loads. The annual DWF and WWF pollutant loadings were characterized over the 2005-2007 period. For nearly all the pollutants studied, contributions from WWF are greater than DWF at CP1, CP2, CP6 and CP7. However, almost all quality parameters show larger contributions from DWF than WWF at CP4, except for TP, TDP, DOP, OP and TSS. The results suggest that DWF quality control measures may be important for CP4. On the other hand, WWF quality management may be important for CP1, CP2, CP6 and CP7.

The analytical approach developed in this study can be applied to other ungauged watersheds near the study site. The results of this study will provide a better understanding on both the flow and pollutant loading from the sub-catchments which will aid in the overall management objective of nutrient load reduction.

Keywords: Stormwater, runoff coefficient, land use, event mean concentration, pollution loading rate.

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Chapter 1

Introduction

1.1 Background

Impact of human perturbations through urbanization has been one of the key reasons for causing changes in land surface hydrology. Urbanization processes involving removal of native vegetation from natural landscape, construction activities, agricultural activities, industrializations and other perturbations have adverse impacts on both storm water quantity and quality. In natural watersheds, terrains like forests, wetlands, and grasslands can trap rainwater, allowing them to undergo evapotranspiration and infiltration processes, before net runoff reaches the receiving water body. In urbanized watersheds, the impervious surfaces, such as streets, roofs, parking lots and manicured lawns etc., can channel rainwater effectively and eventually induce greater net runoff to reach the receiving water compare to that before urbanization. Thus, due to decreased infiltration, excess rainfall causes higher quantity of storm water runoff and consequently escalates the flood peaks, reduces the quantity of baseflow, and increases pollutant concentrations to water bodies. Furthermore, climatic change associated with global warming has affected rainfall patterns and hydrological processes, resulting in increases in storm runoff rate and volume (Neff et al., 2000).

The traditional watershed pollutant control strategy usually focuses on the control of point sources. However, the major sources of pollution of Kranji watershed in Singapore, comprising urban or forested areas, are non-point sources (NPS). Therefore long term monitoring and modeling of runoff quality should take into account the dominance of non-point sources. Urban areas contribute more pollution than forested catchments (Wotling and Bouvier, 2002). Urban activities increase the contribution of pollutants to receiving waters.

Urbanization of the Kranji watershed may have changed the quantity and quality of runoff into the Kranji Reservoir. Kranji Reservoir is one of major the sources of raw water supply in Singapore. The growth of blue-green algae in the reservoir has been

recognized in recent years, which could result reservoir eutrophication. To reduce the pollutant loading in storm runoff, knowledge of relative pollutant contributions from different land uses is important.

Generally, storm runoff from the Kranji catchment discharges directly into the Kranji reservoir. The land areas around the reservoir have been increasingly developed as sites for farming and commercial-industrial activities. There has hence been an increase in waste discharge in the reservoir. The pollutant discharge from the Kranji catchment could have serious long term repercussions on the reservoir such as eutrophication problem. Thus, knowledge on the sources of pollutants is critical, and could help in controlling the pollutant load. In this study, the impacts of land uses on water quality and quantity will be investigated for sub-catchments CP1, CP2, CP4, CP6 and CP7.

1.2 Objective and Scope of the Study

The objective of this study is to develop an approach for estimating the long-term runoffs and pollutant loadings for Kranji catchment in Singapore, as functions of land use. Wet-weather and dry-weather loading rates in each study site will also be analyzed.

The scope of this study is to set up and calibrate the XP-SWMM model to a number of gauged watersheds within the Kranji catchment. Furthermore, this study also derives correlation relationships between rainfall characteristics and storm water quality and pollutant loading rates, taking land use characteristics into consideration. In addition, the concept, definition and existence of the first flush and second flush phenomena as applied to storm flows in Singapore are also examined.

1.3 Organization of the Thesis

This study first collected rainfall and direct runoff data to calibrate a deterministic rainfall-runoff model for continuous runoff prediction in Singapore. The baseflow calculations were based on empirical equations developed by the Lim et al. (2008). Total flows were divided into dry weather flow and wet-weather flow components. Wet-weather pollutant loading rates were based on EMC (event mean concentrations) and regression rating equations. The dry weather loading rates were based on averaged

concentrations. The study procedures are shown in Figure 1.1.1.

This thesis is separated into 6 chapters. Chapter 1 describes the background and motivations of this study. Chapter 2 reviews the relevant literature and discusses the influence of land use on runoff quantity and quality. Chapter 3 describes the study sites. Chapter 4 introduces the XP-SWMM model and the methodology of estimating the loading rates. Chapter 5 compares the wet-weather flow, dry-weather flow and the pollutant loading rates in each study site. Chapter 6 presents the conclusions and recommendations.

A study on characterizing the baseline and storm water quality of five sub-catchments within the Kranji reservoir catchment in Singapore has been conducted, and the results were reported in the Final Project Report (NTU, 2008). In the study, the techniques for hydrograph separation and calibration of the SWMM model for the sub-catchments were also carried out. The present study applies the results obtained from the previous works to estimate the pollutant loadings for the five sub-catchments.

Figure 1.1.1 Study flow chart

Chapter 2

Literature Review

2.1 Storm Runoff Modeling

Rainfall-runoff modeling is an important tool for water resources planning, reservoir operation and flood prediction. Many rainfall-runoff models exist today. They can be broadly categorized into two main types, namely deterministic and stochastic models. (Eagleson, 1970; Stephenson and Meadows, 1986; Braud et al., 1999; Hromadka and Whitley, 1999). The classification of the models is shown in Figure 2.1.1.

Figure 2.1.1 Classification of rainfall-runoff models
(Chow et al. (1988), Applied Hydrology)

Deterministic models can be classified according to whether the hydrological processes involved are empirical, conceptual or distributed. Empirical black box models are developed using measured time series instead of utilising mathematical expression describing the physical processes in a catchment. Several types of empirical models exist. One group of empirical models are based on using statistical methods such as ARIMA (Autoregressive Integrated Moving Average). Another group of empirical models are based on the unit hydrograph concept. The third group of empirical models are data-driven

models using methods such as artificial neural networks (ANN), etc.

In lumped conceptual models, the parameters and variables represent average values over the entire catchment. The hydrological processes are described by semi-empirical equations with a physical basis. The model parameters are usually assessed through a model calibration process.

In physically-based distributed models, processes are represented by one or more partial differential equations and the equations and parameters are distributed in space. The flows of water and energy are directly calculated from the governing continuum equations, such as the Saint Venant equations for overland and channel flow. Distributed models can be applied to catchments with complex channel networks, and varying spatial distributions of land use, soil type and vegetation cover, etc. However, stochastic model is more suitable for events with large or random variations, because the actual output could be quite different from the single value a deterministic model would produce.

Unit hydrograph is another empirical approach for presenting rainfall-runoff relationship. A unit hydrograph is a time-record of stream discharge for unit rainfall input to watershed, derived from rainfall-runoff measurements. In unit hydrograph theory, the rainfall-runoff process is assumed to be a linear relation, and the rainfall is assumed to be uniformly distributed over the entire watershed, and applied at a constant rate over a given duration. Consequently, unit hydrograph has its limitations in application. Its application is usually restricted to small-scale experimental watersheds from 1ha to 25km² (Chow et al., 1988). The actual resultant rainfall-runoff relationship of watershed is determined by various hydrologic and geomorphologic factors, and usually has a non-linear characteristic.

Generally, there are four types of rainfall-runoff routing procedures: physical-based, conceptual, metric and hybrid metric-conceptual models (Young, 2003). A physical-based model is often limited to the solving of interrelated problems, and its application is confined to small-scale watersheds, because it easily reacts to changes in land use (Ramos et al., 1995). A conceptual model is more widely adopted in practice for

a variety of catchments, and consists of empirical relations and physical-based models. Nash (1957) developed the cascade of identical linear reservoirs model. Dooge (1959) proposed the spatially distributed model. Sugawara (1976) proposed the tank model. The concept of this model is uncomplicated and easy to understand. Hsieh and Wang (1999) developed a semi-distributed parallel-type linear reservoir rainfall-runoff model. A metric model is more of a black box that has no insights on the physics of rainfall-runoff processes, and depends solely on rainfall-runoff systematic input and output. The Artificial Neural Networks (ANN) is one example, capable of solving difficult issues such as high nonlinearity and huge amount of variables involved in rainfall-runoff transformation. A hybrid metric-conceptual model combines both metric and conceptual models. An example is the IHACRES model (Identification of unit Hydrographs and Component flows from Rainfalls, Evaporation and Streamflow data) (Jakeman et al., 1990; Littlewood and Jakeman, 1994).

In recent years, many deterministic rainfall-runoff models for urban watersheds have been developed in different countries. It is often used to design hydraulic structures to transport storm water runoff. Examples of the more widely used models include the Storm Water Management Model (SWMM) by U.S.A Environmental Protection Administration (EPA), MIKE-SHE by Denmark's Danish Hydraulic Institute (DHI), Info Works by England's Wallingford, SOBEK by Holland's Delft Hydraulics, HEC-HMS by US Arm, Corps of Engineers and WMS developed by the Environmental Modeling Research Laboratory. The SWMM model originally developed by EPA (1971) is a dynamic rainfall-runoff simulation model, which is used to simulate the quantity and quality of urban storm runoff through systems of pipes and channels. The flow routing for surface and sub-surface transport and groundwater systems include the options of nonlinear reservoir channel routing and fully dynamic hydraulic flow routing. It can be applied for single-event or long-term continuous simulations using various time steps. SWMM's full dynamic hydraulic flow routing option can simulate backwater, surcharging, pressure flow, and looped connections.

The current editions of XP-SWMM, PCSWMM and MIKE SWMM, which are developed by the XP SOFTWARE Company of U.S.A, Computational Hydraulics (CHI)

of USA, and Danish Hydraulic Institute (DHI) of Demark respectively, provide a graphical user interface (GUI). The GUI assists users in determining the amount of data that are adequate to simulate the response of a system, and also to facilitate data input and streamline the data analysis process. These are completely rewritten with innovative model creation, creative input and output visualization tools. They can be also combined with the relevant database software. The software combines the functions of XP-SWMM, PCSWMM and MIKE SWMM. The software can be combined with a geographical information system (GIS) and database software. The geographical information systems (GIS) background layer supports Arc View, ARC/INFO, MapInfo, Access, Excel, and etc.

SWMM (Huber and Dickinson, 1988) was originally developed for urban area setting; however, Jang et al. (2007) applied SWMM to four planned development areas in Korea, found that it is also well suited for model natural watersheds. SWMM has been applied in water quantity and quality studies (Tsihrintzis et al., 1995; McPherson et al., 2005; Chen and Adams, 2006). Generally, SWMM performs well in predicting both the quantity and quality parameters (Tsihrintzis and Hamid, 1998). Moreover, Chen and Adams (2006) revealed that SWMM to be plausible for long term rainfall-runoff simulations. Warwick and Tadeballi (1991) applied the SWMM model to study hydrological processes on 10-square-mil urbanized residential area in Dallas, Texas. The simulated results performed well in predicting both total runoff volume and peak flow rate. Tsihrintzis et al. (1998) utilized SWMM to simulate the quantity and quality of urban storm runoff in South Florida representing high- and low-density residential, commercial, and highway land uses. Results from Choi and Ball (2002) hydroinformatic approach in Centennial Park, Sydney suggest that the new approach can be used effectively to evaluate catchment modeling system control parameters, and to improve the accuracy and efficiency of the catchment modeling system calibration process. These studies mainly focused on continuous simulation using storm events or synthetic hyetographs of long return periods. Several studies have applied the SWMM model to Singapore catchments. Liong et al. (1991) used knowledge-based systems. Liong et al. (1995) combined genetic algorithm with SWMM model to determine the control parameter values. Furthermore,

application of sophisticated mathematical search algorithms also was undertaken by Liong and Ibrahim (1994), who used optimization algorithms for catchments calibration. These studies, applied to a catchment in Singapore of about 6.11 km², showed that SWMM was able to accurately predict peak flow. SWMM model parameter estimation was integrated with PEST Version5 by Tan et al (2008). It was found that SWMM provided reliable continuous simulation for runoff volume. The present study utilizes the results obtained from the previous studies, and presents the calibration and verification of SWMM using single-storm events recorded from five sub-catchments within the Kranji watershed, which comprises various land uses (i.e. residential, reserved site, woodland, agriculture and cemetery).

Land use plays an important role in driving hydrological processes within watersheds. Runoff characteristics of a watershed, such as the volume and timing of runoff and maximum flood flow rates have been significantly impacted by land cover change. Barringer et al. (1994) and Jon et al. (2006) indicated that an urban watershed tends to lead to enhanced peak discharges, and baseflow inputs in urban streams are lower than other watersheds. Researchers such as Luna (1968), Simmons and Reynolds (1982) and Warner (1984) have demonstrated that baseflow in developed areas tends to decrease due to the effects of impervious surface limiting infiltration and enhancing evaporation.

When assessing runoff volume, researchers only focus on single events, and traditional hydrologic methods seldom focus on estimating the long- term hydrologic impact of land-use change. However, long-term simulations are necessary since much of the runoff from urbanized watersheds derive from smaller-intensity, high-frequency storms (Burges et al. 1998). Kim et al. (2002) demonstrated land use change can have a dramatic impact on annual runoff volume by employing GIS-based SCS CN method and the L-THIA GIS model to estimate rainfall event runoff and average annual runoff for NASA's John F. Kennedy Space Center and the India River Lagoon Watershed. Alterations in hydrology stemming from land use change can have negative impacts on ecological processes (Paul and Meyer, 2001). Thus, the effects of land use change on annual or long term runoff should be considered in land use planning, such that forests and grasses are considered as elements of flood prevention and maintenance of habitat

stability.

2.2 Impact of Non-Point Source Pollution

Sources of urban runoff pollutants can be classified into point sources and non-point sources. Point source discharges pollutants from single source at a discrete point. Such sources are usually associated with disposal of water from industrial, commercial or municipal sources. Point sources can feasibly be abated or controlled through the use of wastewater treatment technologies. On the contrary, Non-Point source (NPS) pollution of receiving waters comes from runoff constituents distributed diffusely over the land. Such sources are difficult to control. The impacts of NPS pollution include pollutant shock loading into reservoirs, decrease in dissolved oxygen, and increase in eutrophication, which could result in long term ecological change. Therefore, source water quality protection should be of great concern.

Table 2.2.1 Nonpoint source pollutants and major sources

Sediment	Nutrients (fertilizers, grease, organic matter)	Acids & Salts	Heavy Metals (lead, mercury, zinc)	Toxic Chemicals (pesticides, organic, inorganic compounds)	Pathogens (bacteria, viruses)
<ul style="list-style-type: none"> * Construction sites * Mining operations * Croplands * Logging operations * Stream bank erosion * Shoreline erosion * Grazed woodland 	<ul style="list-style-type: none"> * Croplands * Nurseries * Orchards * Livestock operations * Gardens, lawns, forests * Petroleum storage areas * Landfills 	<ul style="list-style-type: none"> * Irrigated lands * Mining operations * Urban runoff, roads, parking lots * Landfills 	<ul style="list-style-type: none"> * Mining operations * Vehicle emissions * Urban runoff, roads, parking lots * Landfills 	<ul style="list-style-type: none"> * Croplands * Nurseries * Orchards * Building sites * Gardens, lawns * Landfills 	<ul style="list-style-type: none"> * Domestic sewage * Livestock waste * Landfills

Source: Ohio State University, 1992

Knowing about the sources of pollutants is important, and could help in controlling the pollutant load. Thus, it is necessary to recognize the pollutants contributed from different land uses. The main sources of pollutants are briefly summarized in Table 2.2.1.

Table 2.2.2 Pollutant impact on water quality

Pollutant	Impact on water quality
Sediment	Aesthetics, Water Supply, Aquatic Life, Recreation
Nutrients Nitrogen Phosphorus	Eutrophication, Water Supply, Recreation
Toxic Chemicals Pesticides Heavy Metals Industrial Chemicals Petroleum Products	Aesthetics, Water Supply, Aquatic Life, Wild Life, Food Chains
Pathogens	Water Supply, Recreation
pH	Aquatic Life

Urbanization and agricultural areas increase the variety and amount of pollutants transported to receiving waters, and are important causes of surface water quality deterioration. Urban land use such as construction, industry, commerce, streets and roads, residential areas contribute pollutant constituents in urban runoff. These pollutants include suspended solids, bacteria, heavy metals, oxygen demanding substances, nutrients, oil and grease. Agricultural runoff is a major contributor which led to nutrient enrichment of water bodies. In addition, pesticides, sediments, nutrients, organic materials and pathogens related to fertilizer and pesticide applications are transported from agricultural areas. These contaminants can have physical, chemical and biological impact on water bodies, resulting in ecological and environmental inequality (Field, 1998). A brief summary of pollutant impacts on water quality is shown in Table 2.2.2. Nutrients such as carbon, nitrogen, phosphorus, and iron, are essential for algal growth, and could result in eutrophication. Recently, for urban runoff management, Best Management Practices (BMPs) are increasingly used for reducing pollutants at the source, minimizing erosion, and managing runoff quantity and quality. Examples of BMPs include ponds, bioretention facilities, infiltration trenches, grass

swales, filter strips, dry wells, and cisterns. “Low impact development” (LID) or hydrologic source control strives to retain a site’s pre-development hydrologic regime, is also used for reducing WWF and the associated NPS pollution.

Study of urban stormwater quality problems is necessary to understand the characteristics of runoff pollutants and the impacts on receiving waters. The time scale of pollutant effects on the receiving water body is influenced by the characteristics of the various pollutants as shown in Figure 2.2.1. Short-term effects are normally associated with bacteria, biodegradable organic matter and hydraulic effects. Long term effects tend to be associated with suspended solid, nutrients and heavy metals. In order to control the problem, long term monitoring and modeling of runoff quality is necessary. Therefore, annual pollutant loadings were estimated in this study.

Figure 2.2.1 Time scale effects of runoff quality constituents (after U.S. EPA, 1979)

Urban land uses are defined by different degrees of intensity which are related to the potential for pollution. The most widely used measure of urbanization intensity is

proportion of impervious area. Many investigators have demonstrated that increase in pollutant loading from stormwater is related to the extent of impervious area (Luna, 1968; Wothling and Buvier, 2002; Arnold and Gibbons, 1996). Wothling and Bouvier (2002) found that in the urbanized catchment, the TP and TN loads are four and three times greater than those in forested catchment, respectively. In addition, the organic discharge is primarily a result of an increase in the TSS load. Through proper site design and land use management, these impacts can be reduced. Thus, the understanding of pollution sources is important for the prediction and the control of pollutant loads.

Traditionally, streams are only sampled during low-flows in dry season. However, now that the importance of NPS has been recognized due to the fact that causes more pollution than baseflow, sampling during high is required. Numerous studies have been carried out globally on control of non-point source pollutant since the early 1970s. James and Robinson (1986) estimated that wet weather surface runoff and combined sewer overflows contribute 93% of the total suspended solid, 78% of the total biochemical oxygen demand, 66% of the total phosphorus and 45% of the total nitrogen loads to the lake. Pionke et al. (1996) found almost 60% of nitrate loading during non-storm periods, and about 70% of TP load during storm period in agriculture area. Jha et al. (2005) estimated nutrient outflow from agricultural watersheds to the Kali River in India and found higher outflow during the monsoon period than during the non-monsoon period.

Pollutant concentration tends to increase rapidly at the beginning of a storm event, a phenomenon described as the first flush phenomenon. The concept of first flush was first advanced in the early 1970s. Barrett et al., (1998) and Hewitt and Rashed (1992) defined the first flush phenomenon as the occurrence of higher pollutant concentrations at the beginning of a storm event. Researchers have discovered that various factors may be responsible for the occurrence of first flush events. Gupta and Saul (1996) wrote that first flush is influenced by many factors, such as contributing impervious area, antecedent dry weather period (ADWP), rainfall intensity and watershed area. However, Lee et al. (2002) found no correlation between the first flush behavior and ADWP, but the first flush phenomenon was greater for smaller watershed areas. Furthermore, Sansalone et al., (1998) found that longer duration, lower intensity events exhibited stronger first flush of SS.

Complete water quality data of an entire storm event is difficult to obtain, because sampling of early flows, which may be more highly polluted, is usually missed to be triggered (Osborn and Hutchings, 1990). Due to inadequacy of data, it is difficult to generate complete pollutographs (concentration-time plots) or loadographs (mass-time plots). Therefore, event mean concentration (EMC) is widely used to represent NPS pollution (Wanielsita and Yousef, 1993). EMC is the ratio of total pollutant mass to the total runoff volume of an event. The median and mean EMC are used to represent quality of storm water. Generally, quality of storm water is affected by many factors, such as rainfall intensity and duration, number of antecedent dry days, land use, etc. Statistical models are required for many of these influence factors, but they do not have good predictions for specific events. Chen and Adams (2006) indicated that the representative statistical average over many storm events would perform well for annual loadings.

Chapter 3

Study Site Description

3.1 Kranji Reservoir

Kranji Reservoir (1°25'N, 103°43'E) is one of the largest reservoirs in Singapore, and is situated in the northwest region of main Singapore Island. Figure 3.1.1 shows the location of the Kranji reservoir. The Kranji catchment drains into the Kranji Reservoir, which is a major source of raw water supply in Singapore. The total area of the Kranji catchment is approximately 5,450 ha, while the surface area of the Kranji Reservoir is about 200 ha. The maximum storage capacity of the reservoir is about 22.5 million m³; the maximum depth of the reservoir is about 18m.

Figure 3.1.1 Map of Singapore's reservoirs

Source: Public Utilities Board cited in <http://homepage.mac.com/voyager/NoPlace/ctlb.html>

3.2 Kranji Watershed

The Kranji watershed comprises 4 major tributaries flowing into the Kranji Reservoir. These are Sg. Kangkar, Sg. Tengah, Sg. Peng Siang and Sg. Pangsua. Each tributary drains storm runoff from its surrounding catchments.

Figure 3.2.1 Major tributaries of the Kranji Catchment

Figure 3.2.2 Land use of the Kranji Catchment

The headwaters of Neo Tiew and Kangkar are in the northern portion of the watershed. The developed residential areas within the Kranji watershed are estimated at about 110 ha, and continue to grow. A Kranji catchment map is shown in Figure 3.2.1. The pertinent information for each of the major sub-catchments is summarized in Table 3.2.1. The land-use map was classified into 5 classes (i.e. Residential, grassland, recreational, agriculture, undeveloped) as shown in Figure 3.2.2.

Table 3.2.1 Pertinent information of the sub-catchments

Sub Name	Total Area (ha)	Industrial (%)	Agricultural (%)	Residential (%)	Undeveloped (%)
Kangkar	872	0.00	6.10	0.00	93.90
Tengah	993	0.00	13.40	0.00	86.60
Peng Siang	1334	0.08	7.44	33.16	59.32
Pangsua	1570	0.00	0.00	40.00	60.00

3.3 Gauging Station

Figure 3.3.1 Catchment and reservoir sampling locations

Five catchment gauging stations located in storm drains near Bricklands Road (CP1), Choa Chu Kang Walk (CP2), Tengah Airbase (CP4), Ama Keng (CP6) and Pangsua (CP7) were set up within the Kranji Catchment for monitoring of the hydrology data, where the stormwater quantity and quality are measured as shown in Figure 3.3.1. The locations of the gauging stations are shown in Figure 3.3.2 to 3.3.6. The present study sites are CP1 and CP2 situated along the sub-tributaries of Peng Siang, CP4, CP6 and CP7 located along the main tributaries of Tengah, Kangkar, Pangsua, respectively.

Automatic flow measuring and sampling systems were installed and operated at the five stations. The gauging station at CP1 was set-up and has been operating since 15 December 2004, whereas the gauging station at CP2 was set-up and began operation on 2 July 2006. Both CP1 and CP2 are located within the Peng Siang catchment. Gauging stations CP4, CP6, CP7 began operation on 1 April 2007, and are located within the Tengah, Kangkar and Pangsua catchments. The sampling periods are shown in Table

3.3.1. Each gauging station is fitted with a combined level and velocity sensor, a tipping bucket rain gauge, an auto-sampler, a data logger and a modem. All the data including flow level, velocity and rainfall are logged at 5 minutes intervals.

Table 3.3.1 Sampling period for each study site

Site	Sampling period	Number of storm samples
CP1	Jun 05 – Nov 06	17
CP2	Oct 06 – Aug 07	14
CP4	Apr 07 – Aug 07	4
CP6	Apr 07 – Aug 07	6
CP7	Apr 07 – Aug 07	8

Each station is equipped with an auto-sampler, which is triggered during storm flow events. The date and time the samples are collected are also recorded by the data logger. The auto-sampler is triggered by the water level in the drain. Whenever there is a storm event and the water level in the drain exceeds 0.4m, the auto-sampler will be triggered to collect storm water samples at 10-minute intervals until 24 samples were collected. The volume of each storm sample is 1 L. Storm samples are then brought back to the laboratory for analysis. Experiments were conducted on eleven water quality parameters ($\text{NH}_3\text{-N}$, TOC, DOC, TN, TDN, NO_x , TP, TDP, OP, SiO_2 , and TSS). A stormwater abstraction pond is located within the Pangsua catchment which collects and pumps excess stormwater to some other storage reservoir. The data collected were used to estimate the quantitative relationships between rainfall, loading rates and runoff quantity and quality.

**Figure 3.3.2 Sampling site (CP1) near
Bricklands road**

**Figure 3.3.3 Sampling site (CP2) near
CCK Ave 4**

**Figure 3.3.4 Sampling site (CP4) near
Tengah Airbase**

**Figure 3.3.5 Sampling site (CP6) near
Ama Keng**

Figure 3.3.6 Sampling site (CP7) along Sg Pangsua

Table 3.3.2 Land uses of each gauging station

Unit: %

Site	Area (ha)	Institutional	Reserve site	Agricultural	High density residential	Residential/commercial	Cemetery	Forest	Golf course
CP 1	522.2	0.0	10.5	0.0	36.0	3.0	0.0	50.0	0.5
CP 2	198.7	0.0	15.0	0.0	68.0	0.0	0.0	17.0	0.0
CP 4	288.0	8.0	54.0	0.0	0.0	0.0	23.0	15.0	0.0
CP 6	145.0	0.0	78.0	15.0	0.0	0.0	4.0	3.0	0.0
CP 7	1,556.7	0.0	19.0	0.0	32.5	13.5	0	35.0	0.0

Figure 3.3.7 Land use of each study site

Figure 3.3.7 illustrates the land uses of each study site. The total catchment areas of the gauging stations are shown in Table 3.3.2. The land use data are classified to 8 categories. The total catchment area of CP1 is about 522.23 ha, consisting of 36% of residential and extensive commercial areas, while the rest is still in a natural state. CP2 is characterized by high urbanization, and contains 68% of high density residential area, 17% of woodland and 15% of reserve site. CP4 has the largest cemetery coverage at around 23%; and an army logistics base which covers about an additional 8% of the catchment. Agricultural area occupies about 15% of the total area of CP6, while the rests of the catchment are still mainly in a natural state or special use such as recreational land use. CP7 has the largest area compared to other study sites, at about 1556.65 ha. The land uses of the contributing drainage area are approximately 47% urban, 35% forest, 19% reserve site. The remaining area includes 2 quarries, a pond and highway.

3.3.1 Channel Cross-Section

The storm drain at each gauging station has uniform cross-section as shown in Figures 3.3.8 to 3.3.12.

Figure 3.3.8 Channel cross section at CP1 (Bricklands) Source: (NTU, 2008)

Figure 3.3.9 Channel cross section at CP2 (Choa Chu Kang Avenue 4) Source: (NTU, 2008)

Figure 3.3.10 Channel cross section results at CP4 (Tengah Airbase) Source: (NTU, 2008)

Figure 3.3.11 Channel cross section at CP6 (Ama Keng Road) Source: (NTU, 2008)

Figure 3.3.12 Channel cross section at CP7 (Sg Pangsua) Source: (NTU, 2008)

3.4 Meteorological Condition

Singapore's weather is warm and fairly humid. Its temperatures throughout the year is approximately 30°C during day and 23°C in the evening. The average relative humidity is about 84%. Singapore is not short of fresh water as it receives an average of around 2,400 mm of rainfall annually, well above the global average of 1,050 mm. The monthly rainfall data in year 2005 to 2007 are shown in Table 3.4.2. The only constraint faced by the country is capturing and storing as much of this rainfall as possible, on limited amounts of land areas. There are no distinct seasons, or dry and wet periods. Most of the rain falls during the northeast monsoon season from November to January and showers are usually sudden and heavy. The key climatological data are summarized in Table 3.4.1.

Table 3.4.1 Main climatological information at Changi Airport (1961-1990)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Max Temp Degree Celsius	29.9	31	31.4	31.7	31.6	31.2	30.8	30.8	30.7	31.1	30.5	29.6
Mean Temp Degree Celsius	25.8	26.4	26.8	27.2	27.5	27.4	27.1	27	26.8	26.8	26.3	25.7
Mean Min Temp Degree Celsius	23.1	23.5	23.9	24.3	24.6	24.5	24.2	24.2	23.9	23.9	23.6	23.3
Mean Total Prec (mm)	198	154	171	141	158	140	145	143	177	167	252	304
Mean Monthly Prec Days	12	10	13	14	14	13	14	13	14	15	19	19
Mean Daily Sunshine (hr)	5.6	6.5	6.2	5.8	5.8	5.9	6.1	5.8	5.2	5	4.3	4.3
Mean Monthly Wind Speed(m/sec)	2.7	2.6	1.9	1.2	1.2	1.5	1.6	1.7	1.5	1.3	1.3	2

(1°4'N, 104°0'E), Elevation: 16 m

Table 3.4.2 Monthly rainfall data for 2005 to 2007 in Kranji sub-catchment

Unit: mm							
Year	2005	2006	2007				
Site	CP1	CP1	CP1	CP2	CP4	CP6	CP7
Jan	182.08	528.76	379.4	494.2	477.8	477.8	529.2
Feb	78.68	152.94	138.2	241.8	338.4	338.4	197.4
Mar	139.22	104.77	252.6	414.6	297.4	297.4	347.4
Apr	201.67	241.05	354.6	595.4	456.6	456.6	419.8
May	492.52	131.91	134.4	215.8	192.2	150.8	195.6
Jun	96.79	236.62	129	148.6	193.6	245.2	128.2
Jul	332.68	227.28	152.2	149.4	242.4	242.4	186.4
Aug	194.04	147.6	337.6	663.6	472.8	472.8	392.6
Sep	184.3	197.83	371.6	371.6	176.8	176.8	170.8
Oct	282.97	146.42	369.8	369.8	215.2	215.2	225.6
Nov	162.58	491.714	198	401.4	328.2	328.2	207.6
Dec	215.52	116.4	297	641	467.6	467.6	139.4
Sum	2,563.1	2,723.3	3,114.4	4,707.2	3,859	3,869.2	3,140

Chapter 4

Methodology

4.1 Description of XP-SWMM Model

XP-SWMM is an enhanced version of the SWMM model, with graphical interface. XP-SWMM running under Windows XP, provides an integrated environment for editing data entry for the study area, run-time graphics, and viewing the results in different formats, including color-coded drainage area and transportation system maps, time series graphs and tables, profile plots and others. Drainage networks could include pipes and open channels, rivers, loops, bifurcations, pumps, weirs, and ponds can be imported either from a database or created on the screen over topographical backgrounds.

4.1.1 Overview of XP-SWMM Capabilities

The SWMM model can be used for single event or long-term (continuous) simulation of overland water quantity and quality produced by storms in urban watersheds. The model is made up of several blocks, including the RUNOFF, TRANSPORT and EXTRAN modules as shown in Figure 4.1.1.

The hydrologic processes considered in the model include time-varying rainfall, evaporation, snow accumulation and melting, depression storage, infiltration, interflow and nonlinear reservoir routing of overland flow. SWMM has a flexible set of hydraulic modeling capabilities to route runoff and external inflows through a drainage system network of pipes, channels, storage/treatment units and diversion structures. SWMM can also estimate the production of pollutant loads associated with this runoff.

Figure 4.1.1 SWMM, the Storm Water Management Model, program configuration (after Huber and Dickinson 1988)

The RUNOFF block simulates the runoff produced on the surfaces of a sub-catchment. In addition, it could also simulate the pollution build up and wash off processes. The rainfall-runoff simulation is carried out by a nonlinear reservoir approach and with the concept of surface storage balance, which is illustrated in Figure 4.1.2. The RUNOFF block can also be applied for soil-groundwater modeling, which simulates the rainwater filling up surface storages and infiltrating into the soil profiles. Runoff is produced when the soils are saturated. The simulation is affected by the input parameters, such as basin's percent imperviousness, the type of soil, the basin slope, and the sub-catchment width that affects overland flow routing. Monthly average evaporation rates are directly employed to calculate the amount of water evaporated from the surface. The RUNOFF option can also be implemented with simple flow routing through pipes and open channels. The RUNOFF block uses an algorithm to represent the rainfall losses with the limitation that only the Horton or Green-Ampt infiltration models are supported.

Figure 4.1.2 Nonlinear reservoir conceptualization of overland flow (Huber and Dickinson, 1998)

The TRANSPORT Block simulates the transport and routing of the water through the sewer network, i.e. the Runoff Block hydrographs are input to the TRANSPORT Block. TRANSPORT also has the ability to simulate dry-weather or sanitary sewage flows for routing through a sewer system. The flow in this block is modeled with the kinematic wave approach.

EXTRAN block is a dynamic flow routing model capable of routing inflow hydrograph through an open channel and closed conduit system along with heads throughout the system. EXTRAN is based on the concept of dynamic wave equation, which solves the dynamic equations for gradually varied flow using various explicit solution techniques. The "Link-node" concept is being used, which permits parallel pipes, looped systems, lateral diversions and partial surcharges.

4.1.2 RUNOFF Block Routing Method

The XP-SWMM model was established for predicting direct runoff for five sub-catchments in the Kranji catchment. The 5 sub-catchments are Bricklands (CP1), Choa Chu Kang (CP2), Tengah Airbase (CP4), Ama Keng (CP6) and Pangsua (CP7). The node was created under the RUNOFF Block for each study site, the model interface as shown in Figure 4.1.3. The RUNOFF Block of the model generates direct runoff hydrographs for each sub-catchment.

Figure 4.1.3 XP-SWMM model interface

The conceptual view of surface runoff used by the RUNOFF Block is quite simple and is summarized in the following equations.

1. Rainfall is added to the subarea according to a specified hyetograph:

$$D_1 = D_t + R_t \cdot \Delta t \quad (4.1)$$

where, D_1 : water depth after rainfall

D_t : water depth of subarea at time t

R_t : rainfall intensity in time interval Δt

Δt : time interval

2. Horton's equation is used to compute infiltration, I_t which is then subtracted from the existing water depth of the subarea:

$$I_t = f_o + (f_i - f_o)e^{-\alpha t} \quad (4.2)$$

$$D_2 = D_1 - I_t \cdot \Delta t \quad (4.3)$$

where, D_2 : water depth after accounting for infiltration

f_i : minimum infiltration at $t = \text{infinity}$

f_o : maximum infiltration at $t = 0$

α : decay coefficient

3. If the resulting water depth D_2 is larger than the specified detention depth D_d , an overflow discharge is computed with the help of Manning's equation:

$$V = \frac{1}{n} (D_2 - D_d)^{\frac{2}{3}} S^{\frac{1}{2}} \quad (4.4)$$

$$Q_w = V \cdot W \cdot (D_2 - D_d) \quad (4.5)$$

Where, V : velocity (m/s)
 n : Manning's coefficient
 S : ground slope
 W : width (m)
 Q_w : outflow discharge (m³/s)

4. The resulting water depth of the subarea due to rainfall, infiltration and outflow is computed with the help of the continuity equation as:

$$D(t + \Delta t) = D_2 - (Q_w / A) \cdot \Delta t \quad (4.6)$$

where A is surface area of the subcatchment (m²)

6. Gutter inflow (Q_{in}) is computed as a summation of the outflow from the tributary subarea ($Q_{w,i}$) and the flow rate of immediate upstream gutter ($Q_{g,i}$). Thus

$$Q_{in} = \sum Q_{w,i} + \sum Q_{g,i} \quad (4.7)$$

7. The inflow is added to raise the existing water depth of the gutter according to its geometry as:

$$Y_l = Y_t + (Q_{in} / A_s) \cdot \Delta t \quad (4.8)$$

Y_l : initial water depth of gutter
 Y_t : water depth of the gutter at time, t
 A_s : mean water surface area between Y_l and Y_t

8. Manning's equation is used to compute the outflow of the gutter

$$V = 1/n \cdot R^{2/3} \cdot S^{1/2} \quad (4.9)$$

$$Q_g = V \cdot A_c \quad (4.10)$$

Where, R : hydraulic radius (m)
 S : invert slope
 A_c : cross sectional area at Y_l

Water depth of the gutter as a result of the inflow and outflow is computed by continuity equation as:

$$Y(t + \Delta t) = Y_t + (Q_{in} - Q_g) \cdot \Delta t / A_s \quad (4.11)$$

10. Steps (6) to (9) are repeated until all gutter flows are computed.

11. The flows reaching the point concerned are added to produce the flow hydrograph.

12. Steps (1) to (11) are repeated in succeeding time periods until the complete hydrograph is computed.

The step-by-step description of the solution procedure gives a physical picture of the processes being modeled. The integration of variables in each time increment was originally performed by the modified Euler's two step method. This is now accomplished by the Newton-Raphson method, which produces a smoother hydrograph and more stable solution.

The RUNOFF Block has a limited ability to route flows through simple gutter and pipes using the nonlinear reservoir technique. However, the more sophisticated routines in TRANSPORT and EXTRAN Blocks are almost always employed for this purpose. In this study, RUNOFF Block was applied for catchment.

4.1.3 Rainfall Abstraction Methods

RUNOFF Block considers the depression storage on both pervious surfaces and impervious surfaces, as shows in Figure 4.1.4. The final infiltration rate is the sum of the average depression storage on these surfaces and the infiltration rate is calculated with the Horton's equation. Horton's equation was utilized for simulating infiltration process for pervious areas of the Kranji sub-catchment. Horton's equation is based on empirical observations showing that infiltration rate by an exponential relationship (Horton, 1939). It is valid when the potential infiltration rate is greater than or equal to the rate of surface supply, such as rainfall intensity.

Figure 4.1.4 Representation of the RUNOFF algorithm

4.1.4 Routing Methods

The routing methods of XP-SWMM are separated into two parts, catchment modeling and channel routing. The catchment modeling adopts kinematic wave equation by assuming that there is no backwater in the catchment, that runoff goes in to drain eventually. For overland runoff routing, the catchment is divided into several rectangular sub-catchments. Overland flow is assumed to be steady and uniform and parallel to two of the sides and occurs continuously (Bedient and Huber, 2002).

For channel routing, it describes the dynamic condition of runoff in the concrete channel, since it could have backwater effects when the downstream water level is higher. Thus, dynamic wave model was applied for channel routing. The dynamic wave model describes one-dimensional shallow water waves (unsteady, gradually, open channel), by using St. Venant equation.

In the XP-SWMM model, the kinematic wave equation and dynamic wave equations for a channel are automatically adjusted by the Froude number or the Vedernikov number. For normal flow conditions which without backwater effects (Froude numbers > 1.0 and Vedernikov numbers < 1.0), the kinematic wave equation is adopted for channel routing.

Parameters which affect the routing process are cross-section, longitudinal slope and Manning roughness of the channel. In this study, the cross-section and longitudinal slope of the concrete channels are obtained through field surveys, as shown in section 3.3.1.

4.1.5 Hydrograph Separation

This study utilizes the XP-SWMM model to simulate direct runoff by applying various historical hyetographs to the Kanji catchment. The field data of total runoffs were separated into direct runoff and baseflow using “conceptual method” suggested by Nathan and McMahon (1990):

$$Q_{d,i} = \beta_d Q_{d,i-1} + \frac{1 + \beta_d}{2} [Q_i - Q_{i-1}] \quad (4.12)$$

where Q_d is direct runoff ; Q is total runoff and i is the index for time interval. Mugo and Sharma (1999) found that the β_d parameter can be approximated by recession constant (k). The recession constant (k) can be estimated from the traditional “graphical method” (Vogel and Kroll, 1996). Replacing β_d with k and Q_d with $Q - Q_b$ gives:

$$Q_{b,i} = \frac{1-k}{2}(Q_i + Q_{i-1}) + kQ_{b,i-1} \quad (4.13)$$

when

$$Q_{b,i} \leq Q_i \quad (4.14)$$

where Q_b is baseflow rate.

(Eqs.) 4.12 and 4.13 were used for hydrograph separation in this study. To apply the separation method, the recession constant (k) needs to be estimated for the gauging stations. The results show that a mean recession constant (k) of 0.964, 0.985, 0.97, 0.984, 0.992 was appropriate for the CP1, CP2, CP4, CP6 and CP7 respectively (Lim et al. 2008). The direct runoff hydrographs obtained from the hydrograph separations were used for calibration and verification of XP-SWMM model.

4.1.6 Generation of Baseflow

Long term direct runoffs from each gauging station can be generated by XP-SWMM. The empirical equations developed by Lim et al. (2008) were for generating baseflow from total and direct runoff. Generally, baseflow is not constant during the direct runoff periods. Therefore, the one-parameter digital filter algorithm (Mugo and Sharma, 1999) which has been used for hydrographs separation was applied for generation of baseflow.

Algorithm for Periods with Direct Runoff

Rearranging the digital filter algorithm proposed by Nathan and McMahon (1990) (Eq.) 4.13 yields:

$$Q_{b,i} = \frac{1-k}{1+k}(Q_{d,i} + Q_{d,i-1}) + Q_{b,i-1} \quad (4.15)$$

subject to

$$Q_d > 0 \quad (4.16)$$

where Q_b is the estimated baseflow rate and Q_d is the simulated direct runoff rate from XP-SWMM. In order to make the algorithm practical for total runoff generation, (Eq.) 4.13 can be substituted by:

$$\alpha Q_d \geq Q_{b,i-1} \quad (4.17)$$

where α is a constant. Using (Eq.) 4.13, the baseflow hydrographs can be predicted from direct runoff hydrographs, for a given k and the known initial baseflow rate ($Q_{b,0}$). Baseflow rates of 0.0639, 0.017, 0.0042, 0.0132 and 0.702 m³/s were used as the initial baseflow rates for CP1, CP2, CP4, CP6 and CP7 respectively.

Algorithm for Periods without Direct Runoff

During periods without direct runoff, baseflow undergoes recession. The recession can be modeled based on baseflow recession curves recorded at the study sites using regression method. The current baseflow rate can be estimated from the preceding time step baseflow rate, by quadratic or linear equations:

$$Q_{b,i} = x_1 Q_{b,i-1}^2 + x_2 Q_{b-1} + x_3 \quad (4.18)$$

$$Q_{b,i} = y_1 Q_{b-1} + y_2 \quad (4.19)$$

where x_1, x_2, x_3 are the constant values for the quadratic equation; while y_1, y_2 are the constant values for the linear equation. The representative regression equations developed for the 5 study sites, at 15-min time steps, are given in Table 4.1.1.

Table 4.1.1 Regression equations for period without direct runoff

Study Site	Regression equation for periods without direct runoff
CP1	$Q_{b,i} = -0.0317Q_{b,i-1}^2 + 0.9427Q_{b-1} + 0.0068$
CP2	$Q_{b,i} = -0.282(Q_{b,i-1})^2 + 0.9286(Q_{b-1}) + 0.0019$
CP4	$Q_{b,i} = -0.1276(Q_{b,i-1})^2 + 0.9034(Q_{b-1}) + 0.0085$
CP6	$Q_{b,i} = 0.8203Q_{b-1} + 0.0076$
CP7	$Q_{b,i} = 0.9571Q_{b-1} + 0.0232$

4.1.7 Sensitivity Analyses

Table 4.1.2 Rainfall-runoff parameters of kinematic wave model

Parameter	Unit	Description
W	m	Characteristic width of the sub-catchment
$Imp\%$	%	Percentage of impervious area in a sub-catchment
S_o	m/m	Characteristic slope of the sub-catchment
N_{imp}	-	Manning roughness for the sub-catchment impervious area
N_{per}	-	Manning roughness for the sub-catchment pervious area
D_{imp}	mm	Depression storage for impervious area
D_{per}	mm	Depression storage for pervious area
f_o	mm/hr	The maximum or initial infiltration rate
f_c	mm/hr	The minimum or ultimate value of infiltration rate
k	1/sec	Decay coefficient in the Horton infiltration equation

Major parameters of the model are adjusted according to the trial-and-error curve fitting technique during model calibration. The rainfall hyetograph is the input function and the direct runoff hydrograph is the output function.

The time to peak was found to be affected by N_{imp} , S_o , W , $Imp\%$ and D_{imp} , while peak flow and total runoff volume depended strongly on $Imp\%$, followed by W and N_{imp} . These 4 parameters were thus calibrated using a trial-and-error procedure. The remaining six parameters were estimated based on values recommended in the literature and keeping constant. The main parameters in a kinematic wave routing model are listed in Table 4.1.2. A summary of the values used for the constant parameters is given in Table 4.1.3.

Table 4.1.3 Summary of constant parameters in each study sites

Sub-Catchment	N_{per}	D_{imp} (mm)	D_{per} (mm)	f_o (mm/hr)	f_c (mm/hr)	k (1/sec)
CP1	0.4	0.5	6.0	143.3	2.5	0.00039
CP2	0.6	0.6	7.7	143.3	2.5	0.00039
CP4	0.5	2.0	6.0	112.5	3.8	0.00115
CP6	0.6	1.0	8.0	50.0	2.5	0.00100
CP7	0.5	1.0	9.5	57.7	5.7	0.00130

From the results of sensitivity analyses as plotted in Figures 4.1.5 to 4.1.9, the sensitivity coefficients for $Imp\%$, W , S_o and N_{imp} are significantly higher than those of the other six parameters. From the calibration process, it was observed that the most sensitive parameters involved in the RUNOFF Block are the catchment area, slope, impervious and pervious areas, depression storages, Manning roughness and infiltration capacity. The storm event of 18th Aug 2007 at CP6 was chosen for the sensitivity analysis. The peak flow and direct runoff sensitivity analysis is shown in Figure 4.1.10.

Figure 4.1.5 Effect of different slope on the hydrograph

Figure 4.1.6 Effect of different impervious area on the hydrograph

Figure 4.1.7 Effect of different N_{imp} on the hydrograph

Figure 4.1.8 Effect of different width on the hydrograph

Figure 4.1.9 Effect of different D_{imp} on the hydrograph

Figure 4.1.10 Sensitivity analyses

Effect of overland Slope

Figure 4.1.5 shows the resultant peak flow rate and total volume rise by 48% and 2% respectively as a result of an increase in overland slope from 0.003 to 0.01 m/m. In general, milder slopes produce less runoff volume and smaller peak flow. If the slope is mild, the velocity of overland flow will be low and there will be more time for water to infiltrate, therefore reducing the amount of flow volume reaching the water body.

Effect of impervious area

Urbanization usually results in significant increases in impervious surfaces, which results in increased peak flow and total volume. Impervious surfaces are also less rough than natural surfaces, and thus increase the runoff velocities and consequently surface erosion. As shown in Figure 4.1.6, an increase in impervious area from 10% to 30% results in an increase of 64% in peak flow and 49% increase in total volume. An increase in total runoff after a storm due to imperviousness results a decrease in groundwater recharge, and hence a decrease in low (base) flows. Thus, an increase in imperviousness has the effect of increasing peak flow during storm periods and decreasing low (base) flows between storms.

Effect of Overland width

Figure 4.1.8 Shows that an increase in overland width from 350m to 700m would increase the peak flow by 2%, and the total flow volume by 60%. This is because when the effective overland width is increased, the runoff flow travel time will be reduced, which results in a decrease in infiltration loss.

Effect of Depression Storage

In general, storage reduces and delays the time to peak and increases the duration of runoff. The total runoff volume may be reduced by the increasing effect of abstractions. D_{per} were tested over the ranges as suggested in the literature: 2.5 to 15mm for lawns to wooded areas (Nzewi, 2001; ASCE, 1992; Chow, 1964) respectively. Figure 4.1.9 shows that the peak flow is reduced by 1% when depression storage (pervious) increases from 5mm to 9.9mm. The total volume is decreased by 4%.

Figure 4.1.10 shows that an increase in depression storage from 5mm to 9.9mm

would decrease the peak flow by around 2% and the total volume by 4%.

Effect of Manning coefficients

N_{imp} , N_{per} were tested over the ranges suggested in the literature: 0.01 to 0.03 for impervious surfaces (Wanielista, 1997; McCuen *et al.*, 1996); and 0.3 to 0.8 for grass surfaces (Wanielista, 1997; McCuen *et al.*, 1996). Figure 4.1.7 shows that the peak flow is reduced by 50% when N (impervious) increases from 0.01 to 0.02. Total volume decreases by 1%.

The N (pervious) has insignificant influence on peak flow and total volume compared to others. For example, the peak flow and the total volume are reduced by approximately 1% when N_{per} is increased from 0.3 to 0.8. Roughness affects the velocity of overland flow and stream flow. A rough channel is likely to result in a smaller peak than a smooth channel.

Effect of f_o , f_c , k ,

The test range for f_o , f_c and k were set to in the range from 50 to 200mm/hr; 0.5 to 12mm/hr; 0.0003 to 0.0040 1/sec, respectively, which are applicable for low permeable soils (Rawls *et al.*, 1976; Bedient and Huber, 2002). f_o , f_c and k have an insignificant correlation with peak flow and total volume as shown in Figure 4.1.10. Figure 4.1.10 shows that f_o , f_c and k are found to be less significant for peak flow and total volume than other parameters.

Figure 4.1.11 Rainfall hyetograph and runoff hydrographs for 24, Dec 2005 for each study site

Figure 4.1.11 shows the rainfall hyetograph and 5 simulated hydrographs for a storm event in Kranji catchments. The watershed characteristic has a significant influence on runoff characteristic, as shown in Figure 4.1.11. The runoff volumes are generated from a rainfall with an assumed constant depth occurring uniformly over the watershed. Thus, the watershed area is the most important factor affecting the runoff characteristic. The sensitivity of the rainfall-runoff relationships to different parameters have been analyzed above. The key watershed characteristics include length, slope, land use, and Manning coefficients of the watershed. The watershed characteristics are different for each study site. The difference in the significance of the watershed characteristics in the five watersheds studied may be due to spatial variations in rainfall on the five watersheds, or differences in watershed hydrologic processes in each study site. The study reveals that uncertainties in rainfall distribution patterns could play an important role in flood flow predictions.

4.1.8 Evaluation Criteria

Outputs of the XP-SWMM modeling for each rainfall event were compared with the measured data. Model performance was evaluated based on the Nash-Sutcliffe coefficient, relative volume error and relative peak flow error. The Nash-Sutcliffe efficiency is defined as:

$$E_f = 1 - \frac{\sum_{i=1}^n (Q_0 - Q_m)^2}{\sum_{i=1}^n (Q_0 - \bar{Q})^2} \quad (4.20)$$

where E_f is the Nash-Sutcliffe efficiency; Q_0 is the observed direct runoff; Q_m is the modeled direct runoff; and $\bar{Q}$ is the mean observed flow rate over the entire experimental time. $E_f=1$ indicates a perfect fit, and negative E_f values indicate that the mean value of the observed time series could be a better predictor than the model (Nash and Sutcliffe, 1970). The relative volume or relative peak errors are calculated as:

$$R.E = \frac{X_m - X_p}{X_m} \times 100\% \quad (4.21)$$

where X_m and X_p refers to measured and predicted values, respectively of peak flow or runoff volume.

4.2 Load Estimation Method

In this study, the urban runoff is divided into wet-weather flow (WWF) and dry-weather flow (DWF) at each study site. WWF can be defined as the surface runoff produced by rainfall events. DWF is stream flow resulting from rainfall that infiltrates into the soil and eventually moves through the soil to the stream channel, such as baseflow or groundwater flow. Allen and Richard (2005) describe the antecedent dry weather period (ADWP) as an important influence on loading of WWF, which is defined as the time between the end of a rainfall event and beginning of another.

4.2.1 Dry-Weather Flow Load Calculation

The purpose of this study is to conduct dry weather sampling to characterize water quality and to identify areas contributing pollutants to the water body during dry weather period. Baseflow samples were collected from each catchment during dry weather period to calculate the average and median baseflow concentrations (Chua, et al.). Dry weather periods were defined by a minimum duration of 48 hours with no rainfall prior to sample collection. The average and median baseflow pollutant concentrations during baseflow periods for each gauging station are shown in Table 4.2.1. The ranges of baseflow pollutant concentrations are shown in Table 4.2.2. The pollutant concentrations of the baseflow samples were used to examine for any seasonal trends. The correlations between antecedent dry weather duration (ADWD) and DWF loads are shown in Appendix C. The results do not identify any linkage between water quality conditions and ADWD. The graphs show that the generally short antecedent dry period may be responsible for the high pollutant concentrations. The concentrations at CP2 and CP4 are sometimes extremely high when the ADW duration >24 hr. This could be caused by anthropogenic discharges during dry weather period. Furthermore, from a brief examination of the data collected during dry weather, it can be seen from Appendix C that there is insignificant seasonal trends in the dry-weather concentrations, as there is no apparent change during monsoon or non-monsoon seasons.

Table 4.2.1 Mean and median of baseflow pollutant concentrations at each study Site

Unit:mg/L

Site		NH₃-N	DOC	POC	TN	TDN	NO_x	DON	TP	TDP	DOP	OP	SiO₂	TSS
CP1	mean	0.17	4.00	0.56	1.20	1.20	0.46	0.56	0.08	0.04	0.02	0.02	6.18	29.82
	median	0.13	3.77	0.30	0.96	0.85	0.41	0.30	0.05	0.03	0.02	0.01	7.25	16.00
CP2	mean	0.47	3.82	1.08	2.50	1.64	0.92	0.25	0.14	0.09	0.03	0.06	5.96	16.67
	median	0.27	3.39	1.00	1.74	1.43	0.81	0.35	0.09	0.04	0.01	0.03	7.01	5.25
CP4	mean	0.34	5.09	0.99	1.99	0.78	0.33	0.11	0.26	0.04	0.00	0.04	5.06	13.44
	median	0.14	4.98	0.80	1.04	0.67	0.21	0.31	0.06	0.03	0.02	0.01	6.59	7.50
CP6	mean	0.21	6.96	0.55	1.96	1.73	1.06	0.46	0.17	0.13	0.02	0.11	6.15	7.46
	median	0.18	6.61	0.27	1.57	1.35	0.89	0.28	0.13	0.09	0.03	0.06	7.15	5.75
CP7	mean	0.09	2.22	0.25	0.79	0.66	0.66	0.06	0.04	0.03	0.02	0.02	8.92	6.20
	median	0.08	2.02	0.18	0.76	0.65	0.65	0.05	0.04	0.03	0.01	0.01	8.89	5.50

Table 4.2.2 Range of baseflow pollutant concentrations for each study site

Unit:mg/L

Pollutant	CP1	CP2	CP4	CP6	CP7
NH ₃ -N	0.118 ~ 0.190	0.196 ~ 0.515	0.129 ~ 0.431	0.118 ~ 0.273	0.054 ~ 0.140
DOC	3.794 ~ 4.955	3.349 ~ 5.079	4.776 ~ 6.316	5.379 ~ 8.815	1.957 ~ 2.630
POC	0.508 ~ 1.312	0.594 ~ 1.179	0.411 ~ 0.818	0.638 ~ 1.115	0.103 ~ 0.416
TN	0.773 ~ 1.182	1.389 ~ 2.587	0.768 ~ 1.256	1.517 ~ 2.308	0.164 ~ 0.992
TDN	0.504 ~ 0.859	0.990 ~ 1.662	0.583 ~ 0.845	1.025 ~ 1.864	0.104 ~ 1.107
NO _x	0.366 ~ 0.514	0.706 ~ 0.996	0.235 ~ 0.512	0.856 ~ 1.415	0.357 ~ 0.571
DON	0.307 ~ 0.896	0.300 ~ 0.840	0.276 ~ 0.555	0.272 ~ 0.868	0 ~ 0.141
TP	0.035 ~ 0.241	0.091 ~ 0.177	0.059 ~ 0.110	0.152 ~ 0.242	0.019 ~ 0.061
TDP	0.028 ~ 0.051	0.039 ~ 0.107	0.035 ~ 0.075	0.079 ~ 0.165	0.008 ~ 0.049
OP	0.016 ~ 0.233	0.024 ~ 0.107	0.017 ~ 0.062	0.071 ~ 0.506	0.005 ~ 0.016
DOP	0.015 ~ 0.033	0.016 ~ 0.031	0.017 ~ 0.040	0.020 ~ 0.051	0.001 ~ 0.036
SiO ₂	5.306 ~ 8.021	5.152 ~ 8.015	4.371 ~ 6.162	5.144 ~ 7.866	9.073 ~ 10.903
TSS	8.976 ~ 32.604	7.023 ~ 27.562	8.596 ~ 17.152	5.533 ~ 56.842	1.900 ~ 13.350

4.2.2 Wet-Weather Flow Load Calculation

Event Mean Concentration

Due to the difficulty in obtaining required amount of accurate data to generate pollutographs (concentration-time plots) or loadographs (mass-time plots), non-point source pollution is commonly represented by the event mean concentration (EMC). EMC is the ratio of the total pollutant mass to the total runoff volume of a storm event, expressed as:

$$EMC = \bar{C} = \frac{M}{V} = \frac{\int_0^{t_r} q(t)c(t)dt}{\int_0^{t_r} q(t)dt} \quad (4.22)$$

where EMC=event mean concentration (ML^{-3}); $\bar{C}$ =average concentration of contaminant (ML^{-3}); M =total mass transported throughout the duration of the event in M ; V =total volume of runoff (L^3); $q(t)$ =functional relationship expressing runoff as a function of time ($L^3 T^{-1}$); $c(t)$ = pollutant concentration as a function of time (ML^{-3}); and the limits of integration refer to time 0 (the initiation of runoff) and time t_r (the time at which runoff ceases) both in units of T . Where water quality observations are not available, regionalized values need to be used. These are often mean values based on observations collected within the region, and range of the EMC established by all storm events. Mean values are often established for different land use conditions. The site mean concentration (SMC) is used to represent a suitable measure of the central tendency of the EMC's for a specific site. The log-normal distribution is commonly employed (McLeod et al. 2006). It is important to know the SMC when estimating the annual pollutant loads.

4.2.3 Regression Analysis

The correlation of the nutrients and storm runoff is important for water quality management. Because the collection of water quality data records are often insufficient and the measurements are usually taken irregularly, this study aims to develop water quality rating curves, which are useful for prediction of loading rate/concentration from known storm flow rates. The resulting relationship is then used to estimate the nutrients concentration when the flow rate is known but nutrients data are lacking.

The parameters are plotted in log-log scale show that the correlations between concentration and the total flow. Figure 4.2.1 illustrates the log-log relationship between the concentrations (TP) with total flow rate. The log plots for TP, SiO₂, and TSS are shown in appendix D. Other parameters have poor correlations in log plot.

Figure 4.2.1 Total flow and TP concentration log-log graph at CP6

The analysis is performed on measured values of both flow rate and the corresponding loading rate, as shown in Figure 4.2.2. The relationship between total runoff Q and the loading rate L is given by:

$$L = C \times Q_T \quad (4.23)$$

where $Q_T > 0$, L is loading rate (g/s); C is nutrient concentration (mg/L) and Q_T is flow

rates (m^3/s). Once the curve is constructed, the nutrient concentrations can be calculated from the regression equation. The rating correlations for 13 parameters were elucidated, i.e. Ammonium ($\text{NH}_3\text{-N}$), Dissolved organic carbon (DOC), Particulate organic concentration (POC), Total nitrogen (TN), Total dissolved nitrogen (TDN), Nitrate+nitrite (NO_x), Dissolved organic nitrogen (DON), Total phosphate (TP), Total dissolved phosphate (TDP), Ortho-phosphate (OP), Dissolved organic particulate(DOP), Silica (SiO_2) and Total suspended solids (TSS). The ability of these models to predict each concentration loading was evaluated for the storm events that occurred in each study site.

Figure 4.2.2 Developed to rating curve of total flow against TSS loading rate at CP7

4.2.4 Annual Pollutant Loadings

Annual pollutant loading is an important indicator of potential effects on water quality. The estimates were generated by using rainfall data from the rain gauge operated by PUB (2007). In order to estimate the annual load from WWF conditions, direct runoff generated by XP-SWMM was combined with baseflow measured when no direct runoff occurs. Two approaches were used to estimate urban annual load for each parameter for period 2005-2007, by the regression equation and by the Simple Method. In the first approach, regression analysis was applied to determine a regression equation. In the second approach, the quality was described using a SMC.

Annual loading estimated by Simple Method is directly proportional to the annual precipitation, runoff coefficient, and the EMC. Annual pollutant loadings were calculated using the “Simple Method” defined by Schueler (1987). This method has been demonstrated to be accurate in the prediction of total load as complex model and larger catchment area (Chandler, 1994). The annual pollutant loading is defined as follows:

$$L = P \times CF \times R_v \times C \quad (4.24)$$

Where L = annual pollutant load (kg/ha-yr); P = annual precipitation (mm/yr); CF = correction factor that adjusts for storms where no runoff occur; R_v = average runoff coefficient; and C = SMC (mg/L).

Schueler (1987) determined that almost 90% of rainfall events in the Washington, DC areas generate runoff. The CF was applied to the complete rainfall and runoff data at CP1 for the whole year of 2006. It was found that runoff occurred during 98% of rainfall events. Since Singapore has a tropical climate and rainfalls are generally heavy, rainfall events without runoff occurring rarely. Furthermore, since CP1 has a complete whole year's rainfall data, the, 98% is applied to other CPs. Therefore, 0.98 was assumed for CF used in this study. The runoff coefficient can be determined by dividing the runoff volume by the rainfall volume (Wanielista and Yousef, 1993). The dimensionless average runoff coefficient, R_v , is an indication of the site response to rainfall events and is calculated as follows

$$R_v = \frac{R}{P} \quad (4.25)$$

Where R = storm runoff, P = precipitation.

Table 4.2.3 Range of runoff coefficients

Rational Method Runoff Coefficients

	Runoff Coefficients
Business Downtown Neighborhood	Type of Development 0.70 to 0.95 0.50 to 0.70
Residential Single family Multi-units (detached) Multi-units (attached)	 0.30 to 0.50 0.40 to 0.60 0.60 to 0.75
Residential (suburban)	0.25 to 0.40
Apartment	0.50 to 0.70
Industrial Light Heavy	 0.50 to 0.80 0.60 to 0.90
Park, Cemeteries	0.10 to 0.25
Playgrounds	0.20 to 0.35
Railroad Yard	0.20 to 0.35
Unimproved	0.10 to 0.30

Source: Design and Construction of Sanitary and Storm Sewers, American Society of Civil Engineers and the Water Pollution Control Federation, 1969.

The runoff coefficients are shown in Figure 4.2.3 as a function of total rainfall. The runoff coefficients of Table 4.2.3 reflect the effects of land use on runoff potential. The recommended range for the runoff coefficient is shown in Table 4.2.3. The relationship between rainfall and runoff for the observation period is shown in Figure 4.2.3 at each study site, which presents a strong correlation, with the R^2 value above 0.79. Antecedent condition and duration contribute insignificant effects in the rainfall runoff correlation. The average runoff coefficient was found to be 0.33 for CP1, 0.44 for CP2, 0.13 for CP4, 0.22 for CP6, and 0.26 for CP7. Comparing CP2 with CP4, the runoff coefficient is about 3 times higher in CP2, which has the largest developed area, around 68% of residential land use area with large impervious cover. In contrast, CP4 has the lowest runoff coefficient (= 0.13) as it has the largest previous area. Runoff coefficients tend to be higher values when the proportions of effective impervious surfaces increase (Jon et al. 2006).

Figure 4.2.3 Rainfall and runoff relationship

4.3 Description of First Flush

4.3.1 Normalized Mass and Volume Calculations

The ‘mass-based first flush (MBFF)’ has been defined by several investigators. At least three different measures have been widely utilized to describe the first flush phenomena: (1) $M(t) = V(t)$ (Helsel et al. 1979); (2) $M(t) = 0.8$ and $V(t) = 0.2$ (Saget et al., 1996; Bertrand et al., 1998); (3) $M(t) = 0.5$ and $V(t) = 0.25$ (Wanielista and Yousef 1993). The two dimensionless parameters $M(t)$ and $V(t)$ represent the normalized pollutant mass and stormwater volume, as follows:

$$V(t) = \frac{\int_0^t Q(t)dt}{\int_0^n Q(t)dt} \quad (4.26)$$

$$M(t) = \frac{\int_0^t Q(t)C(t)dt}{\int_0^n Q(t)C(t)dt} \quad (4.27)$$

where $V(t)$ is the cumulative runoff volume of the event from any time to t time intervals, normalized by dividing by the total runoff volume. Similarly, $M(t)$ is the cumulative pollutant mass from any time to t time intervals, normalized by dividing by the total pollutant mass. There are three methodologies widely use in the literature to quantify mass or concentration first flush, as shown in Fig 4.3.1. Eleven parameters (TSS, TP, NO_x , POC, DOC, TP, TDP, TDN, TN, SiO_2 , $\text{NH}_3\text{-N}$) were examined for first flush behavior during rainfall-runoff events based on the MBFF using method 1.

In general, the first flush phenomenon is characterized by a greater concentration or mass of pollutant delivery at the initial portion of a storm event, as indicated in the first measure (1). The second measure (2) has been defined as the first 20% of the total event runoff volume having 80% of the total mass loading. The third measure has been defined as the initial 25% of the total event runoff volume having 50% of the mass loading. In addition, a second flush can be determined by 50% of the total pollutant

mass being delivered in any 25% of runoff volume beyond the first portion of the storm volume. The first flush behavior was examined in relation to the effects of land use, rainfall depth and antecedent dry period.

Method 1

The first method compares the variation of cumulative $M(t)$ and $V(t)$ with the elapsed time of the storm graphically, by plotting $V(t)$ and $M(t)$ on the vertical axis and normalized time on the horizontal axis. Where the $M(t)$ plot resides above the V plot, it indicates Mass Based First Flush (MBFF) occurs for any period. (Sansalone and Buchberger 1997; Sansalone et al. 1998; Cristina and Sansalone 2003).

Method 2

The second method replaces the variable t which was plotted in the horizontal axis in Method I with $V(t)$. $M(t)$ is plotted on the vertical axis. In this method, a line L with a slope of 1:1 is drawn from the origin. An MBFF occurs for any period during which $M(t)$ exceeds $L(t)$ (Deletic 1998; Bertrand et al. 1998; Larsen et al. 1998).

Method 3

In this method, $M(t)$ is related to V through the following expression:

$$M(t) = [V(t)]^R$$

In this expression R =fitted exponential parameter. Values of $R < 1$ indicate the occurrence of an MBFF, i.e., a disproportionately high mass delivery of mass. Values of $R > 1$ indicate no MBFF (Saget et al. 1996; Bertrand et al. 1998).

Figure 4.3.1 Three methods used to calculate mass-based first flush during the 6 Jul 2006 event

Chapter 5

Results and Discussion

This research applied a deterministic rainfall-runoff model, the XP-SWMM, to the Kranji catchment for the time period 2005-2007. The model outputs were validated using field observations of direct runoff. The direct runoff data were simulated by XP-SWMM at 15 min interval, and the baseflow was modeled based on empirical equation derived from field measured data. Details of the separation procedure adopted for CP1 can be found in Lim et al. (2008). The simulations of direct runoff were based on the annual rainfall data at CP1 for year 2005-2006. The average rainfall depths are 2563.05 mm for year 2005 and 2723.29 mm for year 2006. The rainfall data for 2007 are based on data measured at each gauging station, 3114.4mm for CP1, 4707.2mm for CP2, 3859mm for CP4, 3869.2mm for CP6 and 3140mm for CP7, refer to Table 3.4.2. More details of the calibration procedure adopted can be found in Tan et al. (2008).

5.1 Calibration and Verification Results for CP1, CP2, CP4 and CP7

The total catchment areas of the gauging stations and land use data are shown in Table 3.3.2. The SWMM model was first calibrated against 9 storm events at CP1, ranging from 6.0 to 199.2 mm in total event rainfall depths; 12 events at CP2, ranging from 25.6 to 76.8 mm; 8 events at CP4, ranging from 27.4 to 126.4 mm; 9 events at CP6, ranging from 22.8 to 205.8 mm and 11 events at CP7, ranging from 23.4 to 79.8 mm. Each storm event had different values for total rainfall depth and antecedent dry period (ADWP). During the calibration of SWMM, an initial value for each parameter is assigned. The simulated direct runoff is compared with the measured direct runoff. The parameter values were systematically adjusted until the deviation or standard error between the simulated and observed runoff was minimized or reduced to a satisfactory level. The performance measures were the peak flow rate and total event runoff volume, and goodness of fit as measured by the Nash-Sutcliffe coefficients.

The optimized parameters for these events are summarized in Tables 5.1.1 to 5.1.5. For CP1 and CP2, the calibrated parameter sets were found to generate direct runoff

hydrographs close to the measurements with the Nash-Sutcliffe Efficiency E_f values ranging from 0.88 to 0.97 and 0.94 to 0.99, respectively, and a mean relative volume errors (EV) of 9.83% and -0.35%, respectively. The calibrated parameter sets for CP4, CP6 and CP7 were also able to predict the direct runoffs reliably, with E_f values ranging from 0.75 to 0.98, 0.75 to 0.96 and 0.82 to 0.96 respectively, and mean relative volume errors (EV) of about -2.54%, -11.91% and -2.13%. The average relative error for the observed and simulated values for peak flow is about -2.97%, 3.17%, -0.38%, 3.29% and -4.45% for CP1 to CP7 respectively.

The most sensitive parameters affecting the peak discharge rate were the Manning's N and W (width) (Bedient and Huber, 1988). The sensitivity analyses were shown in section 4.1.7. The smoother the overland flow roughness, the higher the peak flow, and more runoff volume may be expected (Liong et al., 1991). Tsihrintzis and Hamid, (2001) suggested that D_{imp} (depression storage) has significant effect on total volume of runoff and peak flow rate. However, in this study, D_{imp} was fixed, while calibration of flow volume was achieved mainly by adjusting ($Imp\%$, percent impervious). Thus the EV could be affected by ADWP which was not taken into consideration by SWMM model. Chen and Adams (2006) indicated that SWMM does not generate direct runoff volume when the rainfall event volume was too small. Runoff event will not occur if the rainfall volume is not sufficient to fill the D_{per} and D_{imp} demands. Therefore, increasing the depression storage of the pervious or impervious areas of the catchment will lead to an increased probability of no runoff events. Due to the lack of sufficient field data, the calibration of SWMM in this study has to rely on a limited number of measured runoff events. Considering the limitations in the calibration of SWMM, there is possible discrepancy between the predicted runoff event volumes and measured data.

It is difficult to obtain accurate values for W and $Imp\%$ for CP1 and CP7, as these two sub-catchments have larger watershed areas. For CP4 and CP6, their values of catchment width (W) and slope (S_o) are quite close. However their percent impervious ($Imp\%$) are relatively different, and vary within a range from 10-17% and 16-30% respectively. Most storm events at CP4 and CP6 appear to have multiple peaks, low runoff volumes and longer rainfall durations, which could affect EV and EP. Since CP4 and CP6 have larger undeveloped area, the impervious area depression storage becomes

difficult to estimate, which could significant affect low volume runoff event. However, there is only low flow data available at these stations. Tan et al. (2008), using events with a wide range of runoff peaks and volumes for calibration and verification; suggested that SWMM provides better results for events under ‘medium’ and ‘high’ flow regimes than those under ‘low’ flow regime. Moreover, SWMM is capable of simulating the hydrographs more accurately for simple storms with a single peak. It does not provide as accurate simulations for storms with multiple peaks. The simulation could be affected by the initial soil moisture content, which is difficult to measure, since there are large areas of undeveloped regions. In addition, information on human influences was not available in this study.

The arithmetically averaged calibrated parametric values were applied to the storm events used in the calibration. The averaged parameter set was observed to be capable of regenerating the direct runoff hydrographs close to measured hydrographs. Although there is a reduction in the prediction accuracy for some events, the averaged parameter set (herein refer to as the “Mean Value”) is still able to produce reliable hydrographs compared to hydrographs predicted using the individual calibrated parameter sets (herein refer to as the “Event”). As shown in Tables 5.1.1 to 5.1.5, the averaged parameter sets for CP1 to CP7 were also capable of predicting the direct runoffs reliably, with mean E_f values above 0.75, and a mean relative volume error (EV) about 7.08%, 12.18%, -3.36%, -6.70% and -2.35% for CP1 to CP7 respectively.

Table 5.1.1 Results of simulation and observation for CP1

Parameters		15-Apr-06	16-Apr-05	6-May-05	16-May-05	25-May-05	21-May-05	5-Oct-05	16-Oct-05	24-Dec-05	Average
Antecedent dry period (h:m)		24:15	22:55	18:25	23:40	13:55	43:10	21:50	28:55	22:20	
Rainfall depth (mm)		62.7	199.2	33.0	36.1	6.0	72.6	11.9	6.8	73.4	55.8
Simulation	<i>W</i> (m)	2500	2200	2200	2200	2000	2000	2000	2200	2000	2144.44
	<i>Imp</i> (%)	35	27	24	33	24	20	26	33	27.5	27.72
	<i>S_o</i> (m/m)	0.016	0.015	0.015	0.015	0.01	0.01	0.008	0.015	0.01	0.013
	<i>N_{imp}</i>	0.01	0.01	0.01	0.01	0.01	0.01	0.013	0.01	0.011	0.010
	Calibration	0.97	0.91	0.96	0.88	0.91	0.94	0.93	0.92	0.96	0.93
Nash coefficient, Ef	Recalibration	0.90	0.93	0.63	0.88	0.79	0.92	0.70	0.92	0.88	0.84
Direct runoff volume (x1000m ³)	Observation	120.46	59.76	37.54	53.96	41.52	120.04	58.38	122.75	88.88	78.14
	Calibration	109.22	63.82	38.90	62.40	48.56	117.46	66.65	163.40	99.12	85.50
	Recalibration	91.39	62.87	43.27	52.48	56.10	109.40	73.66	137.37	101.54	80.90
Relative volume error, EV (%)	Calibration	-10.29	4.95	3.50	13.52	14.49	14.75	12.40	24.88	10.33	9.83
	Recalibration	-31.80	6.36	13.24	-2.82	25.98	8.86	20.74	10.64	12.47	7.08
Peak Flow (m ³ /s)	Observation	43.66	21.57	13.63	19.64	12.71	42.08	12.25	34.36	33.44	25.93
	Calibration	41.70	21.82	14.10	19.82	13.90	40.42	12.69	29.39	32.57	25.16
	Recalibration	32.64	20.59	14.29	16.69	16.06	39.17	13.37	24.80	34.23	23.54
Error on Peak Flow, EP (%)	Calibration	-4.49	1.16	3.44	0.95	9.35	-3.95	3.52	-14.45	-2.61	-2.97
	Recalibration	-25.25	-4.52	4.82	-15.01	26.34	-6.93	9.10	-27.80	2.33	-9.22

Table 5.1.2 Results of simulation and observation for CP2

Parameters		2-May-07	3-Nov-06	4-Jun-07	10-Nov-06	16-Apr-07	19-Jan-07	22-Apr-07	23-Apr-07	16-Aug-07	18-Aug-07	16-Jul-06	24-Aug-07	Average
Antecedent dry period (h:m)		21:55	2:45	67:20	19:15	21:40	18:15	20:50	23:25	27:15	19:55	237:10	8:40	
Rainfall depth (mm)		56.6	51.4	25.6	58.7	31.2	52.2	41.2	76.8	39.4	48.4	43	54.2	48.2
	<i>W</i> (m)	2000	1500	1600	1500	1800	1800	1400	1500	1600	1500	1200	1300	1518.2
	<i>Imp</i> (%)	43	43	40	30	45	35	27	51	25	45	27	40	37.09
	<i>S_o</i> (m/m)	0.04	0.025	0.04	0.03	0.035	0.025	0.022	0.03	0.03	0.03	0.024	0.025	0.029
Simulation	<i>N_{imp}</i>	0.008	0.012	0.008	0.013	0.008	0.013	0.011	0.008	0.012	0.012	0.014	0.014	0.011
Nash coefficient, Ef	Calibration	0.94	0.98	0.98	0.99	0.98	0.98	0.97	0.97	0.95	0.99	0.97	0.97	0.975
	Recalibration	0.79	0.92	0.86	0.88	0.84	0.88	0.72	0.84	0.81	0.92	0.62	0.88	0.833
	Observation	47.77	90.28	18.60	32.46	25.62	29.28	20.62	83.47	17.97	42.21	21.36	39.24	38.28
Direct runoff volume (x1000m ³)	Calibration	47.05	90.50	20.52	34.92	27.52	31.51	19.35	79.07	19.23	41.43	22.87	41.37	38.94
	Recalibration	40.22	79.13	18.52	42.34	22.12	37.40	29.67	57.50	28.63	34.48	30.35	38.91	38.09
Relative volume error, EV (%)	Calibration	-1.54	0.25	9.33	7.06	6.89	7.09	-6.56	-5.57	6.53	-1.87	6.62	5.15	3.17
	Recalibration	-18.76	-14.09	-0.47	23.34	-15.85	21.73	30.51	45.18	37.23	-22.43	29.62	-0.84	12.18
	Observation	18.50	31.83	10.40	17.81	17.81	17.53	16.98	40.44	9.82	20.19	13.49	13.13	19.04
	Calibration	14.64	32.04	10.33	17.31	17.31	18.06	17.76	38.93	8.97	19.23	13.64	14.44	18.91
Peak Flow (m ³ /s)	Recalibration	11.40	35.21	8.38	21.01	11.29	24.08	22.72	28.28	12.26	18.32	23.18	15.07	19.98
Error on Peak Flow, EP (%)	Calibration	-20.87	0.66	-0.63	-2.78	-2.78	3.04	4.59	-3.73	-8.61	-4.72	1.12	9.98	-0.35
	Recalibration	-38.37	10.65	-19.46	17.99	-36.59	37.38	33.80	-30.06	24.90	-9.23	71.78	14.79	10.54

Table 5.1.3 Results of simulation and observation for CP4

Parameters		17-Aug-07	18-Aug-07	22-Apr-07	23-Apr-07	25-Jun-07	26-Apr-07	27-Jul-07	28-Aug-07	Average
Antecedent dry period (h:m)		18:25	20:00	22:20	23:35	125:20	9:25	22:05	47:35	
Rainfall depth (mm)		94.4	43.2	44.4	52.4	126.4	81.8	27.4	56.8	65.8
Simulation	<i>W</i> (m)	1500	1500	1500	1500	1500	1500	1500	1500	1500.0
	<i>Imp</i> (%)	12	13	11	14	10	17	10	14	12.6
	<i>S_o</i> (m/m)	0.0035	0.0035	0.0035	0.0035	0.0035	0.0040	0.0040	0.0040	0.004
	<i>N_{imp}</i>	0.015	0.013	0.013	0.013	0.015	0.011	0.011	0.012	0.013
	Calibration	0.94	0.90	0.89	0.95	0.75	0.91	0.90	0.98	0.86
Nash coefficient, Ef	Recalibration	0.87	0.79	0.85	0.90	0.62	0.67	0.68	0.96	0.75
Direct runoff volume (x1000m ³)	Observation	37.23	15.61	11.34	20.52	37.77	39.86	6.69	20.21	23.65
	Calibration	31.51	15.96	12.91	19.39	40.74	38.30	6.15	19.90	23.11
	Recalibration	34.30	13.79	14.38	17.39	48.57	28.60	8.27	18.33	22.95
Relative volume error, EV (%)	Calibration	10.39	6.56	12.15	-4.82	5.68	-2.62	-2.03	-1.29	-2.54
	Recalibration	21.58	-13.22	21.13	-17.98	22.23	-39.38	19.11	-10.24	-3.36
Peak Flow (m ³ /s)	Observation	8.28	7.18	7.49	5.67	7.01	8.10	4.35	13.13	7.65
	Calibration	9.03	6.07	7.03	6.52	8.87	6.49	3.39	14.44	7.73
	Recalibration	9.82	5.50	7.33	5.91	10.75	4.91	2.99	15.07	7.78
Peak Flow relative error, EP (%)	Calibration	9.01	-15.35	-6.20	14.99	26.54	-19.95	-22.02	9.98	-0.38
	Recalibration	18.53	-23.39	-2.10	4.24	53.29	-39.39	-31.19	14.79	-0.65

Table 5.1.4 Results of simulation and observation for CP6

Parameters		1-Jun-07	8-Jul-07	11-May-07	16-Aug-07	17-Aug-07	18-Aug-07	25-Apr-07	26-Apr-07	25-Jun-07	Average
Antecedent dry period (h:m)		18:40	35:00	22:05	0:30	2:20	15:15	26:10	9:05	138:05	
Rainfall depth (mm)		22.8	39.2	27.4	205.8	50.8	50.4	51.8	60.2	95.2	67.1
Simulation	<i>W</i> (m)	700	700	700	700	700	700	700	700	700	700.0
	<i>Imp</i> (%)	19	17	16	22	25	19	19	30	21	20.89
	<i>S_o</i> (m/m)	0.011	0.011	0.0100	0.011	0.011	0.01	0.009	0.011	0.01	0.010
	<i>N_{imp}</i>	0.009	0.012	0.014	0.007	0.008	0.011	0.011	0.008	0.013	0.0103
	Calibration	0.87	0.96	0.91	0.75	0.81	0.92	0.90	0.70	0.84	0.85
Nash coefficient, Ef	Recalibration	0.85	0.88	0.57	0.75	0.72	0.92	0.89	0.58	0.81	0.77
Direct runoff volume (x1000m ³)	Observation	6.96	9.23	5.02	81.33	19.83	14.85	16.83	31.70	32.41	24.24
	Calibration	5.51	9.14	5.90	73.20	18.02	12.27	14.13	24.97	30.20	21.48
	Recalibration	6.26	11.30	7.73	69.09	15.10	13.46	15.52	24.82	30.33	21.51
Relative volume error, EV (%)	Calibration	-26.20	-0.92	14.78	-10.35	-10.05	-21.05	-19.12	-27.00	-7.30	-11.91
	Recalibration	-11.14	18.30	35.04	-17.72	-31.35	-10.38	-8.42	-27.72	-6.87	-6.70
Peak Flow (m ³ /s)	Observation	2.82	3.47	1.29	12.86	3.69	5.36	4.01	3.98	5.18	4.74
	Calibration	2.55	3.78	1.35	11.68	3.84	5.67	4.08	4.07	6.27	4.81
	Recalibration	2.39	4.64	1.79	11.68	3.19	5.67	4.52	4.25	6.21	4.93
Error on Peak Flow, EP (%)	Calibration	-9.69	8.91	4.25	-9.19	4.18	5.80	1.83	2.47	21.07	3.29
	Recalibration	-15.17	33.62	38.28	-9.19	-13.42	5.80	12.93	6.98	19.79	8.85

Table 5.1.5 Results of simulation and observation for CP7

Parameters		17-Aug-07	18-Aug-07	24-Aug-07	31-May-07	23-Apr-07	30-Apr-07	17-May-07	4-Apr-07	2-May-07	9-Aug-07	16-Apr-07	Average
Antecedent dry period (h:m)		20:00	20:45	12:15	48:40	23:35	17:55	54:40	36:55	20:25	186:35	21:40	
Rainfall depth (mm)		79.8	44.6	25.8	16.2	73.6	73.6	32.0	23.4	58.0	35.6	42.6	45.9
Simulation	<i>W</i> (m)	2400	2000	2400	2400	3000	2800	3000	2400	2800	2400	2800	2581.8
	<i>Imp</i> (%)	16	23	22	21	30	23	30	19	22	15	22	22.00
	<i>S_o</i> (m/m)	0.015	0.015	0.020	0.015	0.002	0.020	0.022	0.020	0.020	0.020	0.020	0.019
	<i>N_{imp}</i>	0.01	0.0105	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.01	0.010
	Calibration	0.96	0.94	0.95	0.89	0.95	0.94	0.96	0.82	0.93	0.89	0.94	0.92
Nash coefficient, Ef	Recalibration	0.81	0.92	0.95	0.86	0.81	0.92	0.86	0.63	0.81	0.58	0.92	0.82
Direct runoff volume (x1000m ³)	Observation	213.55	153.59	82.70	43.67	181.95	128.74	148.16	52.66	129.15	82.16	138.57	123.17
	Calibration	169.18	125.47	68.63	36.55	133.13	103.51	117.00	47.47	106.39	65.63	118.78	99.25
	Recalibration	278.56	148.29	82.79	48.29	120.24	118.59	103.65	65.63	127.90	115.38	138.52	122.53
Relative volume error, EV (%)	Calibration	-5.19	-2.04	-0.61	0.45	-13.88	-3.16	-5.53	7.55	-0.15	-3.80	2.92	-2.13
	Recalibration	23.34	-3.57	0.11	9.55	-51.32	-8.56	-42.95	19.76	-0.97	28.79	-0.04	-2.35
Peak Flow (m ³ /s)	Observation	46.66	40.68	15.90	8.07	50.89	25.12	31.18	11.30	21.14	19.81	43.04	28.53
	Calibration	50.66	38.49	14.52	6.00	43.60	25.81	27.02	11.58	20.04	22.34	41.74	27.44
	Recalibration	64.15	38.75	14.95	7.21	30.44	24.20	21.15	12.22	18.60	26.24	37.08	26.82
	Calibration	8.58	-5.38	-8.69	-25.63	-14.34	2.76	-13.33	2.52	-5.24	12.79	-3.03	-4.45
Peak Flow relative error, EP (%)	Recalibration	37.49	-4.73	-6.00	-10.70	-40.19	-3.63	-32.16	8.16	-12.03	32.50	-13.85	-4.10

Since the purpose of the model is to predict the longer term flows from the catchments, model verification was carried out based on the ability of the model to predict continuous flows for the period under study. To do this, the model was used to generate the monthly flows of the five catchments over the period (May, Aug, Dec, 2005) for CP1, (Feb to Jun, Aug, Nov, 2007) for CP2, (Nov, 2007) for CP4, (Apr and May, 2007) for CP7, for the monthly flows can be found in Figure A.6 to A.10. The analysis shows that the model is able to predict the direct runoff reasonably, with E_f values above 0.68, and the EV of about -11.68%, 0.89%, -1.04%, 18.19%, and 6.34% for CP1, CP2, CP4, CP6 and CP7 respectively. Tan et al. (2008) found that EV value for continuous-event calibration at CP1 is around 20%. In general, relatively large values of EV (more than 30%) for some events could arise from uneven spatial rainfall distribution over the watershed, and localized storm cells moving from one place to another (Brath et al., 2004). Tan (1995) found that tropical rainfalls are localized and randomly distributed even within a short distance of 600 m. Other reason could be errors in defining the sub-catchment boundaries. This indicates that the arithmetically averaged parameter set is reliable for representing the average physical characteristics of the CP1, CP2, CP4, CP6 and CP7 sub-catchments. The averaged calibration parameters are shown in Table 5.1.6. The simulated hydrographs for the calibration events can be found in Appendix A. The runoff volumes and peak flow error for each storm events given in Table 5.1.1 to 5.1.5 are plotted in Figure 5.1.1 and 5.1.2. The R-squared values of the regression (R^2) in Figure 5.1.1 and 5.1.2 are estimated at above 0.8 for each study sites, which shows the simulated volumes are reliable. In this case the index indicates a good agreement between simulated and observed results in each study site using the averaged calibrated parameter set.

Figure 5.1.1 Comparison of the measured and simulated direct runoff depth for each study area

Figure 5.1.2 Comparison of the measured and simulated peak flow for each study area

5.1.1 Long-Term Runoff Simulation

XP-SWMM has been widely employed for continuous simulation in urban drainage system planning and design. While system design based on long term continuous simulation is deemed reliable, the runoff generation needs to be calibrated for several sensitive parameters. The more sensitive parameters include characteristic width, characteristic slope and percentage of impervious areas and N_{imp} . In addition, long-term rainfall and evaporation data collected for the catchment are required. However, there are no existing rain gauges set up in many of the ungauged sub-catchments, hence rainfall data from the nearest rain gauges are assumed to apply to the un-gauged catchments.

Table 5.1.6 Summary of calibration results

Sub-catchment	Area (ha)	W (m)	Imp (%)	S_o (m/m)	N_{imp}
Bricklands (CP1)	552	2,144.44	27.72	0.0127	0.0104
CCKAVE4 (CP2)	200	1,518.18	37.09	0.0287	0.0114
TG AIRBASE (CP4)	288	1,500.00	12.63	0.0037	0.0129
AMK (CP6)	145	700.00	20.89	0.0104	0.0103
Sg Pangsua (CP7)	1560	2,581.82	22.05	0.0188	0.0100

The verification results show that XP-SWMM provides satisfactory predictions for the runoff volume and peak flow for storm events and continuous event. Therefore the model parameters in Table 5.1.6 were used to generate the yearly (2005-2007) direct runoff at each study site. The results are illustrated in Appendix B.

Table 5.1.7 Runoff ratios for major inflows year 2005 to 2007

Sub-Catchment	Year	DR/P	BF/P	TR/P	Annual Total Runoff Volume
					($\times 10^6 \text{ m}^3$)
CP1	2005	0.23	0.38	0.61	8.2
	2006	0.28	0.38	0.66	9.4
	2007	0.27	0.35	0.62	10.2
Average		0.26	0.37	0.63	9.27
CP2	2005	0.31	0.22	0.54	2.7
	2006	0.37	0.23	0.6	3.3
	2007	0.39	0.18	0.57	5.4
Average		0.36	0.21	0.57	3.8
CP4	2005	0.16	0.37	0.59	4.2
	2006	0.12	0.36	0.48	3.8
	2007	0.12	0.27	0.39	4.4
Average		0.13	0.33	0.49	4.13
CP6	2005	0.22	0.37	0.59	2.2
	2006	0.23	0.35	0.58	2.3
	2007	0.23	0.26	0.5	2.8
Average		0.23	0.33	0.56	2.43
CP7	2005	0.19	0.47	0.66	26.5
	2006	0.2	0.45	0.65	27.4
	2007	0.21	0.4	0.61	29.8
Average		0.2	0.44	0.64	27.9

Note: DR: Direct Runoff (mm); BF: Baseflow (mm); TR: Total Runoff (mm); P: Precipitation (mm)

The direct runoff/rainfall, baseflow/rainfall and total runoff/rainfall ratios based on simulations at the different gauging stations are summarized into Tables 5.1.8 for year 2005 to year 2007. The annual hydrological balance at each sub-catchment has different runoff ratio, as shown in Figure 5.1.3. The results reveal that the direct runoff ratio tends to increase with proportion of impervious area, as shown in Table 5.1.7. The direct runoff ratio is 0.12~0.16 at CP4, and 0.31~0.39 at CP2. The direct runoff ratios reflect the effects of land use on direct runoff generation.

2007 Hydrological Balance

Figure 5.1.3 Hydrological balances (2007) for the each sub-catchment
(Hr: total amount of rainfall; Hf: total amount of flow, in mm)

Table 5.1.8 Dry-weather flow and wet-weather flow volumes 2005-2007

Sub-Catchment	Contribution to Annual Total Runoff Volume							
	2005		2006		2007		Average	
	WWF	DWF	WWF	DWF	WWF	DWF	WWF	DWF
CP1	59.5	40.5	64.6	35.4	67.3	32.7	64	36
CP2	70.7	29.3	74.7	25.3	84.1	15.9	77	24
CP4	40.4	59.6	37.5	62.5	43.4	56.6	40	60
CP6	42.4	57.6	45.8	54.2	56.5	43.5	48	52
CP7	38.5	61.5	41.9	58.1	46.4	53.6	42	58

Note: WWF: Wet-weather flow (%); DWF: Dry-Weather flow (%)

Figure 5.1.4 Dry-weather and wet-weather flow volumes (2007)

Comparing the DR/R and BF/R ratios against the TR/R ratios, the relative contributions of WWF and DWF to the total runoff at each sub-catchment are estimated in Table 5.1.8. Liu (2006) applied GIS system-based distributed model to predict storm runoff from different land uses and showed that the direct runoff from urban areas is more dominant during flood events compared with runoff from other land uses. The direct runoff generated by XP-SWMM shows that for the present study, urbanization tends to increase the direct runoff volume and decrease the baseflow volume. It can be seen in Table 5.1.8 that DWF contributed about 60% of the annual total runoff volume in CP4 sub-catchments in year 2005-2007, which contains largest proportion of undeveloped and pervious areas.

Among the different sub-catchments, the smallest contribution of DWF to total runoff of 24% is from CP2, which has the largest high density residential land use areas. The contribution of DWF to total runoff at CP1, which has a lower proportion of high density residential areas, is about 36%. McPherson et al. (2005) examined the urban stormwater drainage systems in Southern California, and indicated that approximately 9%-25% of the total annual flow volume is DWF. The urbanized watersheds generate a larger volume of the received rainfall as direct runoff and less as ground water flow, resulting in lower baseflows in urbanizing watersheds (Meyer, 2002). High baseflow in the CP4 was likely due to the high infiltration capacities of reserve sites, which may lead to baseflow discharge. Additionally, the lack of forest cover also reduces

transpiration losses at KC02, which results in lower BF/R (Whitehead and Robinson, 1993).

Figure 5.1.4 compares the DWF and WWF volume for year 2007. The results show higher DWF than WWF in CP4 and CP7. From Singapore's street directory, there are several ponds within CP7, which could have affected the DWF. From these results, it can be seen that impervious areas and existence of abstraction ponds significant influence on runoff generation in a watershed, due to the generation of direct runoff during small storm events.

The results from XP-SWMM simulations show that the model is capable of providing good results for continuous flow simulations, and is also highly efficient in the estimation of urban storm water runoff volumes.

5.2 Impact of Land Use on Runoff-Loading Rates

5.2.1 Analysis of Event Mean Concentrations

Event Mean Concentration (EMC) values were calculated for the gauging stations CP1, CP2, CP4, CP6, and CP7 using Eq. (4.22). The average concentrations from different storm events expressed in terms of EMCs can provide representative patterns that can be used for long-term predictions and also for predicting annual pollutant loads.

The EMCs results are presented in Table 5.2.1, with the ranges of 95% confidence intervals. Overall, TSS exhibited the highest concentration among all the pollutants. A wide range of TSS concentrations has been noted, from 24.86~349.93 for CP1, 35.69~102.83 for CP2, 31.97~103.13 for CP4, 133.76~469.13 for CP6 and 13.60~108.85 for CP7. Many of the events in CP6 have TSS values of over 100 mg/L, which are considered high and can result in serious water quality problems, because TSS increases the turbidity of a stream, which can be harmful to aquatic life, and increases the cost of purifying water for public water supplies. Allen and Richard (2005) indicated that nitrogen and phosphorus are major pollutants contributing to eutrophication problems in many water bodies that should also be of concern.

At CP6, the concentration of $\text{NH}_3\text{-N}$, DOC, TN, TDN, NO_x , TDP, OP, and TSS were observed to have significantly higher values (up to 10 times of other gauging stations). This could be contributed by the wash-offs of organic wastes or excessive fertilizers from agricultural land use. The cemetery land use in CP4 might have contributed more phosphorus than domestic land use. Elsewhere, EMC values for all the contaminants were observed to be quite close for CP1, CP2, and CP7. These could be due to the similarities in their land uses. Agricultural land use at CP6 could increase nutrients loads in stream water from fertilizers applications. Although CP4 has the largest proportional undeveloped area, TP, TDP, DOP concentrations are high, with concentration values higher in CP4 than CP1, CP2 and CP7.

Table 5.2.1 Event Mean Concentration (EMC) for each study site**Unit: mg/L**

Pollutant		CP1	CP2	CP4	CP6	CP7
NH ₃ -N	Median	0.132	0.142	0.060	0.160	0.054
	Range	0.069~0.304	0.024~0.337	0.033~0.104	0.031~0.420	0.034~0.187
DOC	Median	3.660	3.097	3.228	5.270	5.604
	Range	2.660~7.875	1.859~4.723	3.228~5.507	4.288~8.1884	2.894~6.131
POC	Median	0.374	0.319	0.393	0.240	0.695
	Range	0.229~2.369	0.222~0.634	0~1.186	0~1.071	0.057~0.603
TN	Median	0.668	1.250	0.940	3.664	1.600
	Range	0.287~1.903	0.186~1.722	0.301~1.216	2.3425~7.298	0.789~2.177
NO _x	Median	0.397	0.574	0.177	0.774	0.471
	Range	0.188~0.518	0.046~1.243	0.017~0.387	0.032~5.660	0.266~1.060
DON	Median	0.382	0.102	0.197	1.452	0.080
	Range	0.352~0.556	0.054~0.219	0.099~0.175	0.093~0.283	0.023~0.395
TP	Median	0.087	0.076	0.097	0.993	0.072
	Range	0.036~0.206	0.038~0.140	0.054~0.437	0.364~1.201	0.034~0.182
TDP	Median	0.031	0.034	0.010	0.439	0.009
	Range	0.007~0.054	0.007~0.052	0.008~0.405	0.055~0.569	0.007~0.012
DOP	Median	0.020	0.020	0.012	0.049	0.009
	Range	0.009~0.037	0.006~0.038	0~0.254	0~0.217	0.004~0.016
OP	Median	0.022	0.005	0.005	0.321	0.004
	Range	0.006~0.027	0.001~0.013	0.002~0.043	0.040~0.487	0.0036~0.043
SiO ₂	Median	5.587	2.330	3.624	3.262	5.250
	Range	3.837~7.1226	1.310~4.117	2.453~4.417	2.175~4.484	3.990~7.66
TSS	Median	100.926	57.429	82.204	226.218	75.754
	Range	24.86~349.93	35.69~102.83	31.97~103.13	133.76~469.13	13.60~108.85

5.2.2 Analysis based on Rating Curve

The correlations between pollutant concentrations/loading rates and storm flow rates were investigated for 13 pollutants. The development of water quality rating curves is useful for prediction of loading rate/concentration from known storm flow rates.

Table 5.2.2 The correlation between pollutant concentration and storm flow rate

Study Site	Equation	R ²
CP1	TP(mg/L)= 0.0788(Q _T) ^{0.0982}	0.0449
CP2	TP(mg/L)= 0.0666(Q _T) ^{0.1041}	0.1345
CP4	TP(mg/L)= 0.135(Q _T) ^{0.3163}	0.3348
CP6	TP(mg/L)= 0.7344(Q _T) ^{0.3356}	0.6568
CP7	TP(mg/L)= 0.0431(Q _T) ^{0.2652}	0.1322
CP1	SiO ₂ (mg/L)= 5.3862(Q _T) ^{-0.0286}	0.0037
CP2	SiO ₂ (mg/L)= 3.7276(Q _T) ^{-0.2964}	0.6864
CP4	SiO ₂ (mg/L)= 4.038(Q _T) ^{-0.0961}	0.1552
CP6	SiO ₂ (mg/L)= 3.2748(Q _T) ^{-0.1699}	0.6255
CP7	SiO ₂ (mg/L)= 7.3906(Q _T) ^{-0.3072}	0.6066
CP1	TSS(mg/L)= 46.275(Q _T) ^{0.3522}	0.1267
CP2	TSS(mg/L)= 34.347(Q _T) ^{0.4543}	0.5694
CP4	TSS(mg/L)= 53.543(Q _T) ^{0.1627}	0.1997
CP6	TSS(mg/L)= 200.16(Q _T) ^{0.4809}	0.5662
CP7	TSS(mg/L)= 21.153(Q _T) ^{0.5196}	0.4546

Note: Q_T: Total flow (m³/s)

The log plots for TP, SiO₂ and TSS show a good agreement between total flow rate and concentration. The plots, as shown in appendix D, are found to have a linear positive trend for TP and TSS concentrations with flow rate, as shown in Figure D.6 and D.7. However, SiO₂ decreases with increasing flow rate, as shown in Figure D.8. From all the plots for storm flow, CP6 is well correlated for each nutrient. A set of predictive equations (in terms of power equations) is summarized in Table 5.2.2.

The water quality rating curves developed between pollutant loading rates and storm flow rates are summarized in Tables 5.2.3 to 5.2.7, together with their respective R² values. From graphs plotted for flow rates and loading rates, it is found that TSS has a better correlation with flow rate. The instantaneous storm flow rates were found to

correlate well with the loading rates for $\text{NH}_3\text{-N}$, DOC, POC, TN, NO_x , TDP, OP, and TSS, with R^2 values higher than 0.80 in CP1. TN, TDN, NO_x , DON, TP, TDP, OP, DOP, SiO_2 and TSS in CP2 show R^2 values higher than 0.80. Most parameters in CP4 shown good fit with R^2 values higher than 0.80, except for POC which has R^2 values of about 0.60. For CP6, correlations were observed between storm flow rates and loading rates for $\text{NH}_3\text{-N}$, DOC, TN, TDN and NO_x , with R^2 lower than 0.8. The total runoff rate appears to have weak correlation with loading rates for $\text{NH}_3\text{-N}$, DOC, POC, DON, TP, TDP, OP in CP7. Among the different forms of correlation equations investigated, the quadratic equation was generally observed to provide a better fit. The water quality rating curves developed can be found in Appendix D.

In summary, the pollutant loading rate appears to be a function of total runoff rate for most of the pollutants. Rating curves with R^2 values higher than 0.70 could therefore be applied to predict loading rates from total runoff rate, assuming $R^2 > 0.70$ is an indicator value for reliable correlations. For those pollutants exhibiting loading rate – total runoff rate relationships with R^2 lower than 0.70, the pollutant concentrations are assumed to be less dependent on total runoff rate and EMC values are recommended to represent the pollutant concentrations during storm flows.

Table 5.2.3 Summary of water quality rating curves for storm flows (CP1)

Pollutant	Equation	Application Range (m ³ /s)	R ²
NH ₃ -N	$\text{NH}_3\text{-N (g/s)} = 0.0008(Q_T)^2 + 0.073(Q_T) + 0.0209$	all	0.8076
DOC	DOC (mg/L) = 15.89 DOC (g/s) = 3.9498(Q _T) + 0.7048	Q _T < 0.06 Q _T ≥ 0.06	0.8398
POC	POC (mg/L) = 0.59 POC (g/s) = 0.0398(Q _T) ² + 0.4004(Q _T) + 0.4744 POC (mg/L) = 2.149	Q _T < 2.38 2.38 ≤ Q _T ≤ 43.66 Q _T > 43.66	0.8095
TN	TN (mg/L) = 0.697 TN (g/s) = 0.5662(Q _T) + 0.3602	Q _T < 2.67 Q _T ≥ 2.67	0.8344
TDN	TDN (mg/L) = 0.822 TDN (g/s) = -0.0028(Q _T) ² + 0.3727(Q _T) + 0.374 TDN (mg/L) = 0.259	Q _T < 0.83 0.83 ≤ Q _T ≤ 43.66 Q _T > 43.66	0.7175
NO _x	NO _x (mg/L) = 0.403 NO _x (g/s) = 0.3973(Q _T) + 0.0563	Q _T < 10.19 Q _T ≥ 10.19	0.8326
DON	DON (mg/L) = 0.607 DON (g/s) = 0.4039(Q _T) + 0.0189	Q _T < 0.10 Q _T ≥ 0.10	0.7364
TP	TP (g/s) = 0.0012(Q _T) ² + 0.1025(Q _T) + 0.0042 TP (mg/L) = 0.155	Q _T ≤ 43.66 Q _T > 43.66	0.73
TDP	TDP (mg/L) = 0.733 TDP (g/s) = 0.0018(Q _T) ² + 0.0341(Q _T) + 0.0132 TDP (mg/L) = 0.113	Q _T < 0.03 0.03 ≤ Q _T ≤ 43.66 Q _T > 43.66	0.8579
OP	OP (mg/L) = 0.825 OP (g/s) = 0.003(Q _T) ² + 0.0023(Q _T) + 0.0162 OP (mg/L) = 0.825	Q _T < 0.02 0.02 ≤ Q _T ≤ 273 Q _T > 273	0.8069
DOP	DOP (g/s) = 0.0014(Q _T) ² + 0.0151(Q _T) + 0.0155	all	0.6231
SiO ₂	SiO ₂ (mg/L) = 7.241 SiO ₂ (g/s) = 3.3819(Q _T) + 3.9212 SiO ₂ (mg/L) = 3.472	Q _T < 1.02 1.02 ≤ Q _T ≤ 43.66 Q _T > 43.66	0.7865
TSS	TSS (mg/L) = 534.5 TSS (g/s) = 10.574(Q _T) ² + 63.208(Q _T) + 0.7255 TSS (mg/L) = 534.5	Q _T < 0.0023 0.0023 ≤ Q _T ≤ 44.5152 Q _T > 44.5152	0.886

Note: Q_T: Total flow (m³/s)**Source:** (NTU 2008)

Table 5.2.4 Summary of water quality rating curves for storm flows (CP2)

Pollutant	Equation	Application Range (m ³ /s)	R ²
NH ₃ -N	NH ₃ -N (mg/L) = 0.258 NH ₃ -N (g/s) = 0.0054(Q _T) ² + 0.1047(Q _T) - 0.023	Q _T < 0.8 Q _T ≥ 0.8	0.7244
DOC	DOC (mg/L) = 3.559 DOC (g/s) = 2.3843(Q _T) + 1.6805	Q _T < 1.43 Q _T ≥ 1.43	0.7831
POC	POC (mg/L) = 0.701 POC (g/s) = 0.2912(Q _T) + 0.2817	Q _T < 0.69 Q _T ≥ 0.69	0.4738
TN	TN (g/s) = 0.1661(Q _T) ² + 0.8316(Q _T) + 0.01	all	0.8955
TDN	TDN (mg/L) = 0.178 TDN (g/s) = 0.0182(Q _T) ² + 0.5096(Q _T) + 0.0342	Q _T < 0.9 Q _T ≥ 0.9	0.831
NO _x	NO _x (mg/L) = 0.813 NO _x (g/s) = 0.0048(Q _T) ² + 0.5666(Q _T) + 0.0691 NO _x (mg/L) = 0.764	Q _T < 0.28 0.28 ≤ Q _T ≤ 40.8 Q _T > 40.8	0.8954
DON	DON (mg/L) = 0.352 DON (g/s) = 0.0067(Q _T) ² + 0.0507(Q _T) + 0.0105 DON (mg/L) = 0.104	Q _T < 3 3 ≤ Q _T ≤ 7.7 Q _T > 7.7	0.8411
TP	TP (mg/L) = 0.093 TP (g/s) = 0.0948(Q _T) - 0.0125	Q _T < 6.94 Q _T ≥ 6.94	0.8521
TDP	TDP (mg/L) = 0.039 TDP (g/s) = 0.0008 Q _T ² + 0.0142(Q _T) + 0.0089	Q _T < 0.3 Q _T ≥ 0.3	0.8813
OP	OP (mg/L) = 0.29 OP (g/s) = 0.0004(Q _T) ² + 0.0022(Q _T) + 0.0018	Q _T < 2.6 Q _T ≥ 2.6	0.9565
DOP	DOP (mg/L) = 0.086 DOP (g/s) = 0.0027x ² - 0.0005x + 0.0111 DOP (mg/L) = 0.013	Q _T < 0.05 Q _T ≥ 0.05 ≤ 20 Q _T > 20	0.8101
SiO ₂	SiO ₂ (mg/L) = 7.128 SiO ₂ (g/s) = -0.0269(Q _T) ² + 1.8782(Q _T) + 1.6749 SiO ₂ (mg/L) = 0.822	Q _T < 0.32 0.32 ≤ Q _T ≤ 40.8 Q _T > 40.8	0.9029
TSS	TSS (mg/L) = 6 TSS (g/s) = 1.5005(Q _T) ² + 68.293(Q _T) - 5.7275 TSS (mg/L) = 129.371	Q _T < 0.09 0.09 ≤ Q _T ≤ 40.8 Q _T > 40.8	0.9441

Note: Q_T: Total flow (m³/s)**Source: (NTU 2008)**

Table 5.2.5 Summary of water quality rating curves for storm flows (CP4)

Pollutant	Equation	Application Range (m ³ /s)	R ²
NH ₃ -N	NH ₃ -N (mg/L) = 0.157 NH ₃ -N (g/s) = 0.0013(Q _T) ² + 0.0494(Q _T) + 0.0044 NH ₃ -N (mg/L) = 0.066	Q _T < 0.04 0.04 ≤ Q _T ≤ 12.77 Q _T > 12.77	0.8794
DOC	DOC (mg/L) = 5.299 DOC (g/s) = 3.2773(Q _T) + 0.7043	Q _T < 0.35 Q _T ≥ 0.35	0.9457
POC	POC (mg/L) = 0.37 POC (g/s) = 0.4666(Q _T) - 0.0519	Q _T < 0.54 Q _T ≥ 0.54	0.6512
TN	TN (mg/L) = 0.794 TN (g/s) = 1.2787(Q _T) - 0.1975	Q _T < 0.41 Q _T ≥ 0.41	0.9569
TDN	TDN (mg/L) = 0.039 TDN (g/s) = 0.0073(Q _T) ² + 0.5188(Q _T) - 0.0062 TDN (mg/L) = 1.167	Q _T < 0.013 0.013 ≤ Q _T ≤ 88 Q _T > 88	0.9793
NO _x	NO _x (mg/L) = 0.297 NO _x (g/s) = 0.0227(Q _T) ² + 0.1669(Q _T) + 0.0249 NO _x (mg/L) = 0.459	Q _T < 5.53 5.53 ≤ Q _T ≤ 12.77 Q _T > 12.77	0.9256
DON	DON (mg/L) = 0.39 DON (g/s) = 0.1198(Q _T) + 0.0131	Q _T < 0.05 Q _T ≥ 0.05	0.9119
TP	TP (mg/L) = 0.07 TP (g/s) = 0.4415(Q _T) - 0.1993	Q _T < 0.54 Q _T ≥ 0.54	0.8914
TDP	TDP (mg/L) = 0.037 TDP (g/s) = 0.4022(Q _T) - 0.2228	Q _T < 0.61 Q _T ≥ 0.61	0.8502
OP	OP (mg/L) = 0.014 OP (g/s) = 0.0474(Q _T) - 0.0151	Q _T < 0.45 Q _T ≥ 0.45	0.8321
DOP	DOP (mg/L) = 0.017 DOP (g/s) = 0.3871(Q _T) - 0.1775	Q _T < 0.48 Q _T ≥ 0.48	0.9236
SiO ₂	SiO ₂ (mg/L) = 6.502 SiO ₂ (g/s) = 2.3807(Q _T) + 1.4947	Q _T < 0.36 Q _T ≥ 0.36	0.9128
TSS	TSS (mg/L) = 7.6 TSS (g/s) = 101.83(Q _T) - 25.991	Q _T < 0.28 Q _T ≥ 0.28	0.8604

Note: Q_T: Total flow (m³/s)**Source:** (NTU 2008)

Table 5.2.6 Summary of water quality rating curves for storm flows (CP6)

Pollutant	Equation	Application Range (m ³ /s)	R ²
NH ₃ -N	$\text{NH}_3\text{-N (g/s)} = 0.0024(Q_T)^2 + 0.0322(Q_T) + 0.0377$	all	0.6887
DOC	DOC (mg/L) = 6.402 DOC (g/s) = 3.9367(Q _T) + 1.1304	Q _T < 0.46 Q _T => 0.46	0.9651
POC	POC (mg/L) = 0.774 POC (g/s) = 0.2138(Q _T) + 0.1244	Q _T < 0.22 Q _T => 0.22	0.6293
TN	TN (mg/L) = 10.444 TN (g/s) = 2.3326(Q _T) + 0.955	Q _T < 0.118 Q _T => 0.118	0.7625
TDN	TDN (mg/L) = 1.091 TDN (g/s) = 0.8745(Q _T) + 1.0306	Q _T < 4.76 Q _T => 4.76	0.6269
NO _x	NO _x (mg/L) = 0.927 NO _x (g/s) = 0.6169(Q _T) + 0.6657	Q _T < 2.15 Q _T => 2.15	0.4436
DON	DON (mg/L) = 0.307 DON (g/s) = 0.0027(Q _T) ² + 0.2191(Q _T) - 0.0059	Q _T < 0.7 Q _T => 0.7	0.9723
TP	TP (mg/L) = 0.183 TP (g/s) = 1.0533(Q _T) - 0.0341	Q _T < 0.04 Q _T => 0.04	0.8903
TDP	TDP (mg/L) = 0.092 TDP (g/s) = 0.6045(Q _T) - 0.0908	Q _T < 0.18 Q _T => 0.18	0.852
OP	OP (mg/L) = 0.758 OP (g/s) = 0.3186(Q _T) + 0.0296	Q _T < 0.0674 Q _T => 0.0674	0.8194
DOP	DOP (mg/L) = 0.019 DOP (g/s) = 0.0291(Q _T) ² - 0.6808(Q _T) + 0.0355 DOP (mg/L) = 0.341	Q _T < 0.8 0.8 <= Q _T <= 12.57 Q _T > 12.57	0.8001
SiO ₂	SiO ₂ (mg/L) = 7.148 SiO ₂ (g/s) = 2.0528(Q _T) + 0.6935	Q _T < 0.14 Q _T => 0.14	0.9691
TSS	TSS (mg/L) = 887 TSS (g/s) = 39.695(Q _T) ² + 127.45(Q _T) + 20.009 TSS (mg/L) = 887	Q _T < 0.0264 0.0264 <= Q _T <= 19.11 Q _T > 19.11	0.9401

Note: Q_T: Total flow (m³/s)**Source: (NTU 2008)**

Table 5.2.7 Summary of water quality rating curves for storm flows (CP7)

Pollutant	Equation	Application Range (m ³ /s)	R ²
NH ₃ -N	NH ₃ -N (mg/L) = 0.389 NH ₃ -N (g/s) = -5E-05(Q _T) ² + 0.0338(Q _T) + 0.1074	Q _T < 5.5 Q _T => 5.5	0.6259
DOC	DOC (g/s) = 0.0979(Q _T) ² + 2.5121(Q _T) + 1.7527	all	0.6824
POC	POC (mg/L) = 37.59 POC (g/s) = 0.3128(Q _T) + 0.1774	Q _T < 0.0048 Q _T => 0.0048	0.2241
TN	TN (mg/L) = 0.655 TN (g/s) = 1.9694(Q _T) - 2.3183	Q _T < 1.76 Q _T => 1.76	0.8152
TDN	TDN (mg/L) = 0.627 TDN (g/s) = 0.0206(Q _T) ² + 0.5286(Q _T) + 1002 TDN (mg/L) = 1.599	Q _T < 3.3 3.3 <= Q _T <= 51.89 Q _T > 51.89	0.8158
NO _x	NO _x (mg/L) = 0.487 NO _x (g/s) = 0.0296(Q _T) ² + 0.0322(Q _T) + 0.4749 NO _x (mg/L) = 1.577	Q _T < 14.24 14.24 <= Q _T <= 51.89 Q _T > 51.89	0.8818
DON	DON (mg/L) = 0.053 DON (g/s) = 0.2845(Q _T) - 0.1892	Q _T < 0.82 Q _T => 0.82	0.3511
TP	TP (mg/L) = 1.046 TP (g/s) = 0.0947(Q _T) + 0.0735	Q _T < 0.0773 Q _T => 0.0773	0.3978
TDP	TDP (g/s) = 0.0028(Q _T) ² + 0.0005(Q _T) + 0.0103	all	0.6674
OP	OP (mg/L) = 0.008 OP (g/s) = 0.0004(Q _T) ² + 0.001(Q _T) + 0.0042 OP (mg/L) = 0.009	Q _T < 16.88 16.88 <= Q _T <= 19.81 Q _T > 19.81	0.5449
DOP	DOP (g/s) = 0.0023(Q _T) ² - 0.0042(Q _T) + 0.0269	all	0.6213
SiO ₂	SiO ₂ (mg/L) = 9.973 SiO ₂ (g/s) = 2.8196(Q _T) + 6.2627	Q _T < 0.88 Q _T => 0.88	0.8919
TSS	TSS (mg/L) = 5 TSS (g/s) = 0.9891(Q _T) ² + 84.222(Q _T) - 89.018 TSS (mg/L) = 136.327	Q _T < 1.08 1.08 <= Q _T <= 51.89 Q _T > 51.89	0.9596

Note: Q_T: Total flow (m³/s)**Source: (NTU 2008)**

Table 5.2.8 Unit area pollutant loads in each study site for dry weather flow

Unit: kg/yr-ha														
Site	Year	NH₃-N	DOC	POC	TN	TDN	NO_x	DON	TP	TDP	DOP	OP	SiO₂	TSS
CP1	2005	0.82	24	1.89	6.09	5.35	2.61	1.92	0.34	0.16	0.10	0.06	39	101
	2006	0.83	24	1.91	6.13	5.39	2.63	1.93	0.34	0.17	0.10	0.06	39	102
	2007	0.83	24	1.9	6.11	5.37	2.62	1.92	0.34	0.17	0.10	0.06	39	102
	mean	0.83	24	1.9	6.11	5.37	2.62	1.92	0.34	0.17	0.10	0.06	39	102
CP2	2005	1.11	14	4.00	6.99	5.76	3.26	1.01	0.35	0.17	0.05	0.12	24	21
	2006	1.12	14	4.10	7.16	5.9	3.34	1.04	0.35	0.18	0.05	0.13	25	22
	2007	1.16	14	4.23	7.39	6.09	3.45	1.07	0.37	0.18	0.05	0.13	25	22
	mean	1.13	14	4.11	7.18	5.92	3.35	1.04	0.36	0.18	0.05	0.13	25	22
CP4	2005	1.22	43	6.9	8.97	5.74	1.82	2.69	0.52	0.29	0.19	0.10	44	65
	2006	1.16	41	6.51	8.47	5.42	1.72	2.54	0.49	0.28	0.18	0.10	41	61
	2007	1.22	43	6.85	8.91	5.7	1.81	2.67	0.51	0.29	0.19	0.10	43	64
	mean	1.20	42	6.76	8.78	5.62	1.78	2.63	0.51	0.29	0.19	0.10	43	63
CP6	2005	1.57	58	2.33	13.7	11.73	7.71	2.45	1.13	0.76	0.23	0.53	54	50
	2006	1.53	56	2.28	13.4	11.48	7.54	2.4	1.11	0.74	0.22	0.52	52	49
	2007	1.50	55	2.24	13.16	11.27	7.41	2.36	1.09	0.73	0.22	0.51	51	48
	mean	1.53	56	2.28	13.42	11.49	7.55	2.4	1.11	0.74	0.22	0.52	52	49
CP7	2005	0.88	21	1.86	7.91	6.72	6.72	0.55	0.42	0.28	0.14	0.15	93	57
	2006	0.86	21	1.82	7.75	6.59	6.59	0.54	0.41	0.28	0.13	0.14	91	56
	2007	0.86	21	1.82	7.78	6.61	6.61	0.54	0.41	0.28	0.13	0.14	91	56
	mean	0.86	21	1.83	7.81	6.64	6.64	0.55	0.41	0.28	0.13	0.14	92	57

Table 5.2.9 Unit area pollutant loads in each study site for storm runoff

Unit: kg/yr-ha													
Site	Year	NH ₃ -N	DOC	POC	TN	TDN	NO _x	DON	TP	TDP	DOP	OP	SiO ₂ TSS
CP1	2005	1.50	35.30	4.91	17.74	7.16	6.09	2.01	0.87	1.66	0.32	0.16	29 679
	2006	1.87	41.47	5.69	23.64	8.86	7.54	2.40	1.08	2.41	0.38	0.19	33 870
	2007	2.06	46.99	6.46	24.87	9.83	8.46	2.68	1.22	2.28	0.44	0.20	38 964
	mean	1.81	41.25	5.69	22.08	8.62	7.36	2.36	1.06	2.12	0.38	0.18	33 838
CP2	2005	1.50	41.09	5.83	15.12	7.38	6.46	2.36	0.91	0.99	0.37	0.18	35 669
	2006	1.88	48.23	6.76	20.48	9.12	8.02	2.75	1.14	1.49	0.43	0.21	40 863
	2007	3.39	78.38	10.70	43.34	16.21	14.48	4.48	2.10	3.77	0.71	0.33	63 1675
	mean	2.26	55.90	7.76	26.31	10.90	9.65	3.19	1.38	2.08	0.50	0.24	46 1069
CP4	2005	0.34	22.28	2.52	6.63	3.15	1.57	0.77	1.79	1.52	0.67	1.54	20 474
	2006	0.29	20.37	2.03	5.13	2.56	1.27	0.70	1.16	0.95	1.19	0.97	21 320
	2007	0.37	25.54	2.80	7.32	3.49	1.62	0.87	1.89	1.55	0.73	1.61	24 516
	mean	0.34	22.73	2.45	6.36	3.07	1.49	0.78	1.61	1.34	0.86	1.37	22 437
CP6	2005	1.11	40.84	2.98	27.64	16.49	5.35	1.74	6.87	3.08	0.54	2.56	23 1535
	2006	1.15	46.87	3.58	32.44	17.20	5.58	1.86	7.03	2.87	0.75	2.78	26 1755
	2007	2.09	65.45	4.82	44.52	27.80	22.69	2.74	10.74	4.73	1.01	4.05	36 2599
	mean	1.45	51.05	3.79	34.86	20.50	11.21	2.11	8.21	3.56	0.77	3.13	28 1963
CP7	2005	1.13	25.88	2.75	8.58	7.96	3.85	1.12	0.66	0.33	0.09	0.24	27 442
	2006	1.50	31.94	3.44	10.54	9.26	4.19	0.58	0.53	0.30	0.03	0.25	36 535
	2007	1.57	36.95	4.14	13.25	10.04	4.60	1.69	0.99	0.34	0.13	0.28	40 668
	mean	1.40	31.59	3.44	10.79	9.09	4.21	1.13	0.73	0.32	0.08	0.26	34 548

The comparison of the unit area loading rates (flux rates) of dry weather flow and storm flow for each study site are shown in Figure 5.2.1. The WWF loading rates were estimated from the regression equation in Table 5.2.3 to 5.2.7. For those pollutants exhibiting loading rate – total runoff rate relationships with R^2 lower than 0.70, EMC values were represent the pollutant concentrations during storm flows. The regression and EMC results are summarized in Table 5.2.9. The dry weather loads were computed using Table 4.1.1, assuming constant concentration values. Tables 5.2.8 and 5.2.9 show the annual dry weather and wet-weather flux rates for TN, TDN, NO_x, TP TDP and OP at CP6 were greater than the other four study sites. This result could be related to the land use in CP6 which contains an agriculture area. Table 5.2.9 shows estimates of the loading rates during WWF. The unit area WWF loading rate at CP2 is more dominant compared with CP1 and CP7. It is concluded that urbanization has significant influence on wet-weather loading rates.

Estimates of unit area annual loads by the regression approach for each study site given in Table 5.2.10. The figures in Appendix E and Table 5.2.10 show DOC, TN, TP,

TSS are the dominant loads in CP6. Unit area TSS loads at CP6 is greater by approximately 2 to 4 times compared to those of other study sites, and TN and TP are about 2 times and 4 times higher respectively. In general, the mass loading rates of TSS, TN, NO_x, and TDN are in the order of: agriculture > residential > undeveloped watershed. The mass loadings of SiO₂ are in the order: residential > agriculture > undeveloped area. The unit area loading rates from CP2 (68% urban) are higher than CP1 (39% of urban) and CP7 (46% of urban). These suggest that the pollutant loading rates are affected by urbanization.

Table 5.2.10 Unit area pollutant loads in each study site

Unit: kg/yr-ha														
Site	Year	NH ₃ -N	DOC	POC	TN	TDN	NO _x	DON	TP	TDP	DOP	OP	SiO ₂	TSS
CP1	2005	2.3	59.1	6.8	24	12.5	8.7	3.9	1.2	1.8	0.4	0.2	68	780
	2006	2.7	65.5	7.6	30	14.3	10.2	4.3	1.4	2.6	0.5	0.3	73	972
	2007	2.9	70.9	8.4	31	15.2	11.1	4.6	1.6	2.4	0.5	0.3	77	1065
	mean	2.6	65.2	7.6	28	14.0	10.0	4.3	1.4	2.3	0.5	0.2	73	939
CP2	2005	2.6	54.7	9.8	22	13.1	9.7	3.4	1.3	1.2	0.4	0.3	59	690
	2006	3.0	62.2	10.9	28	15.0	11.4	3.8	1.5	1.7	0.5	0.3	65	885
	2007	4.6	92.8	14.9	51	22.3	17.9	5.5	2.5	4.0	0.8	0.5	88	1,697
	mean	3.4	69.9	11.9	33	16.8	13.0	4.2	1.7	2.3	0.6	0.4	71	1,091
CP4	2005	1.6	65.3	9.4	16	8.9	3.4	3.5	2.3	1.8	0.9	1.6	64	539
	2006	1.5	60.9	8.5	14	8.0	3.0	3.2	1.6	1.2	1.4	1.1	62	382
	2007	1.6	68.2	9.7	16	9.2	3.4	3.5	2.4	1.8	0.9	1.7	67	580
	mean	1.5	64.8	9.2	15	8.7	3.3	3.4	2.1	1.6	1.0	1.5	64	500
CP6	2005	2.7	98.4	5.3	41	28.2	13.1	4.2	8.0	3.8	0.8	3.1	76	1,585
	2006	2.7	103.2	5.9	46	28.7	13.1	4.3	8.1	3.6	1.0	3.3	79	1,804
	2007	3.6	120.7	7.1	58	39.1	30.1	5.1	11.8	5.5	1.2	4.6	88	2,647
	mean	3.0	107.4	6.1	48	32.0	18.8	4.5	9.3	4.3	1.0	3.7	81	2,012
CP7	2005	2.0	46.9	4.6	16	14.7	10.6	1.7	1.1	0.6	0.2	0.4	120	499
	2006	2.4	52.6	5.3	18	15.8	10.8	1.1	0.9	0.6	0.2	0.4	127	591
	2007	2.4	57.7	6.0	21	16.7	11.2	2.2	1.4	0.6	0.3	0.4	131	724
	mean	2.3	52.4	5.3	19	15.7	10.9	1.7	1.1	0.6	0.2	0.4	126	605

Figure 5.2.1 Comparison of the unit area loading rates of dry weather flow and storm flow for each study site

Table 5.2.11 reveals that over 87 % of TSS is contributed by storm flows in all of the catchments. Nearly 88%-98% of the TSS annual average load is from urban (CP1, CP2, CP7) catchments' WWF. These are close to the finding of McPherson et al. (2005) at 95%. This demonstrates that TSS is picked up during the overland runoff and channel flow. Greater than 65% of the P loading from each catchment occurs in the WWF. For

nearly all the contaminants, storm flow contribution is greater than dry-weather contribution at CP1, CP2, CP6 and CP7. However, for almost all the parameters, DWF contributes more pollutants than WWF at CP4, except for P and TSS. Thus, DWF quality control may be important for CP4.

Furthermore, DWF contributes more NO_x , DOP and SiO_2 than WWF at CP7, which has several abstraction ponds. The detention of flow in the ponds could affect the pollutant loads in flow. A stormwater abstraction pond located within the CP7 catchment which collects and pumps excess stormwater to other storage reservoirs could reduce the pollutant loading during storm weather period. CP6 which has 15% agriculture area, shows $\text{NH}_3\text{-N}$, DOC, NO_x , DON contributions are larger during DWF than WWF. Table 5.2.11 shows the percentage of pollutant contribution from storm runoff in 2007 is greater than 2005 and 2006. As the rainfall depth in 2007 was higher by 36~45% compared to the previous two years, it can be seen that the total pollutant load is related to rainfall amount.

Table 5.2. Percent of unit area loads from storm runoff

Units: %														
Site	Year	NH_3	DOC	POC	TN	TDN	NO_x	DON	TP	TDP	DOP	OP	SiO_2	TSS
CP1	2005	65	60	72	74	57	70	51	72	91	76	71	43	87
	2006	69	63	75	79	62	74	55	76	94	79	75	46	90
	2007	71	66	77	80	65	76	58	78	93	81	76	49	90
	mean	69	63	75	78	62	74	55	76	93	79	74	46	89
CP2	2005	58	75	59	68	56	66	70	72	85	88	59	59	97
	2006	63	78	62	74	61	71	73	76	89	90	62	62	98
	2007	75	84	72	85	73	81	81	85	95	93	72	71	99
	mean	67	80	65	79	65	74	75	80	92	91	65	65	98
CP4	2005	22	34	27	43	35	46	22	78	84	78	94	32	88
	2006	20	33	24	38	32	43	22	70	77	87	91	33	84
	2007	23	37	29	45	38	47	25	79	84	80	94	36	89
	mean	22	35	27	42	35	46	23	76	82	82	93	34	87
CP6	2005	41	42	56	67	58	41	42	86	80	70	83	30	97
	2006	43	45	61	71	60	43	44	86	79	77	84	33	97
	2007	58	54	68	77	71	75	54	91	87	82	89	41	98
	mean	49	48	62	72	64	60	47	88	83	78	86	35	98
CP7	2005	56	55	60	52	54	36	67	61	54	39	62	23	89
	2006	64	61	65	58	58	39	52	56	52	17	64	28	90
	2007	65	64	69	63	60	41	76	71	55	50	66	30	92
	mean	62	60	65	58	58	39	67	64	54	38	64	27	91

Table 5.2.12 The comparison of annual pollutant load with other studies**Units: kg/yr-ha**

	NH₃-N	TN	NO_x	TP	OP	TSS	Land use type
CP1	2.6	28.2	10.0	1.4	0.2	939	Residential/ undeveloped
CP2	3.4	33.5	13.0	1.7	0.4	1,091	Urban
CP4	1.5	15.1	3.3	2.1	1.5	500	Undeveloped
CP6	3.0	48.3	18.8	9.3	3.7	2,012	Agricultural /undeveloped
CP7	2.3	18.6	10.9	1.1	0.4	605	Urban/ undeveloped
Barrett et al. (1998)	-	-	5.4	2	-	1,306	Rural/residential (West 35 th St. T.X., US)
	-	-	8.7	0.8	-	977	Commercial/ high density residential (Convict, T.X., US)
	-	-	0.7	0.2	-	101	Commercial/ residential (Walnut, T.X., US)
Neal et al. (2004)	25.2	-	6.5	-	-	-	Rural (Frilsham, UK)
	24	-	6.9	-	-	-	Rural / agricultural (Warren Farm, UK)
Flint (2004)	25 (TKN)	-	11.2-11.6 (NO ₃)	3.9 - 4.6	-	3,100	Urban (Mt. Rainier, MD, US)
McPherson et al. (2005)	-	-	-	-	-	2,786	Urban (Ballona, L.A. Calif., US)
Lee and Bang (2000)	22.4 (TKN)	-	1.6	14.8	7.3	1,803	Residential (Taejon and Chongju, Korea)
Ide et al. (2003)	-	8.35	-	0.07	-	-	Forest/ commercial (various city, Japan)
Dhia et al. (2008)	-	37.5	-	2.1	-	-	Urban (Orange, Australia)
Chen and Adams (2006)	3.85 (TKN)	-	-	0.71	-	223	Urban (Great Lake, Canada)
Wotling and Bouvier (2002)	-	10.9	-	3.6		1,288	Urban (Vaiami, France)
	-	3.6	-	0.9		535	Forested (Matatia, France)

Several investigations on estimated annual pollutant loading are shown in Table 5.2.12. The annual pollutant loads for TSS and TP at CP1 and CP2 were close to the finding of Barrett et al. (1998) in US and Wothling and Bouvier (2002) in France.

However, they are not consistent with the results of Flint (2004), McPherson et al. (2005) and Chen and Adams (2006), as shown in Table 5.2.12. Neal et al. (2004) found that $\text{NH}_3\text{-N}$ in Warren Farm (agricultural land use) was up to 10 times larger than CP6, whereas NO_x was found to be 2 times lower than CP6. Most of land use for CP6 are fish farm and chicken farm, it suggests of soil fertilizer or animal production will be different. The simulated TP and TN loadings from the CP4 were comparable with those reported by Wotling and Bouvier (2002) for a rural watershed in France. TN loading was close to the finding of Dhia et al. (2008) for a rural catchment in Australia. The findings on NO_x (10 -13 kg/yr-ha) in this study, which includes urban areas, is higher than that found by Lee and Bang (2002) (1.6kg/yr-ha for a residential water shed). TN and TP loading reported are lower than those of Ide et al. (2003) for an urban area. The relative contribution of the loadings for TN, NO_x , TP, OP and TSS in this study is in order of: agriculture > urban > undeveloped areas.

The total rainfall and unit area runoff were correlated to unit area load on a weekly basis, with the linear relationships developed for parameters (DOC, POC, TP, TN, SiO_2 , TSS) as shown in Table 5.2.13. The relationships between the rainfall depth and the water quality unit loads show R^2 values >0.9 for CP1, CP2 and CP7, R^2 >0.8 for CP6, and R^2 >0.5 for CP4. This indicates that rainfall depth and unit load for CP1, CP2, CP6 and CP7 are highly correlated. The correlation between flow rate and water quality for all the parameters showed good fit with R^2 values higher than 0.9. Furthermore, the correlations between TSS and TN, TP, POC, DOC were also high (R^2 >0.9), as shown in Table 5.2.13. The loading rates DOC, POC, TP and TN can hence be simulated in terms of the TSS loading.

Table 5.2.13 The correlations of rainfall, total flow and TSS and loading rate**Unit: kg/week-ha**

Site	Pollutant	Equation	R ²	Equation	R ²	Equation	R ²
CP1	DOC	0.0187(R) + 0.2165	0.915	0.0032(Q _T) + 0.1253	0.8663	0.0068(TSS) + 0.0815	0.9695
	POC	0.004(R) + 0.0356	0.9307	0.0009(Q _T) - 0.0059	0.9583	0.0051(TSS) + 0.0531	0.9493
	TP	0.0005(R) - 0.0004	0.9127	0.0001(Q _T) - 0.0061	0.9957	0.0008(TSS) + 0.005	0.9582
	TN	0.0033(R) + 0.0835	0.9191	0.0007(Q _T) + 0.0453	0.9824	0.0054(TSS) + 0.1252	0.9029
	SiO ₂	0.0233(R) + 0.6424	0.9081	0.0051(Q _T) + 0.3819	0.9625	-	-
	TSS	0.5782(R) - 5.8062	0.8986	0.1268(Q _T) - 12.035	0.9397	-	-
CP2	DOC	0.0168(R) + 0.3334	0.9531	0.0032(Q _T) + 0.1253	0.8663	0.0069(TSS) + 0.0962	0.9486
	POC	0.0023(R) + 0.0756	0.9483	0.0005(Q _T) + 0.0368	0.9459	0.0066(TSS) + 0.0994	0.9491
	TP	0.0004(R) + 0.0062	0.9625	9E-05(Q _T) - 0.0013	0.9818	0.0013(TSS) + 0.0098	0.9943
	TN	0.0082(R) + 0.0883	0.8575	0.0018(Q _T) - 0.0647	0.9079	0.0255(TSS) + 0.1411	
	SiO ₂	0.0141(R) + 0.6374	0.9502	0.003(Q _T) + 0.4041	0.9453	-	-
	TSS	0.3304(R) - 2.5163	0.945	0.0707(Q _T) - 8.3768	0.9746	-	-
CP4	DOC	0.0063(R) + 0.8167	0.7084	0.0036(Q _T) + 0.2441	0.9977	0.0048(TSS) + 0.1129	0.9965
	POC	0.0006(R) + 0.1155	0.5854	0.0004(Q _T) + 0.047	0.9905	0.0047(TSS) + 0.1233	0.982
	TP	0.0005(R) + 0.0074	0.5305	0.0003(Q _T) - 0.0473	0.9723	0.0038(TSS) + 0.0041	0.9956
	TN	0.0018(R) + 0.1668	0.6065	0.0011(Q _T) - 0.0213	0.9953	0.0132(TSS) + 0.161	0.9977
	SiO ₂	0.0061(R) + 0.8287	0.7728	0.0034(Q _T) + 0.3039	0.9704	-	-
	TSS	0.1299(R) + 0.6272	0.5747	0.0857(Q _T) - 13.661	0.9875	-	-
CP6	DOC	0.0226(R) + 0.3751	0.862	0.0053(Q _T) + 0.2048	0.9855	0.0018(TSS) + 0.0767	0.9626
	POC	0.0016(R) + 0.0515	0.8828	0.0004(Q _T) + 0.0405	0.9699	0.0016(TSS) + 0.0543	0.9588
	TP	0.0004(R) + 0.0046	0.9434	0.001(Q _T) - 0.0887	0.9981	0.0045(TSS) + 0.0041	0.9777
	TN	0.0148(R) + 0.0107	0.8681	0.0035(Q _T) - 0.0999	0.9787	0.0164(TSS) + 0.2303	0.9739
	SiO ₂	0.012(R) - 0.0643	0.8611	0.0029(Q _T) - 0.1586	0.9825	-	-
	TSS	0.8709(R) - 11.809	0.8296	0.213(Q _T) - 19.468	0.9778	-	-
CP7	DOC	0.0169(R) + 0.3275	0.9432	0.0062(Q _T) - 0.8761	0.9973	0.0689(TSS) + 0.4205	0.9744
	POC	0.0017(R) + 0.0306	0.9433	0.0007(Q _T) - 0.0997	0.9967	0.0068(TSS) + 0.0415	0.972
	TP	0.0039(R) - 0.0472	0.7957	0.0001(Q _T) - 0.0239	0.996	0.0015(TSS) + 0.0069	0.9708
	TN	0.0061(R) + 0.1004	0.9358	0.0023(Q _T) - 0.3602	0.9976	0.0242(TSS) + 0.1348	0.9896
	SiO ₂	0.0368(R) + 1.9818	0.9433	0.0139(Q _T) - 0.7642	0.9954	-	-
	TSS	0.2487(R) - 1.2048	0.9142	0.0947(Q _T) - 20.012	0.9799	-	-

Note: R: Rainfall (mm/week), Q_T: Total flow (m³/week-ha), TSS: (kg/week-ha)

5.2.3 Analysis based on Simple Method

Simple Method (SM) and Regression Method were used to estimate the wet-weather pollutant loadings for the study sites. The annual pollutant loadings, as computed using the Simple Method (SM) specified in (Eq.) 4.24, are directly proportional to the annual precipitation and runoff coefficient. The method is based on the annual rainfall data at CP1 for year 2005-2006. The average rainfall depth is 2563.05 mm for year 2005 and 2723.29 mm for year 2006. The rainfall data in 2007 are based on data measured at each gauging station, 3114.4mm for CP1, 4707.2mm for CP2, 3859mm for CP4, 3869.2mm for CP6 and 3140mm for CP7. The direct runoff data were simulated by XP-SWMM at 15 min intervals, and the baseflow was modeled based on the empirical equations, as discussed in Section 4.1.6.

In general, Simple Method neglects baseflow as it is intended for small urban areas, since the contribution of dry weather to the total load is insignificant. It is primarily designed for developed watershed having less than 259 ha and is not intended for use on the undeveloped areas, agriculture areas and industrial areas (Schueler, 1987).

In this study, the pollutant loadings computed from the Simple Method are compared to estimates based on the full year analysis storm runoff in Table 5.2.14. Due to the fact that the Simple Method estimates pollutant load during wet-weather flow, it implicitly neglects pollutants associated with baseflow volume. The loading rate deduced by Simple Method in this study is compared to pollutant loading in term of storm runoff volume, which is close to the wet-weather flow loading rate. The comparison shows that the values estimated by the 2 methods are quite close with each other, except for CP4. The annual load estimated by the Simple Method is less than half of the full year estimates. In this study, it is suggested that Simple Method can accurately predict annual loading rate in urban area. Chandler (1994) compared estimation from Simple Method and complex model. The results showed that the annual loading rates derived by these two methods are quite close to each other. This indicates that the estimation based on Simple Method can be used to provide good estimates for unit area loading rate in undeveloped watersheds.

Table 5.2.14 Comparison of pollutant loading rates from storm runoff based on rating curve and the Simple Method (SM)

Units: kg/ha-yr																		
Site	Estimation method	NH ₃ -N	DOC	POC	TN	TDN	NO _x	DON	TP	TDP	DOP	OP	SiO ₂	TSS	Rv	Annual Rainfall (mm)	Area (ha)	Reference
CP1	SM	1.2	33.7	3.4	6.2	8.4	3.7	3.5	0.8	0.29	0.08	0.21	51	929	0.33	2800	522	This Study
	Regression	1.8	41.3	5.7	22.1	8.6	7.4	2.4	1.1	2.12	0.38	0.18	33	838				
CP2	SM	2.1	45.4	4.7	18.3	9.8	8.4	2	1.1	0.49	0.42	0.07	34	842	0.44	3331	200	
	Regression	2.3	55.9	7.8	26.3	10.9	9.7	3.2	1.4	2.08	0.50	0.24	46	1069				
CP4	SM	0.2	12.2	1.5	3.6	1.6	0.7	0.7	0.4	0.04	0.02	0.02	14	311	0.13	3048	288	
	Regression	0.3	22.7	2.5	6.4	3.1	1.5	0.8	1.6	1.34	0.86	1.37	22	437				
CP6	SM	1.1	34.6	1.6	24.1	15.7	5.1	9.5	6.5	2.88	0.77	2.11	21	1485	0.22	3052	145	
	Regression	1.5	51.1	3.8	34.9	20.5	11.2	2.1	8.2	3.56	0.77	3.13	28	1963				
CP7	SM	0.3	36.2	4.5	10.3	3.9	3	0.5	0.5	0.06	0.04	0.02	34	490	0.26	2809	1,560	
	Regression	1.4	31.6	3.4	10.8	9.1	4.2	1.1	0.7	0.32	0.08	0.26	34	548				
West 35th St. T.X., US Convict, T.X., US Walnut, T.X., US	SM	-	-	-	-	-	5.4	-	2	-	-	-	-	1,306	0.93	825	0.5	Barrett et al. (1998)
	SM	-	-	-	-	-	8.7	-	0.8	-	-	-	-	977	0.4		0.05	
	SM	-	-	-	-	-	0.7	-	0.2	-	-	-	-	101	0.83		10.5	
Mt. Rainier, MD, US	SM	TKN: 25	-	-	-	-	-	-	3.9-4.6	-	-	-	-	3100	0.95	1123	0.56	Flint (2004)
Santa Clara (US)	SWMM	-	0.39	-	-	-	0.1	-	0.2	-	-	-	-	73	0.26	1270	178,710	Chandler (1994)
	SM	-	0.39	-	-	-	0.2	-	0.1	-	-	-	-	38	-		-	
Lake Union (US)	SWMM	-	21.7	-	-	-	-	-	0.9	-	-	-	-	1487	0.59	1006	1054	
	SM	-	19.5	-	-	-	-	-	3.3	-	-	-	-	1131	-		-	
Covington (US)	HSPF	-	-	-	-	-	-	-	0.3	-	-	-	-	278	0.29	911	501	
	SM	-	-	-	-	-	-	-	0.7	-	-	-	-	239	-		-	
Scriber (US)	HSPF	-	-	-	-	-	-	-	2.2	-	-	-	-	1633	0.34	911	1776	
	SM	-	-	-	-	-	-	-	3.2	-	-	-	-	1082	-		-	

Note: SM: Simple Method, Regression: Rating curve and EMC

5.3 First Flush and Second Flush Behavior

Table 5.3.1 Evaluation of Pollutant Flushing

Sub-Catchments	Pollutant	No. of storm events	No. of events exhibiting qualitative flushing	Events exhibiting qualitative flushing (%)	Median values	Median value
					for % mass pollutant load flushed in the first flush	for % mass pollutant load in the second flush
CP1	TSS	8	6	75	33	19
	TP	12	4	33	27	20
	NO _x	5	2	40	27	19
	POC	8	4	50	30	26
	OP	8	3	38	26	25
	TN	8	1	13	25	24
	DOC	12	2	17	22	24
	SiO ₂	12	0	0	20	23
	TDP	12	1	8	24	27
	TDN	5	1	20	27	21
	NH ₃ -N	6	1	17	24	26
CP2	TSS	9	6	67	33	28
	TP	10	6	60	30	26
	NO _x	9	3	33	29	25
	POC	7	6	86	33	28
	OP	7	2	29	20	18
	TN	4	3	75	32	24
	DOC	7	7	100	33	22
	SiO ₂	9	0	0	23	18
	TDP	9	3	33	24	22
	TDN	9	3	33	24	22
	NH ₃ -N	11	3	27	24	26

Continued

Sub-Catchments	Pollutant	No. of storm events	No. of events exhibiting qualitative flushing	Events exhibiting qualitative flushing (%)	Median values	Median value
					for % mass	for % mass
					pollutant load	pollutant load
					flushed in the first flush (%)	in the second flush (%)
CP4	TSS	4	1	25	25	29
	TP	4	0	0	17	28
	NO _x	4	2	50	39	15
	POC	2	2	100	44	31
	OP	2	0	0	21	23
	TN	2	0	0	19	26
	DOC	2	1	50	30	12
	SiO ₂	4	1	25	24	23
	TDP	4	0	0	22	25
	TDN	3	0	0	17	22
	NH ₃ -N	4	1	25	26	24
CP6	TSS	7	5	71	42	25
	TP	5	4	80	35	28
	NO _x	5	2	40	23	25
	POC	4	3	75	55	16
	OP	6	1	17	18	29
	TN	4	2	50	29	22
	DOC	4	2	50	30	19
	SiO ₂	5	1	20	23	20
	TDP	5	1	20	21	29
	TDN	4	1	25	17	21
	NH ₃ -N	5	1	20	16	21
CP7	TSS	10	8	80	40	28
	TP	9	7	78	33	32
	NO _x	9	4	44	26	26
	POC	7	6	86	46	15
	OP	1	0	0	22	32
	TN	8	6	75	38	24
	DOC	7	5	71	38	21
	SiO ₂	8	0	0	21	18
	TDP	4	1	25	15	25
	TDN	8	3	38	29	23
	NH ₃ -N	9	4	44	28	22

Table 5.3.2 Qualitative Evaluation of Flushing for Total Event

Pollutant	No. of storm events	No. of events exhibiting qualitative flushing	Events exhibiting qualitative flushing (%)
TSS	38	26	68
TP	40	21	53
NO _x	32	13	41
POC	28	21	75
OP	24	6	25
TN	26	12	46
DOC	32	17	53
SiO ₂	38	2	5
TDP	34	6	18
TDN	29	8	28
NH ₃ -N	35	10	29

Note: No: number

In order to qualitatively characterize the pollutant mass flushing throughout the entire storm event, graphs of mass loading ratio, $M(t)$ against runoff volume ratio, $V(t)$, (Eqs 4.26 and 4.27) were plotted for each of the four pollutants, as shown in the Appendix F. Qualitatively, where the $M(t)$ plot resides above the $V(t)$ plot, it indicates that Mass Based First Flush (MBFF) occurs.

The present study examined 24 to 40 rainfall-runoff events for TSS, TP, NO_x, POC, OP, TN, DOC SiO₂, TDP, TDN, NH₃-N. The first flush behavior is evaluated based on the MBFF concept, for the parameters of TSS, TP, NO_x, POC, OP, TN, DOC, SiO₂, TDP, TDN and NH₃-N at the 5 gauging station. The plots for the 11 parameters can be found in Appendix F. From Appendix F, the measured MBFF of TSS and POC exhibited maximum concentration during the rising limb of the hydrograph.

Quantitative flushing was observed to occur in the highest percentage of storm

events for POC at 75%, and in the smallest percentage of the storm events for SiO_2 , at 5%. NO_x exhibited first flush in 41% of all examined storm events, TP in 53% of the events. These are not consistent with Flint and Davis (2007)'s observation of 50% and 73% of storm events for NO_x and TP respectively. The results indicate that first flush does not occur very frequently for NO_x . By Helsel et al. (1979)'s first flush criteria of $M(t) > V(t)$, quantitative flushing occurred in 68 % of storm events for TSS (Table 5.3.2), which is close to the findings of Deletic (1998) at 70%.

Based on the % of pollutant mass load flushed in the first and subsequent 25% portion of the runoff volume, CP7 shows highest probability flushing behavior, which have a the largest watershed area. However, this result converse with Lee et al. (2000), who supported the first flush phenomenon, was greater for smaller watershed areas.

Table 5.3.1 shows the Median values for % of pollutant mass load flushed in the first 25 %, and subsequent 25% portions of the runoff volume. The median values for TSS, OP, TP, TN, TDP and TDN in the second flush at CP4 are greater than the first flush. 'Second flush' was observed to be stronger than 'first flush' for SiO_2 , TDP and $\text{NH}_3\text{-H}$ at CP1, $\text{NH}_3\text{-N}$ at CP2, NO_x , OP, TDP, TDN, $\text{NH}_3\text{-N}$ at CP6, OP and TDP at CP7. Table 5.3.2 shows the number of 'flush events' out of the 'total events'.

Table 5.3.3 % of pollutant mass load flushed in the first 25 %, and subsequent 25% portions of the runoff volume in CP1

Site	DATE	Antecedent dry period(h)	Rainfall mm	TSS		TP		NO _x		POC		OP		TN		DOC		SiO ₂		TDP		TDN		NH ₃ -N	
				FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF
CP1	8-Jun-05	165:10	13.18	-	-	21	25	-	-	28	30	19	23	24	31	30	21	29	27	38	19	-	-	29	44
	29-Dec-05	28:20	5.84	-	-	30	25	-	-	-	-	-	-	22	20	-	-	-	-	-	-	-	-	-	-
	18-Mar-06	529:15	4.81	22	28	-	-	-	-	43	20	30	24	-	-	36	25	13	8	-	-	-	-	-	-
	8-Apr-06	42:40	24.87	-	-	24	25	26	26	-	-	15	14	23	24	22	22	20	23	9	9	-	-	22	28
	11-May-06	1:10	25.39	-	-	11	18	-	-	49	15	-	-	28	22	20	21	22	21	26	31	29	17	63	10
	25-May-06	59:30	18.54	32	20	23	25	28	26	-	-	37	33	25	27	26	29	22	24	29	28	-	-	26	25
	30-May-06	44:55	9.14	48	28	42	28	16	20	12	21	23	43	31	25	23	21	20	18	22	31	22	24	20	27
	14-Jun-06	68:00	17.00	31	10	21	9	45	11	10	20	37	26	26	25	18	26	15	23	25	29	27	26	15	21
	6-Jul-06	373:00	22.06	55	14	38	22	8	16	23	36	26	25	-	-	29	25	-	-	22	19	32	18	-	-
	29-Aug-06	335:35	43.19	10	47	-	-	32	14	-	-	-	-	22	24	17	24	15	28	13	13	22	24	-	-
	15-Sep-06	18:10	59.65	36	14	13	30	-	-	-	-	-	-	-	-	20	27	23	24	22	29	-	-	-	-
	18-Oct-06	ND	ND	-	-	33	30	-	-	49	28	-	-	-	-	25	23	24	24	26	29	-	-	-	-
	31-Oct-06	25:10	78.47	-	-	26	28	-	-	-	-	6	39	-	-	-	-	21	20	18	18	-	-	-	-
	3-Nov-06	21:25	59.44	32	27	26	27	-	-	37	28	26	24	-	-	19	24	20	22	26	25	-	-	-	-

Note: (FF: First flush, SF: Second flush)

Table 5.3.4 % of pollutant mass load flushed in the first 25 %, and subsequent 25% portions of the runoff volume in CP2

Site	DATE	Antecedent dry period(h)	Rainfall mm	TSS		TP		NO _x		POC		OP		TN		DOC		SiO ₂		TDP		TDN		NH ₃ -N	
				FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF
CP2	31-Oct-06	25:10	114.81	-	-	17	25	34	23	-	-	-	-	-	-	34	22	24	14	10	40	10	40	28	36
	3-Nov-06	21:25	107.19	29	27	18	21	19	23	13	34	37	38	-	-	-	-	17	17	19	22	20	22	21	23
	10-Nov-06	22:50	58.67	37	24	24	34	25	25	-	-	31	32	-	-	-	-	16	16	30	31	30	31	22	29
	20-Nov-06	24:50	22.35	42	38	30	27	24	23	31	27	17	17	-	-	31	24	18	18	33	20	33	20	24	24
	10-Dec-06	80:25	12	41	36	30	36	14	22	-	-	-	-	12	34	-	-	24	22	22	24	22	24	31	11
	25-Feb-07	23:15	22	38	22	39	21	28	29	45	32	19	18	-	-	37	24	23	26	35	21	35	21	34	25
	26-Feb-07	21:15	12.20	45	28	42	22	30	22	31	28	20	21	-	-	33	20	23	19	18	18	18	18	18	29
	28-Feb-07	43:10	73.00	28	13	33	16	48	20	53	28	10	11	-	-	37	24	27	30	25	18	25	18	17	26
	6-Mar-07	94:10	13.80	41	29	29	27	10	20	38	22	21	13	-	-	33	25	25	18	24	27	24	27	31	30
	11-Jul-07	54:30	11.4	-	-	-	-	-	-	-	-	-	-	30	24	-	-	-	-	-	-	-	-	24	24
	12-Aug-07	83:15	27.8	-	-	-	-	-	-	-	-	-	-	37	23	-	-	-	-	-	-	-	-	22	29
	16-Aug-07	27:15	35.4	-	-	-	-	-	-	-	-	-	-	34	23	-	-	-	-	-	-	-	-	-	-
	24-Oct-07	12:55	54.20	20	43	32	24	-	-	35	34	-	-	-	-	33	23	-	-	-	-	-	-	-	-

Note: (FF: First flush, SF: Second flush)

Table 5.3.5 % of pollutant mass load flushed in the first 25 %, and subsequent 25% portions of the runoff volume in CP4 & CP6

Site	DATE	Antecedent dry period(h)	Rainfall mm	TSS		TP		NO _x		POC		OP		TN		DOC		SiO ₂		TDP		TDN		NH ₃ -N	
				FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF
CP4	23-Apr-07	23:45	52.40	18	37	9	30	71	7	-	-	-	-	-	-	-	-	26	22	19	24	-	-	24	29
	2-May-07	21:35	24.60	12	32	16	26	35	15	-	-	21	25	13	26	-	-	20	25	22	31	17	26	19	25
	31-May-07	89:15	15.20	13	35	23	27	6	18	51	33	20	22	19	27	39	27	30	29	22	20	15	20	28	24
	16-Aug-07	26:00	115.60	38	21	24	25	21	23	36	29	-	-	31	25	21	23	23	20	23	26	21	23	32	22
CP6	20-Apr-07	50:30	4.80	29	16	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	23-Apr-07	23:45	17.00	42	30	37	30	38	27	-	-	23	32	-	-	-	-	23	20	22	33	-	-	28	21
	1-May-07	22:35	20.00	38	24	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	27-May-07	27:10	16.20	52	20	35	28	5	23	64	8	8	29	18	21	35	18	23	18	8	29	7	23	16	10
	31-May-07	87:35	26.20	26	34	24	33	11	30	24	23	11	29	14	29	26	20	19	20	13	29	12	29	8	10
	1-Jul-07	36:10	3.80	58	29	45	30	24	24	85	9	18	21	40	23	59	11	25	14	21	22	23	20	15	21
	16-Aug-07	24:30	206.60	30	31	36	23	41	19	40	24	40	19	42	21	25	22	30	24	39	24	39	20	34	26

Note: (FF: First flush, SF: Second flush)

Table 5.3.6 % of pollutant mass load flushed in the first 25 %, and subsequent 25% portions of the runoff volume in CP7

Site	DATE	Antecedent dry period(h)	Rainfall mm	TSS		TP		NO _x		POC		OP		TN		DOC		SiO ₂		TDP		TDN		NH ₃ -N	
				FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF	FF	SF
CP7	12-Apr-07	46:35	0.60	37	34	36	26	30	23	33	23	-	-	-	-	27	26	20	23	-	-	-	-	32	22
	23-Apr-07	20:50	41.20	29	29	26	46	32	36	-	-	-	-	31	33	-	-	23	19	-	-	30	35	30	19
	25-Apr-07	26:05	28.00	37	29	33	23	45	22	-	-	22	32	40	23	-	-	19	18	23	24	44	23	23	27
	1-May-07	22:50	5.80	32	31	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	3-May-07	26:05	7.20	36	26	20	20	18	17	31	8	-	-	23	21	38	21	20	24	-	-	22	20	26	20
	15-May-07	38:40	9.00	49	22	43	29	24	21	59	14	-	-	39	24	40	20	25	16	-	-	29	20	20	36
	26-Jul-07	65:10	5.60	54	24	38	29	6	34	64	11	-	-	42	19	48	16	21	16	2	11	10	28	24	21
	12-Aug-07	83:15	12.80	36	27	45	27	9	30	41	15	-	-	28	26	27	26	-	-	27	20	17	25	30	34
	23-Aug-07	34:05	29.40	30	26	46	18	20	31	84	7	-	-	48	26	47	18	29	15	-	-	36	28	49	37

Note: (FF: First flush, SF: Second flush)

Tables 5.3.3 to 5.23.6 show the mass load flushed in first 25% runoff volume, and a second flush determined by % of the total pollutant mass being delivered in any 25% of runoff volume beyond the first 25% of the storm volume. Tables 5.3.3 to 5.23.6 show that 50% or more mass load flushed in first 25% runoff volume occurs only during 1 event for NO_x , DOC and $\text{NH}_3\text{-N}$, 4 events for TSS, 7 events for POC, and no event for TP, OP, TN, SiO_2 , TDP and TDN. POC exhibited the highest probability of 85% in CP6 and 84% in CP7 for occurrence of the first flush. POC exhibited first flush in 25% of the examined storm event, TSS in 8%, NO_x , DOC and $\text{NH}_3\text{-N}$ in 3%.

In addition, a second flush can be determined by % of the total pollutant mass being delivered in any 25% of runoff volume beyond the first 25% of the storm volume. None of the storm events exhibited a second flush for all the parameters. Overall, the occurrences of first and second flush based on this determination were insignificant.

Using the definition of 30% mass in the first 25% runoff volume for first flush and the next 25% volume beyond the first 25% as second flush (Flint and Davis, 2007), the following results are observed:

TSS:

Tables 5.3.3 to 5.23.6 show all the storm events exhibited ‘first flush’ phenomena for TSS in each gauging station except for CP4. However, ‘second flush’ was exhibited less frequently in all gauging stations, except at CP4. From land use analysis, CP4 has the largest undeveloped area of about 93%, which could slow down TSS transfer to gauging station in the first 25% runoff volume. From these observations, it appears that the occurrence of the ‘first flush’ phenomena could be related to land use.

TP:

Generally, TP exhibited the first flush phenomena more commonly than second flush, except for CP4 where no first flush occurs. It is surprising that although CP4 has 23% of area with cemetery land use, TP shows no occurrence of first flush.

NO_x :

Overall, no predominant occurrence of first flush was observed for NO_x . First flush occurred in 2 events in CP1, 3 events in CP2, 2 events in CP4 and 4 events in CP7,

while second flush occurs less frequently at each gauging station.

POC:

POC exhibited 'first flush' at all the gauging stations. Although CP4 has the largest undeveloped area, POC exhibited significant first flush, which is different from the other nutrients.

TN and DOC:

TN and DOC exhibited 'first flush' more commonly at CP2 and CP7, which are high density residential areas. Furthermore, these parameters could also be affected by agriculture areas, which exhibit 50% 'first flush' occurrences in the examined events at CP6.

OP, SiO₂, TDP, TDN, NH₃-N:

Overall, no predominant occurrence of first flush was observed on these parameters. Especially for SiO₂, there was occurrence of first flush at each gauging station except CP6.

The relative strength of first flush occurrence from the current study is POC> TSS> TP> DOC> TN> NO_x> NH₃-N> TDN> OP> TDP> SiO₂. CP1, CP2 and CP7 possess common land use characteristics. The land uses include residential, reserve site and forested areas. The undeveloped areas are about 60%, 32% and 54% respectively. Comparing these 3 sites, first flush of all nutrients occurs less frequent in CP1, while the results for first flush for CP2 and CP7 are quite close. This shows that water quality in the Kranji Catchment is associated with land use patterns. Furthermore, comparison of all parameters in the first 25% of the runoff with antecedent dry period and rainfall depth, show insignificant correlations. Several investigations such as Granier et al (1990) Gupta and Saul (1996) and Lee et al (2001) also found no correlation was found between the first flush and ADWP. The qualitative effects of first flush for each parameter shows different behavior, so these conclusions are not generally valid for all type of parameters.

Chapter 6

Conclusions & Recommendations

6.1 Conclusions

1. Simulation results for the five sub-catchments (gauging stations CP1, CP2, CP4, CP6 and CP7) using the XP-SWMM model suggest that the model is capable of simulating the single-peak storm events well. However, the model does not simulate multiple-peak events consistently.
2. Direct runoffs generated by XP-SWMM for 2005-2007 period show that urbanization tends to increase the proportional direct runoff volume, resulting in a proportional decrease in the baseflow volume. It can be concluded that imperviousness significantly influences the runoff volume generation in a watershed, due to the fact that imperviousness could increase direct runoffs even in small storm events.
3. The results from XP-SWMM simulation show that the model is capable of producing good outcomes for continuous flow simulations, and is also highly efficient in the estimation of urban storm water runoff volumes.
4. Among the sub-catchments, simulated results suggested that the largest contribution of DWF occurs at CP4 (of 60%), which has the largest proportion of undeveloped and pervious area to total sub-catchment area. In contrast, the smallest contribution of DWF occurs at CP2 (of 24%), which has the largest proportion of residential land use to total sub-catchment area.
5. The runoff coefficients are found to be a function of land use and total rainfall. In comparing CP2 with CP4, the average runoff coefficient is about 3 times higher for CP2, which has the largest proportion of developed area, around 68% of mainly residential land use with high impervious land cover. In contrast, CP4, which has the largest proportional pervious areas, has the lowest runoff coefficient of 0.13.
6. The pollutant concentrations of the baseflow samples were used to examine for seasonal trends in the water quality of the baseflow, and correlations between water quality of baseflow and ADWP. No relationship could be developed between the

baseflow quality and ADWP, or any seasonal trends observed, as there is no apparent difference in water quality during monsoon or non-monsoon seasons.

7. EMC analyses of TN, TP, DOC and TSS revealed that storm flow water quality has significant correlation to agricultural land use.
8. The relationships between dry weather concentrations, storm flow EMC, land use, rainfall depth and ADWD were investigated. The results show that land use has greater impact on pollutant concentrations for the investigated quality parameters than ADWD or rainfall depth.
9. Comparisons of the unit area loading rates (flux loading rates) of dry weather flow and storm flow for each study site reveal that over 87% of TSS is contributed by storm flows in all the catchments. More than 65% of the P loading from each sub-catchments occur in WWF. As the rainfall depth in year 2007 was 36~45% higher compared to the previous two years, it can be seen that the total pollutant load was elevated by the higher total runoffs.
10. In general, the mass loading rates of TSS, TN, NO_x and TDN from different land uses are in the order of: agriculture > residential > undeveloped watershed. The mass loadings of SiO₂ for different land uses are in the order: residential > agriculture > undeveloped area.
11. The present study shows that the parameters which are likely to experience the first flush phenomenon are in the order: POC> TSS> TP> DOC> TN> NO_x> NH₃-N> TDN> OP> TDP> SiO₂. Comparisons of all the parameters' loadings in the first 25% of the runoff volume show little correlation with the antecedent dry period and rainfall depth.

6.2 Recommendations

Based on the results so far, the following further studies are recommended.

The findings were based on data collected from storm runoffs at the sampling sites during storm events. It was found that for CP7, the results obtained are not very consistent, and consequently yield regression equations which provide poor estimations of the pollutant concentrations and loadings. Further analyses of the data can be carried out to yield more reliable pollutant loading estimations.

The storm water quality data collected might not be complete, and could have missed storm water samples during the early stage of the event. If possible, the autosamplers could be adjusted to collect data during the early stage of a storm flow event. This will be useful for more reliable analysis of the first flush phenomenon.

The watershed land use data were based on findings of previous related studies, and updated using the latest street directory. More precise information on the watershed land use pattern is important. Further works can be carried out to gather more updated information on the land use and future planned land use for the watersheds.

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APPENDIX A

Calibration and Verification Events for XP-SWMM

CP1: Bricklands Sub-catchment

Figure A.1 Calibration Events for XP-SWMM at CP1

CP2: CCKAVE4 Sub-catchment

Figure A.2 Calibration Events for XP-SWMM at CP2

CP4: TG AIRBASE Sub-catchment

Figure A.3 Calibration Events for XP-SWMM at CP4

CP6: AMK Sub-catchment

Figure A.4 Calibration Events for XP-SWMM at CP6

CP7: Sg Pangsua Sub-catchment

Figure A.5 Calibration Events for XP-SWMM at CP7

Monthly Verification for XP-SWMM

Figure A.6
Verification of
SWMM on monthly
basis at CP1

Figure A.7
Verification of
SWMM on monthly
basis at CP2

Figure A.8
Verification of
SWMM on monthly
basis at CP4

Figure A.9
Verification of
SWMM on monthly
basis at CP6

Figure A.10
Verification of
SWMM on monthly
basis at CP7

Date

APPENDIX B

Simulation results for each study site (2005~2007)

Figure B.1 Simulated direct runoff, baseflow, total runoff hydrographs from year 2005 to 2007 in each study site

APPENDIX C

Dry Weather Flow Loads

The correlation between antecedent ADP and DWF load

Figure C.1 Relationship between dry weather concentration and ADWP

Figure C.2 TSS concentration during dry weather period for each study site

Figure C.3 POC concentration during dry weather period for each study site

Figure C.4 TP concentration during dry weather period for each study site

Figure C.5 TN concentration during dry weather period for each study site

Figure C.6 SiO₂ concentration during dry weather period for each study site

Figure C.7 DOC concentration during dry weather period for each study site

Figure C.8 NH₃-N concentration during dry weather period for each study site

Figure C.9 NO_x concentration during dry weather period for each study site

Figure C.12 TDP concentration during dry weather period for each study site

APPENDIX D

Water Quality Rating Curves

CP1: Bricklands Sub-catchment

Figure D.1 Water Quality Rating Curve between Total Runoff Rate and Loading Rate (CP1)

CP2: CCKAVE4 Sub-catchment

Figure D.2 Water Quality Rating Curve between Total Runoff Rate and Loading Rate (CP2)

CP4: TG AIRBASE Sub-catchment

Figure D.3 Water Quality Rating Curve between Total Runoff Rate and Loading Rate (CP4)

CP6: AMK Sub-catchment

Figure D.4 Water Quality Rating Curve between Total Runoff Rate and Loading Rate (CP6)

CP7: Sg Pangsua Sub-catchment

Figure D.5 Water Quality Rating Curve between Total Runoff Rate and Loading Rate (CP7)

CP1

CP2

CP4

CP6

CP7

Figure D.6 TP and total flow
log-log graph

CP1

CP2

CP4

CP6

CP7

Figure D.7 TSS and total flow
log-log graph

CP1

CP2

CP4

CP6

CP7

Figure D.8 SiO_2 and total flow log-log graph

APPENDIX E

Relationship between Rainfall Depth, Total Flow, TSS and Loading Rate on the Weekly Basis

Figure E.1 Relationship of rainfall depth with DOC on the weekly basis

Figure E.2 Relationship of rainfall depth with POC on the weekly basis

Figure E.3 Relationship of rainfall depth with TP on the weekly basis

Figure E.4 Relationship of rainfall depth with TN on the weekly basis

Figure E.5 Relationship of rainfall depth with SiO₂ on the weekly basis

Figure E.6 Relationship of rainfall depth with TN on the weekly basis

Figure E.7 Relationship of flow with DOC on the weekly basis

Figure E.8 Relationship of flow with POC on the weekly basis

Figure E.9 Relationship of flow with TP on the weekly basis

Figure E.10 Relationship of flow with TN on the weekly basis

Figure E.11 Relationship of flow with SiO₂ on the weekly basis

Figure E.12 Relationship of flow with TSS on the weekly basis

Figure E.13 Relationship of TSS with DOC on the weekly basis

Figure E.14 Relationship of TSS with POC on the weekly basis

Figure E.15 Relationship of TSS with TP on the weekly basis

Figure E.16 Relationship of TSS with TN on the weekly basis

APPENDIX F

Mass-Based First Flush

Figure F.1 Normalized mass loading versus runoff volume as a function of the elapsed time of the storm events in CP1

Figure F.2 Normalized mass loading versus runoff volume as a function of the elapsed time of the storm events in CP2

Figure F.3 Normalized mass loading versus runoff volume as a function of the elapsed time of the storm events in CP4

Figure F.4 Normalized mass loading versus runoff volume as a function of the elapsed time of the storm events in CP6

Figure F.5 Normalized mass loading versus runoff volume as a function of the elapsed time of the storm events in CP7

% of mass pollutant load flushed in the FF

Figure E.6 Relationship between FF and ADWP for CP1

Figure E.7 Relationship between FF and ADWP for CP2

% of mass pollutant load flushed in the FF

Figure E.8 Relationship between FF and ADWP for CP4

Figure E.9 Relationship between FF and ADWP for CP6

% of mass pollutant load flushed in the FF

Figure E.10 Relationship between FF and ADWP for CP7

ADWP (hr)

ADWP (hr)

Figure F.11 Relationship between FF and rainfall depth for CP1

Figure F.12 Relationship between FF and rainfall depth for CP2

Figure F.13 Relationship between FF and rainfall depth for CP4

Figure F.14 Relationship between FF and rainfall depth for CP6

Figure F.15 Relationship between FF and rainfall depth for CP7

Rainfall depth (mm)

Rainfall depth (mm)