Development of an Integrated Catchment-Stream Water Quality Model

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Dissertation
Submitted in Fulfillment of the Requirements for the Degree of Doctor of Philosophy

Faculty of Engineering and Industrial Sciences
Swinburne University of Technology
2012
Dedication

The thesis is dedicated to my parents for their unlimited support, love and motivation.
Abstract

Water quality models are extensively used for the estimation of water quality parameters of an aquatic environment, which is essential for the development of efficient pollution mitigation strategies. However, due to the lack of specific local information and poor understanding of the limitations of various estimation techniques and underlying physical parameters, modelling approaches are often subjected to producing gross errors. Moreover, the usual practice of water quality modelling is performed through separate models in isolation, which leads to inconsistencies and significantly biased results in the prediction of water quality parameters.

This research project presents the development of an integrated catchment-stream water quality model to be able to continuously simulate different water quality parameters, i.e. SS (suspended sediments), TN (total nitrogen) and TP (total phosphorus). The integrated model is comprised of two individual models, i.e. the catchment water quality model and the stream water quality model. The catchment water quality model estimates the amount of pollutants accumulated on catchment surfaces during the antecedent dry days, and their transportation with surface runoff into nearby waterways and receiving water bodies throughout storm events. The stream water quality model estimates the amount of transported pollutants through a particular stream reach.

Considering the time-area method, a rainfall-runoff model was developed. Water quality parameters were incorporated with the rainfall-runoff model, which represents the catchment water quality model. A stream water quality model was developed with the same water quality parameters as used in the catchment water quality model. Finally, the catchment water quality model and the stream water quality model were integrated for the continuous simulation of previously mentioned water quality parameters.

For calibration and validation of the model, different published data and reliable source data collected from Gold Coast City Council (GCCC) were used. The calibration results demonstrated the suitability of the developed model as a tool to help with water quality management issues. Sensitivity analysis of the model parameters was performed to assess the most sensitive parameter and to enhance understanding of the modeller’s knowledge about the adopted modelling approach.
The major advantage of the developed model is the continuous simulation of water quality parameters associated with surface runoff during any rainfall event using an integrated modelling approach. The preparation process of the input data for the model is simple. The capability of the model to simulate surface runoff and pollutant loads from a wide range of rainfall intensities make the integrated model useful in assessing the impact of stormwater pollution flowing into waterways and receiving water bodies and to design effective stormwater treatment measures.
Keywords

Integrated model, continuous simulation, suspended solids/sediments, total nitrogen, total phosphorus, calibration, parameter estimation, sensitivity analysis.
Acknowledgements

Infinite thanks to Almighty Allah, the Creator, the Beneficent, the Merciful, who made it possible to complete successfully the Doctoral program within the stipulated time.

The author would like to express his deepest gratitude to his parents for their devotion, love and sacrifice which continues to encourage him, it has always done.

The author wishes to express his profound gratitude to his principle supervisor Dr. Monzur Alam Imteaz for introducing the subject of integrating modelling technique, and for his guidance, continual encouragement, thought provoking criticism, remarkable patience and professional advice made throughout the study. His ability to quick understanding of a problem and providing suggestions for a solution was very helpful at various points throughout the PhD project. Otherwise, this research would not have been possible.

Special thanks are also given to author’s associate supervisors A/Prof. Arul Arulrajah and Dr. Shirley Gato-Trinidad for their advices during the research. The author is highly grateful to the Faculty of Engineering and Industrial Sciences, Swinburne University of Technology for providing financial support during the candidature, which enables him to pursue Doctoral study at the University. This research was generously supported by a Swinburne University Postgraduate Research Award (SUPRA).

The author would like to express his appreciation to his fellow colleagues at Centre for Sustainable Infrastructure (CSI) for the support that received. The author’s appreciation is further extended to his fellow researcher Mr. Md. Imrul Hassan for his assistance in developing the computer code.

The research would not be possible without the support received from external organisation. The support received from the Gold Coast City Council (GCCC) in providing essential data is gratefully acknowledged.

Finally, the author would like to express his gratitude to his brothers, sisters, relatives and friends for the encouragement that received.
Author’s Declaration

The candidate hereby declares that the thesis entitled “Development of an Integrated Catchment-Stream Water Quality Model”, submitted in fulfilment of the requirements for the Degree of Doctor of Philosophy in the Faculty of Engineering and Industrial Sciences, Swinburne University of Technology:

➢ is candidate’s own work and has not been submitted previously, in whole or in part, in respect of any other academic degree or diploma
➢ to the best of candidate’s knowledge and belief, it contains no material previously published or written by another person except where due reference is made in the text of the thesis
➢ has been carried out during the period from August 2008 to March 2012 under the supervision of Dr. Monzur Alam Imteaz, A/Prof. Arul Arulrajah and Dr. Shirley Gato-Trinidad

__________________________

Iqbal Hossain

20 July 2012
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<th>Description</th>
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<tbody>
<tr>
<td>$A_c$</td>
<td>Contributing area of catchment</td>
</tr>
<tr>
<td>$A_{CF}$</td>
<td>Area of streamflow</td>
</tr>
<tr>
<td>$A_{CL}$</td>
<td>Parameter for exponential decreasing continuing loss</td>
</tr>
<tr>
<td>$A_D$</td>
<td>Dimensionless catchment area</td>
</tr>
<tr>
<td>$ARR$</td>
<td>Australian Rainfall and Runoff</td>
</tr>
<tr>
<td>$A_{sub}$</td>
<td>The area between two consecutive isochrones</td>
</tr>
<tr>
<td>$A_T$</td>
<td>Total area of a particular catchment</td>
</tr>
<tr>
<td>$A_x$</td>
<td>Cross sectional area of flow</td>
</tr>
<tr>
<td>$a$</td>
<td>Coefficient for stage-discharge curve</td>
</tr>
<tr>
<td>$a_{SD}$</td>
<td>Constant for stage-discharge characteristics of control sections</td>
</tr>
<tr>
<td>$B_{CL}$</td>
<td>Coefficient for exponential decreasing continuing loss</td>
</tr>
<tr>
<td>$BMPs$</td>
<td>Best Management Practices</td>
</tr>
<tr>
<td>$BOD$</td>
<td>Biochemical Oxygen Demand</td>
</tr>
<tr>
<td>$BOD_t$</td>
<td>Concentration of $BOD$ at time ‘$t$’</td>
</tr>
<tr>
<td>$BOD_{t-1}$</td>
<td>Concentration of $BOD$ at time $t-1$</td>
</tr>
<tr>
<td>$BoM$</td>
<td>Bureau of Meteorology</td>
</tr>
<tr>
<td>$B_T$</td>
<td>Top width of stream reach</td>
</tr>
<tr>
<td>$B_{ij,4}$</td>
<td>Mass of pollutant per unit area of a catchment surface</td>
</tr>
<tr>
<td>$(B_{ij})_D$</td>
<td>Amount of pollutant developed on a catchment surface during dry days</td>
</tr>
<tr>
<td>$B_{ij}(p,s)$</td>
<td>Accumulation of pollutant ‘$p$’ on catchment surface ‘$s$’</td>
</tr>
<tr>
<td>$(B_{ij})_R$</td>
<td>Amount of pollutant remained on land surface after the previous storm</td>
</tr>
<tr>
<td>$BUWO$</td>
<td>Build-up wash-off</td>
</tr>
<tr>
<td>$B_W$</td>
<td>Width of a particular stream reach</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$b$</td>
<td>Power of stage-discharge curve</td>
</tr>
<tr>
<td>$b_{SV}$</td>
<td>Mirror of the stage-volume characteristics of a stream reach section</td>
</tr>
<tr>
<td>$C_{CL}$</td>
<td>Coefficient for logarithmic continuing loss</td>
</tr>
<tr>
<td>$CFc$</td>
<td>Coefficient of the compensation factor</td>
</tr>
<tr>
<td>$CFe$</td>
<td>Exponent of the compensation factor</td>
</tr>
<tr>
<td>$CL$</td>
<td>Continuing loss</td>
</tr>
<tr>
<td>$CM0$</td>
<td>Muskingum routing coefficient</td>
</tr>
<tr>
<td>$CM1$</td>
<td>Muskingum routing coefficient</td>
</tr>
<tr>
<td>$CM2$</td>
<td>Muskingum routing coefficient</td>
</tr>
<tr>
<td>$CMC0$</td>
<td>Muskingum-Cunge routing coefficient</td>
</tr>
<tr>
<td>$CMC1$</td>
<td>Muskingum-Cunge routing coefficient</td>
</tr>
<tr>
<td>$CMC2$</td>
<td>Muskingum-Cunge routing coefficient</td>
</tr>
<tr>
<td>$CN$</td>
<td>Curve number</td>
</tr>
<tr>
<td>$CR$</td>
<td>Concentration of reactive pollutant</td>
</tr>
<tr>
<td>$C_i$</td>
<td>Overall Chezy coefficient</td>
</tr>
<tr>
<td>$C_i'$</td>
<td>Grain related Chezy coefficient</td>
</tr>
<tr>
<td>$Cu$</td>
<td>Unit conversion coefficient</td>
</tr>
<tr>
<td>$Cy$</td>
<td>Dimensionless runoff coefficient corresponding to return period ‘$y$’</td>
</tr>
<tr>
<td>$C_1(p,s)$</td>
<td>Maximum amount of pollutant that can be build-up on catchment surface</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>Coefficient for pollutant build-up parameter</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>Exponent for pollutant build-up parameter</td>
</tr>
<tr>
<td>$c_δ$</td>
<td>Reference level concentration of $SS$ at height ‘$δ$’</td>
</tr>
<tr>
<td>$(c_δ)t$</td>
<td>Reference level concentration of $SS$ at reference level ‘$δ$’ and time ‘$t$’</td>
</tr>
<tr>
<td>$c_k$</td>
<td>Kinematic wave celerity/ celerity of flood wave</td>
</tr>
<tr>
<td>$c_0$</td>
<td>Maximum volume concentration of $SS$</td>
</tr>
</tbody>
</table>
\[ \tilde{c} \] Concentration vector for different water quality parameters

\[ \tilde{c}_{ss} \] Depth average SS concentration vector

\( D \) Rates of SS deposition or erosion

\( DHI \) Danish Hydraulic Institute

\( DO \) Concentration of dissolved oxygen

\( DO_{sat} \) Concentration of dissolved oxygen at saturation level

\( D_s \) Representative particle diameter of SS

\( D_{sed} \) Decrease in sediment flow rate due to sedimentation

\( D_{ss} \) Deposition of SS per unit area of stream bottom

\( dS_{csv}/dt \) Change in storage within the channel reach with time ‘\( t \)’

\( D_{50} \) Median particle diameter of SS

\( D_{90} \) Sediment diameter for which 90% of the material is finer

\( EMC \) Event Mean Concentration

\( E_p \) Power of wash-off parameter

\( EPA \) Environmental Protection Agency

\( Evp \) Evaporation

\( E_1(p,s) \) Pollutant wash-off coefficient

\( E_2(p,s) \) Pollutant wash-off exponent

\( E_3(p,s) \) Coefficient for wash-off parameter

\( E_4(p,s) \) Exponent of the wash-off parameter

\( E_5(p,s) \) Wash-off exponent

\( e_b \) Bed-load work rate per unit stream power

\( e_s \) Suspension work rate per unit stream power

\( F_{DS} \) Dimensionless shape factor

\( (F_{DS})_t \) Dimensionless shape factor at time ‘\( t \)’
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>FF</td>
<td>First-flush</td>
</tr>
<tr>
<td>$F_{imp(s)}$</td>
<td>Impervious or pervious fraction of the land surface 's'</td>
</tr>
<tr>
<td>$f$</td>
<td>Rate of change of state variables</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Final infiltration capacity</td>
</tr>
<tr>
<td>$f(C_R)$</td>
<td>General reactive term for pollutant</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Infiltration capacity at time 't'</td>
</tr>
<tr>
<td>$f_0$</td>
<td>Initial infiltration capacity</td>
</tr>
<tr>
<td>GCCC</td>
<td>Gold Coast City Council</td>
</tr>
<tr>
<td>GIS</td>
<td>Geographical Information System</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity</td>
</tr>
<tr>
<td>$h$</td>
<td>Depth of flow</td>
</tr>
<tr>
<td>$h_t$</td>
<td>Depth of flow at time 't'</td>
</tr>
<tr>
<td>$I$</td>
<td>Average rainfall intensity for a storm</td>
</tr>
<tr>
<td>$I(s)$</td>
<td>Average rainfall intensity for a storm in land surface 's'</td>
</tr>
<tr>
<td>IL</td>
<td>Initial loss</td>
</tr>
<tr>
<td>IL-CL</td>
<td>Initial loss - continuing loss</td>
</tr>
<tr>
<td>IMPV</td>
<td>Imperviousness</td>
</tr>
<tr>
<td>$I_s$</td>
<td>Inflow rate to a stream reach at the upstream</td>
</tr>
<tr>
<td>$I_y$</td>
<td>Average rainfall intensity for a storm with return period 'y'</td>
</tr>
<tr>
<td>$(inSS)_t$</td>
<td>Concentration of inflow SS</td>
</tr>
<tr>
<td>$j$</td>
<td>Time step</td>
</tr>
<tr>
<td>$K_R$</td>
<td>Reaction parameter or rate coefficient</td>
</tr>
<tr>
<td>$K_{stor}$</td>
<td>Storage time constant for the reach</td>
</tr>
<tr>
<td>$k$</td>
<td>Von Karman constant</td>
</tr>
</tbody>
</table>

xx
\( k_B \)  
Pollutant build-up coefficient

\( k_{BOD} \)  
Oxidation of \( BOD \) or \( BOD \) decay rate

\( k_{BOD(Te)} \)  
Rate of oxidation of \( BOD \) at temperature \((Te)\) °C

\( k_{BOD(20)} \)  
Rate of oxidation of \( BOD \) at 20 °C temperature

\( k_{DEN} \)  
Denitrification coefficient of \( TN \)

\( k_{DEN(Te)} \)  
Denitrification coefficient of \( TN \) at \((Te)\) °C temperature

\( k_{DEN(20)} \)  
Denitrification coefficient of \( TN \) at 20 °C temperature

\( k_{DI} \)  
Rate of decrease in the infiltration capacity

\( k_L \)  
Linear pollutant build-up coefficient

\( k_p(p,s) \)  
Accumulation rate coefficient for pollutant ‘\( p\)’ on land surface ‘\( s\)’

\( k_{ref} \)  
Process rate at a reference temperature

\( k_s \)  
Roughness height

\( k_{sBOD} \)  
Rate of \( BOD \) loss due to settling

\( k_{sTN} \)  
Release rate of \( TN \) from sediment

\( k_{sTN(Te)} \)  
Release rate of \( TN \) from sediment at \((Te)\) °C temperature

\( k_{sTN(20)} \)  
Release rate of \( TN \) from sediment at 20 °C temperature

\( k_{sTP} \)  
Release rate of \( TP \) from sediment

\( k_{sTP(Te)} \)  
Release rate of \( TP \) from sediment at \((Te)\) °C temperature

\( k_{sTP(20)} \)  
Release rate of \( TP \) from sediment at 20 °C temperature

\( k_{susBOD} \)  
Re-suspension rate of \( BOD \)

\( k_{Te} \)  
Process rate parameter at \((Te)\) °C temperature

\( k_{TN} \)  
Coefficient for the mineralization of \( TN \) from \( BOD \)

\( k_{TN(Te)} \)  
Coefficient for mineralisation of \( TN \) from \( BOD \) at \((Te)\) °C temperature

\( k_{TN(20)} \)  
Coefficient for mineralisation of \( TN \) from \( BOD \) at 20 °C temperature

\( k_{TP} \)  
Coefficient for the mineralization of \( TP \) from \( BOD \)
$k_{TP(Te)}$  Mineralization coefficient of TP from BOD at temperature ($Te$) °C

$k_{TP(20)}$  Coefficient for the mineralization of TP from BOD at temperature 20 °C

$k_w$  Wash-off coefficient

$L$  Length of the main channel

$L_{impv}$  Rainfall loss from impervious surface area

$L_{pev}$  Rainfall loss from pervious surface area

$\Delta L$  Considered length of the channel reach

$MUSIC$  Model for Urban Stormwater Improvement Conceptualisation

$m_{SV}$  Mirror of the stage-volume characteristics of a stream reach section

$N$  Number of sub-reach of a stream reach

$n$  Manning’s roughness coefficient

$n_{SD}$  Constant for stage discharge characteristics of control sections

$ODE$  Ordinary Differential Equation

$O_s$  Outflow rate from stream reach section at the downstream

$PAHs$  Polycyclic aromatic hydrocarbons

$P_c$  Captured pollutant loads

$PEV$  Perviousness

$P_i$  Sum of all input pollutants

$P_{\text{max}}$  Threshold at which addition pollutant does not accumulate

$P_t$  Available pollutants accumulated at time ‘$t$’

$P_w$  Wetted perimeter

$P_m$  Model parameters

$P_{str}$  Available stream power per unit bed area

$Q_{in}$  Inflow rate to the reach at the upstream

$Q_{in,t}$  Inflow discharge at time ‘$t$’
\(Q_{out}\) Outflow rate from the reach at the downstream
\(Q_{out,t}\) Outflow discharge at time ‘t’
\(Q_p\) Peak discharge
\(Q_S\) Surface runoff
\(Q_{str}\) Streamflow rate
\((Q_{str})_t\) Streamflow rate at time ‘t’
\(Q_t\) Surface runoff at time ‘t’
\(Q_t(s)\) Surface runoff from the land surface ‘s’ at time ‘t’

**QUASAR** QUALity Simulation Along River Systems

\(q_A\) Surface runoff rate per unit catchment area
\(q_{A,t(s)}\) Runoff rate per unit area at time ‘t’ from the surface area ‘s’

\(q_L\) Lateral inflow

\(q_{ss,in}\) Suspended sediment flow rate per unit width
\((q_{ss,in})_t\) Suspended sediment flow rate per unit width at time ‘t’

\(q_{ss,tr}\) Transport of suspended sediment per unit width
\((q_{ss,tr})_t\) Transport of suspended sediment per unit width at time ‘t’

\(q_w\) Streamflow rate per unit width of stream reach
\((q_w)_t\) Streamflow rate per unit width of stream reach at time ‘t’

\(R_a\) Rainfall

\(R_D\) Depth of runoff

\(RE\) Rainfall excess

\(R_{ep}\) Particle Reynolds number

\(R_R\) Average runoff rate for a particular storm event

\(R_S\) Specific weight of sediment particles

\(S_{csv}\) Volume of channel storage
<table>
<thead>
<tr>
<th>SCS</th>
<th>Soil Conservation Service</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_e$</td>
<td>Equal area slope of the main stream projected to the catchment</td>
</tr>
<tr>
<td>$S_f$</td>
<td>Energy slope/ friction slope</td>
</tr>
<tr>
<td>\textit{SIMCAT}</td>
<td>SIMulation of CATchments</td>
</tr>
<tr>
<td>$S_0$</td>
<td>Slope of the channel bottom</td>
</tr>
<tr>
<td>$S_R$</td>
<td>Potential rainfall storage</td>
</tr>
<tr>
<td>SS</td>
<td>Suspended solids/sediments</td>
</tr>
<tr>
<td>$(SSD)_t$</td>
<td>Deposition of SS per unit area of stream bottom at time ‘$t$’</td>
</tr>
<tr>
<td>$(SSD,s)_t$</td>
<td>Deposition of SS per second at time ‘$t$’</td>
</tr>
<tr>
<td>SS$_{SET}$</td>
<td>Settetable SS</td>
</tr>
<tr>
<td>$S_{stor}$</td>
<td>Channel storage</td>
</tr>
<tr>
<td>\textit{SWCC}</td>
<td>Saltwater Creek Catchment</td>
</tr>
<tr>
<td>\textit{SWMM}</td>
<td>Storm Water Management Model</td>
</tr>
<tr>
<td>TA</td>
<td>Time-area</td>
</tr>
<tr>
<td>TAH</td>
<td>Time-area histogram</td>
</tr>
<tr>
<td>Tem</td>
<td>Water temperature</td>
</tr>
<tr>
<td>TKN</td>
<td>Total kjeldahl nitrogen</td>
</tr>
<tr>
<td>TN</td>
<td>Total nitrogen</td>
</tr>
<tr>
<td>$TN_t$</td>
<td>Concentration of TN at time ‘$t$’</td>
</tr>
<tr>
<td>$TN_{t-1}$</td>
<td>Concentration of TN at time $t-1$</td>
</tr>
<tr>
<td>\textit{TOMCAT}</td>
<td>Temporal/Overall Model for CATchments</td>
</tr>
<tr>
<td>TP</td>
<td>Total phosphorus</td>
</tr>
<tr>
<td>TPH</td>
<td>Total petroleum hydrocarbons</td>
</tr>
<tr>
<td>$TP_t$</td>
<td>Concentration of TP at time ‘$t$’</td>
</tr>
<tr>
<td>$TP_{t-1}$</td>
<td>Concentration of TP at time $t-1$</td>
</tr>
</tbody>
</table>
$T_t$ Bed shear (transport stage) parameter

$t_{BS}$ Time since beginning of storm event

$t_c$ Time of concentration

$t_d$ Number of antecedent dry days

$t_{DC}$ Dimensionless time

$t_p(p,s)$ Half saturation constant, i.e. days to reach half of the maximum build-up

$(trSS)_t$ Transport rate of SS at time ‘$t$’

$t_{ud}$ Pollutants travel time from upstream to downstream of a stream reach

$\Delta t$ Travel time of a water parcel between isochrones

$\Delta t_{BW}$ Time increment for pollutant build-up and wash-off

$\Delta t_R$ Routing time interval

$u_f$ Average flow velocity

$u_s$ Transport velocity of suspended sediment

$u_x$ Depth average horizontal velocity component

$u_y$ Depth average longitudinal velocity component

$u_z$ Depth average vertical velocity component

$u_*$ Bed-shear velocity

$\bar{u}$ Depth average flow velocity

$\bar{u}_t$ Depth average flow velocity at time ‘$t$’

$V_{Ts}$ Surface runoff volume during the integration time interval

$V_{Ts(s)}$ Surface runoff volume from land surface ‘$s$’

$W$ Pollutant wash-off

$W_p$ Amount pollutant wash-off from a catchment surface

$W_t(p,s)$ Pollutant wash-off rate at time ‘$t$’ for pollutant ‘$p$’ from land surface ‘$s$’

$w_s$ Settling velocity or fall velocity of the particles
\( X_p \)  Relative weights of a given inflow to the outflow for a stream reach

\( \Delta x_{SR} \)  Sub-reach length of stream reach

\( x, y, z \)  Spatial coordinates

\( Z \)  Suspension parameter (number)

\( Z' \)  Modified suspension number

\( 1D \)  One dimensional

\( 2D \)  Two dimensional

\( 3D \)  Three dimensional

\( \alpha_u \)  Coefficient for uniform flow

\( \alpha_w \)  Pollutant wash-off rate constant

\( \beta \)  Ratio of sediment and fluid mixing coefficients

\( \beta_u \)  Power for uniform flow

\( \theta_{BOD} \)  Temperature correction factor for organic decay

\( \theta_{cr} \)  Particle mobility parameter at initiation of motion.

\( \theta_{DEN} \)  Temperature correction factor for denitrification coefficient

\( \theta_{ref} \)  Temperature correction factor of reactive pollutants

\( \theta_{sTN} \)  Temperature correction factor for sediment \( TN \) release

\( \theta_{sTP} \)  Temperature correction factor for sediment \( TP \) release

\( \theta_{TN} \)  Temperature multiplier for mineralisation of \( TN \) from \( BOD \)

\( \theta_{TP} \)  Temperature multiplier for mineralisation of \( TP \) from \( BOD \)

\( \delta \)  Height of reference level

\( \delta_{BOD} \)  Half saturation constant for the \( BOD \)

\( \delta_{CL} \)  Constant value of continuing rainfall loss

\( \delta_{Conv} \)  Conversion factor

\( \delta_t \)  Height of the reference level at time ‘\( t \)’
\( \delta_{TP} \)  Adjustment factor for sediment TP release

\( \varepsilon_x \)  Turbulent diffusion coefficient longitudinal direction

\( \varepsilon_y \)  Turbulent diffusion coefficient horizontal direction

\( \varepsilon_z \)  Turbulent diffusion coefficient vertical direction

\( \rho_s \)  Density of SS particles

\( \rho_w \)  Density of water

\( \varphi \)  Stratification correction factor

\( (\tau_b)^t \)  Current related effective bed-shear stress at time ‘t’

\( \tau_{b,cr} \)  Critical time-averaged bed shear stress according to Shields

\( (\tau_b)_t \)  Grain related bed shear stress at time ‘t’

\( \eta_t \)  Grain friction efficiency

\( \nu \)  Kinematic viscosity coefficient
Chapter 1
Introduction

1.1 Background
As the recognition of the concept ‘Sustainable Development’ is increasing throughout the world, understanding the adverse impact of water quality parameters is highly important for the protection and improvement of aquatic environments from the impact of pollution. Urban expansion, land development, agricultural activities, industrial/commercial activities, atmospheric fallout and other human activities alter the natural conditions of catchment surfaces and increase the accumulation of pollutant loads. During rainfall events, these accumulated pollutants dissolve and/or get transported into nearby waterways and receiving bodies (Bannerman et al. 1993; Novotny et al. 1985; Sartor et al. 1974).

Rainfall also brings the atmospheric pollutants to catchment surfaces, and dislodges the dissolved and suspended pollutant particles from both impervious and pervious surfaces (Zoppou 2001). During the initial period of a rainfall event, catchment surfaces get wet and most of the soluble pollutants begin to dissolve (Kibler 1982). At the same time, some of the pollutants of a catchment surface are loosened by the energy of falling raindrops (Egodawatta 2007). With an increase in rainfall, catchment surfaces become wet enough to have surface runoff, which transports the dissolved and suspended pollutants to downstream aquatic environments.

The natural system can sustain some pollutants without affecting the environment. When pollutant loads exceed the limit, there is environmental pollution. Pollutants are accrued cumulatively in aquatic environments due to poor stormwater quality. The long term exposure of pollutants above the standard damages the quality of an aquatic environment (CRC for Catchment Hydrology 2005). Therefore, aquatic ecosystems of catchments, streams and associated environments are significantly altered due to the accumulation of the water borne pollutants.
The changes in water quality parameters of an environment result in the degradation of the quality of receiving water bodies and aquatic habitats, including social, economic and environmental costs, with short and long term consequences. Therefore, the impact of water pollution is an increasing matter of concern amongst watershed management groups (Wong et al. 2000). However, the severity of the deterioration of an aquatic environment depends on the amount of pollutants transported from upstream catchments and the characteristics of receiving environments. Hence, the measurement of water quality parameters is required to protect and improve aquatic environments from the impact of pollution (Fletcher and Deletic 2007; Vaze and Chiew 2002). An accurate estimation of runoff and pollutant loads will help watershed management authorities to adopt proper impact mitigation strategies. Inaccurate determination of pollutant loads can lead to the design of undersized and ineffective measures, or oversized measures with the excessive capital costs and the maintenance requirements.

1.2 Statement of the Problem

The estimation of water quality parameters from direct field measurement is costly, time consuming and sometimes impossible. Therefore, to address the widespread degradation of aquatic environments, watershed management authorities need appropriate modelling techniques so that they can meet the legislative requirements and societal expectations of sustainable water resources. Computer simulation and numerical models are the best ways for the hydrodynamics studies and for the control of water quality parameters. Models are essential and powerful tools, which can influence decision makers for the implementation of proper management strategies. Modelling techniques also help to integrate scientific understandings of the impact of management changes, and to give broader and long term perspectives about management interventions.

Water quality modelling is of crucial importance for the assessment of the physical, chemical and biological changes in water bodies. Mathematical approaches of water quality models have become prevalent over recent years. Different water quality models, ranging from the detailed physical to the simplified-conceptual are widely available for the simulation of various water quality parameters. However, the application of an appropriate modelling approach depends on the research goal and data availability.
The usual practices of water quality investigations are performed through separate models in isolation. This can lead to the inconsistencies and significantly biased prediction results. However, due to the lack of in-depth knowledge on the pollutant processes and the lack of data, limited attention has been given to develop integrated water quality model. On the other hand, the adoption of a continuous simulation approach is recommended in water quality modelling literature (CRC for Catchment Hydrology 2005). Although numerous water quality models have been developed since 1970, including some probabilistic models, there has been little effort in the development of a continuous modelling approach.

In Australia, water quality models are used for the estimation of different water quality parameters to provide information about the pollutant processes to watershed management groups. However, most of the models developed are based on urban areas and they estimate event based pollutant loads. Although Parker et al. (2002) stated that the integrated modelling approach should be developed, no such model that considers the integration of watershed basins has been developed. These lead to a serious gap for the continuous simulation of different water quality parameters through the integrated modelling approach. Hence, there is a necessity to develop an integrated process, which underpins the continuous simulation of various non-point source water quality parameters.

Proper estimation of pollutant parameters through integrated model will help authorities to control the transportation of pollutant loads into receiving water bodies by guiding them for the implementation of appropriate management options (Vaze and Chiew 2004). Urban environments could benefit greatly from an accurate prediction of water quality parameters by the integrated model. Integrated water quality management is only possible when the impact of pollutants to an aquatic ecosystem can be predicted quantitatively by means of an integrated water quality model (Rauch et al. 1998). The integrated approach recognises the complexity of the natural systems and their interaction. The key purpose of the integrated modelling approach is the evaluation of measures to improve the operation of a system (Rauch et al. 2002). This approach places emphasis on all aspects of water quality, including physical and chemical quality, habitat quality and biodiversity (Laenen and Dunnette 1997).
1.3 Aims and Objectives

It is understood that the application of an integrated model could eliminate the inconsistent results produced by the isolated models. Therefore, the primary objective of this research study was the development of an integrated catchment-stream water quality model for the continuous simulation of different water quality parameters. The goal was to develop an effective water quality prediction tool, which will help in the analysis, improvement and/or update of existing best management practices (BMPs). The knowledge on isolated water quality models was extended to understand the physio-chemical processes of the integrated modelling approach. Consequently, the major aims of the study were to:

- develop a detailed understanding of catchment water quality model for the prediction of stormwater quality pollutants, which are transported from upstream catchments
- refine and develop techniques for a stream water quality model that incorporate the stream hydraulics and stream water quality processes
- the integration of the developed catchment water quality model and stream water quality model to be able to continuously simulate different water quality parameters

The integrated model will help watershed management authorities by enabling them to implement economically viable and effective management design, and mitigation strategies to protect aquatic environments from the impact of pollution.

1.4 Research Hypothesis

The research study concentrated on the two major hypotheses:

- An integrated modelling approach can be adopted to enable the estimation of water quality parameters in order to continuously simulate different water quality parameters, which will provide reliable guidance to decision makers when implementing management strategies.
- The hydraulic radius of a particular stream reach can be calculated by introducing a compensation factor to enable the accurate estimation of stream water quantity and pollutant loads.
1.5 Research Scope

The research focused on the development of an integrated catchment-stream water quality model to be used for the prediction of different water quality parameters. In each stage of the model development, both water quantity and quality processes were incorporated. There were some constraints in the scope of the developed integrated model. The important issues in relation to this research can be described as follows for some practical reasons:

- The application of this model’s parameters was confined only to Gold Coast area. This limits the model outcomes in terms of regional and climatic parameters. In order to use the model in other areas, it must be calibrated and validated according to the characteristics of the selected regions. However, the general knowledge developed is applicable outside the regional and climatic conditions of Gold Coast.

- The experimental data which was used for the estimation of the catchment water quality model’s parameters was collected only from residential urban areas. This limits the wider applicability of some research outcomes, where land-use is a significant variable. However, the general knowledge developed is applicable for other land-uses as well.

- The research was confined only to the simulation of water quality parameters suspended solids/sediment (SS), total nitrogen (TN) and total phosphorus (TP). These three parameters are the most commonly used indicators of the quality of water and the impact of pollution into receiving water bodies (Vaze and Chiew 2004).

- It was considered that at the very beginning of the simulation, there was no available pollutant on the catchment surfaces.

- The study did not consider the spatial variability of rainfall events, i.e. rainfall was assumed to happen uniformly over entire catchment surface areas.

- The cohesive properties of sediment particles were not considered in this research project.

- Due to the scarcity of data, the model was calibrated with limited number of field observation data.
1.6 Outline of the Thesis

The thesis consists of seven chapters. An outline of the thesis is as follows:

Chapter 1: Introduction

Chapter 1 introduces background information about the topic of this thesis, its aims and objectives, research scope and provides an overview of the subsequent chapters.

Chapter 2: Literature Review

Chapter 2 of the thesis reviews literature on water quality modelling of catchments and streams. This chapter describes water quality research and identifies the existing knowledge gaps. The chapter starts with a literature review about the different categories of rainfall losses and their consideration in catchment water quality models. Moreover, the chapter describes the primary water quality parameters and their sources of the surrounding environments. In addition, this chapter presents a review of hydrology, hydraulics and water quality modelling approaches. The capabilities and limitations of the current modelling approaches are also discussed in this chapter. Finally, the chapter includes a review of the integrated modelling approach on water quality research.

Chapter 3: Model Development

Chapter 3 presents the developed technique adopted for the construction of this integrated model for the continuous simulation of SS, TN and TP. The chapter begins with the processes used to develop the deterministic catchment water quality model, which incorporates catchment hydrology and water quality. Then the chapter describes methodologies employed in the development of the stream water quality model, including stream hydraulics and pollutants processes. Finally, the chapter describes the integration of the developed catchment water quality model and the stream water quality model.

Chapter 4: Data Collection and Study Catchment Description

This chapter describes both experimental and observed field data used for this research. Moreover, the chapter demonstrates the requirements of data for calibration and validation of the developed model. The chapter also focuses on the characteristics of the physical environment that influences the quality of water to data collection areas.
Chapter 5: Calibration and Parameters Estimations

Chapter 5 demonstrates the estimation of the parameters for the developed model. The chapter focuses on the use of the calibration procedure to estimate the model parameters. Calibration is shown separately for the catchment water quality model and for the stream water quality model. In each stage of the parameters estimation, both the water quantity calibration and the quality calibration are shown in this chapter.

Chapter 6: Sensitivity of the Model Parameters

Chapter 6 focuses on the identification of the sensitive parameters of the developed integrated model. The chapter starts with a description of the different sensitivity analysis approaches. Then the chapter describes sensitivity of the acceptable behaviour of the model. Finally, the chapter discusses sensitivities of the different model parameters to both the catchment water quality model and the stream water quality model.

Chapter 7: Conclusions and Recommendations

This chapter provides conclusions of this research study together with suggested recommendations for the future research.

Finally, references used throughout the thesis are listed.

Appendices A to C contain information additional to the main text.
2.1 Introduction

It is widely accepted that water quality parameters originating from catchment surfaces alter the quality of nearby waterways and receiving water bodies. During storm events, rainwater undergoes different categories of losses. As rainfall continues to meet all these losses, it produces surface runoff, which comes into contact with different types of physical, chemical and biological substances (natural and manmade), such as SS, nutrients, heavy metals, pathogens etc. of a catchment surface (US EPA 1983; Duncan 1999); and hence rainwater becomes polluted. The momentum associated with surface runoff also dislodges other contaminant-laden particles from catchment surfaces (Zoppou 2001). The dislodged and dissolved pollutants are subjected to transport with the movement of water, exchange with atmosphere and aquatic sediments, and deteriorate the quality of aquatic environments (Kibler 1982).

Therefore, the impact of water pollution is an increasing matter of concern amongst watershed managers. To mitigate the adverse impact of water quality parameters, it is essential to have efficient management initiatives and treatment designs (Egodawatta 2007). With the impact of water quality parameters on aquatic environments as a foregone conclusion, an accurate prediction of pollutant loads would enable watershed management authorities to develop more efficient impact mitigation strategies.

Many regulatory authorities from government to catchment management groups strive to implement water quality management strategies to mitigate the adverse impact of water quality parameters. However, the productivity and effectiveness of such initiatives strongly rely on the accuracy and reliability of water quality parameters measurements (Chiew and McMahon 1999). To achieve this, it may require the need to integrate a wide range of planning and discipline tools, including hydrology, hydraulics, land-use planning, landscape design, path of pollutants transportation, self-purification of streams and so many other factors.
However, the allocated resources for the management of water quality parameters are small in relation to what is required for the remediation. The intensive monitoring, analysis and direct estimation of these pollutants on a wide scale are labour intensive, time consuming and prohibitively expensive with the available limited public funds (Davis and Birch 2009). Consequently, clever management of aquatic ecosystems is essential with the allocated budgetary constraints. Hence, water quality models are used for the prediction of water borne pollutants of waterways and receiving water bodies.

Water quality models are developed primarily based on modelling approaches which replicate hydrologic, hydraulic and water quality processes (Zoppou 2001). However, proper understanding of the actual methods for the accumulation and transportation of pollutants is often lacking. The lack of knowledge on the primary pollutant processes and the lack of data make modelling approaches inherently difficult. Although numerous research studies on water quality modelling have been undertaken, unfortunately comprehensive studies on the integrated modelling approach is yet to appear in scientific literature.

This chapter of the thesis describes background information relating to water quality research. The chapter starts with a literature review on identifying different rainfall losses associated with catchments and their consideration in water quality models. The chapter further describes the primary water quality parameters, their sources and related influential variables in hydrologic and water quality regimes. Moreover, this chapter intends to provide a comprehensive review of water quantity and quality modelling approaches with particular focus on catchment water quality models and stream water quality models. Additionally, this chapter describes the issues relating to the integrated modelling approach to explore possible knowledge gaps.

### 2.2 Rainfall Losses

During storm events, all of the rainfall does not contribute to surface runoff to the catchment outlet (Daniels and Gilliam 1996). The portion of rainwater which does not contribute to surface runoff is called rainfall loss. More specifically, rainfall loss is that part of storm precipitation which does not appear as immediate runoff, i.e. surface runoff (Hill et al. 1998).
At the very beginning, rainwater is usually reduced by the evaporation (Evp) loss. This reduced rainfall creates runoff which is further reduced by infiltration of the soil surface. If the soil is not saturated, water does not flow to the downstream and the generated runoff infiltrate into the soil surface. Once the infiltration capacity of the soil surface is exceeded (i.e. there is enough rainfall to meet the soil infiltration capacity), surface depressions begin to fill in. If the magnitude of rainfall is high enough to meet all these losses, excess rainwater starts to flow to the downstream and consequently surface runoff occurs. Physical processes of rainfall losses are shown in Figure 2.1.

Rainfall loss can occur with both impervious and pervious surfaces of a particular catchment. Rainfall loss from impervious area is too small to have an appreciable effect on the runoff peak and volume (Kibler 1982). However, loss coefficients for pervious areas affect the runoff volume appreciably. Hence, the selection of an appropriate loss value is highly important in the rainfall based runoff calculation. A low loss value results in the over estimation of runoff and a high loss value results in the under estimation of runoff.
There are different types of rainfall losses that can occur from a particular catchment surface. Rahman et al. (2002a) identified causes for the common rainfall losses:

- Interception due to surface vegetation
- Depression storage (retention on the surface)
- Infiltration into the soil ground
- Evaporation
- Watershed leakage

### 2.2.1 Interception Loss

Interception loss refers to that portion of rainfall which is intercepted by trees and plant leaves, stems and buildings, and returned to the atmosphere by evaporation \((Evp)\) without reaching to the ground surface (Akan and Houghtalen 2003). Rainwater held by vegetation which does not fall on the ground surface also fall into this category of loss. The amount of interception loss depends on types, density and growth rate of vegetation, intensity and volume of rainfall, roughness of catchment surfaces and the seasons of the year (Novotny and Olem 1994).

### 2.2.2 Depression Storage

Depression storage is the amount of rainwater which gets trapped after a storm event due to the undulating earth surface. Rainwater reaching the ground surface fills surface depressions, forms puddles, ponding, or adds to the general wetness of the environment (Novotny and Olem 1994). This water becomes stagnant and does not contribute to surface runoff and either evaporates or percolates into soil zones.

### 2.2.3 Infiltration Loss

Infiltration is a process by which precipitation moves downward through the earth surface and replenishes soil moisture, recharges aquifers and ultimately supports streamflow during dry periods (Veissman and Lewis 2003). During storm events, a portion of rainwater percolates into the soil surface. Rainwater entering into the soil surface is called infiltrated water. The rate of this loss is higher at the beginning of a storm event and consequently decreases to a fairly steady rate as rainfall continues (Novotny and Olem 1994).
2.2.4 Evaporation Loss

Once there is rainfall there is evaporation (Evp) of rain water, depending upon the weather conditions and the characteristics of catchment surfaces of a particular environment. The Evp loss occurs from open water surfaces, e.g. reservoirs, lakes, ponds, streams or from the soil surface. If the weather condition is dry, there is more Evp than during the wet weather.

2.2.5 Watershed Leakage

Watershed leakage is also called transmission loss. This loss is due to groundwater movement from one basin to another or to the sea. Transmission loss occurs through the stream bed or banks.

2.3 Catchment Scale Losses

Proper estimation of rainfall losses from a particular catchment surface is a pre-requisite for an accurate estimation of surface runoff. Since the processes of rainfall loss are well defined only at a point, it is difficult to estimate a representative rainfall loss value over an entire catchment. The spatial variability in the topography, catchment characteristics and rainfall make it difficult to link the loss values to catchment characteristics (Hill et al. 1998).

To overcome the difficulty in the rainfall loss calculation, simplified lumped conceptual models are used. These models are widely accepted because of their simplicity and ability to approximate the catchment runoff behaviour. They describe how the loss properties of a particular catchment changes as it becomes wetter with rainwater. They combine different loss processes and treat them in a simplified fashion (Hill et al. 1998).

Some of the most frequently used rainfall loss calculation methods in Australia are:

- Constant fraction of rain
- Constant loss rate
- Initial loss-continuing loss
- Infiltration curve equation
- Proportion loss rate
- SCS curve number procedure
2.3.1 Constant Fraction of Rain

This method assumes that rainfall loss and runoff is a constant fraction of rainfall in each time period of a rainfall event. In this method, a fraction of rainfall is subtracted in each time to calculate rainfall excess. If saturated overland flow occurs from a fairly constant proportion of a catchment surface, this constant fraction loss method is the best approach for the design cases (IEAust 2001). For a significant rainfall event, the values of constant fraction rainfall loss and effective rainfall which contribute to surface runoff are shown in Figure 2.2.

![Figure 2.2: Constant fraction of rainfall loss](image)

2.3.2 Constant Loss Rate

This method of rainfall loss calculation assumes that a constant amount of rainfall does not contribute to surface runoff in each time period of a rainfall event. In this method, a constant rate of loss is subtracted from the designed storm event as shown in Figure 2.3. For the design cases, this method is suitable for large storms having significant runoff volume (IEAust 2001). However, the method is not suitable for storms producing low runoff volume.
2.3.3 Initial Loss-Continuing Loss

The initial loss-continuing loss (IL-CL) model is the most commonly used method in Australia (IEAust 2001; Hill et al. 1998). The IL-CL model is shown in Figure 2.4.
The method is similar to the constant loss rate except that there is no runoff assumed to occur until a given initial loss (IL) capacity is satisfied, regardless of rainfall rate. At the beginning of a storm event, IL starts prior to the commencement of any surface runoff, and throughout the remainder of that storm continuing loss (CL) occurs, which is the average rate of rainfall loss after the fulfilment of IL from surface runoff.

### 2.3.4 Infiltration Curve Equation

Infiltration loss is very complex and the actual evaluation of the infiltration capacity is a difficult task (Akan and Houghtalen 2003). However, considerable research on the determination of infiltration loss has taken place. The first attempt to describe infiltration loss was made by Horton in 1940, which is the best-known and most widely used infiltration equation (Kibler 1982). A typical infiltration curve of the Horton equation is shown in Figure 2.5.

![Infiltration Curve](image)

In determining the infiltration capacity at any time from the start of an adequate supply of rainfall, Horton derived Equation 2.1:

\[
f_t = f_c + (f_0 - f_c)e^{-k_{st}t}
\]  

(2.1)
Where, \( f_i \) is the infiltration capacity (mm/hr) at time \( 't' \), \( f_0 \) is the initial infiltration capacity (mm/hr), \( f_c \) is the final or equilibrium or constant infiltration capacity (mm/hr) and \( k_{DI} \) is the exponential decay constant representing the rate of decrease in the infiltration capacity.

Equation 2.1 is suitable for small catchments because the values of \( f_c \) and \( k_{DI} \) are dependent on soil types and vegetation. Larger catchments tend to be heterogeneous, and soil types and vegetation are not the same throughout an entire catchment (Ilahee 2005). In addition, difficulties in the determination of \( f_0 \) and \( k_{DI} \) restrict the use of Equation 2.1 (Veissman and Lewis 2003).

### 2.3.5 Proportion Loss Rate

Proportion loss is considered to be a (constant) fraction of rainfall after surface runoff has commenced. In this model, rainfall loss is assumed to be a fixed proportion of rainfall as shown in Figure 2.6. In Australia, the model is used in conjunction with the IL model (Ilahee 2005). However, for the widespread application of this model, further investigation is needed.

![Figure 2.6: Proportion loss](image)
2.3.6 SCS Curve Number Procedure

The Soil Conservation Service (SCS) method was developed by the US Soil Conservation Service in 1972. This empirical model for rainfall abstraction is based on the potential for the soil surface to absorb a certain amount of moisture. On the basis of field observations, empirical equations of the SCS method can be described as follows:

\[ S_R = \frac{1000}{CN} - 10 \text{ (inches)} \]  \hspace{1cm} (2.2)

\[ S_R = \frac{25400}{CN} - 254 \text{ (mm)} \]  \hspace{1cm} (2.3)

Where, \( S_R \) is the potential rainfall storage (mm or inches) and \( CN \) is the curve number.

The input parameter of this model is the SCS curve number which is defined in terms of the soil type, antecedent soil moisture conditions, land-use treatment and hydraulic conditions of catchment surfaces. The method has given fairly good results when tested in the United States. Figure 2.7 shows typical SSC curve for different catchment conditions.

Figure 2.7: Typical SCS curves (IEAust 2001)
2.4 Primary Water Quality Parameters

Water quality parameters have the significant impact upon the quality of surrounding environments and aquatic ecosystems. During the antecedent dry days ($t_d$) (i.e. the days without rain), pollutant loads are accumulated on catchment surfaces. Throughout storm events, rainfall encounters these contaminants from the time rainwater falls on the ground surface to the time it reaches into receiving water bodies (Tsihrintzis and Hamid 1997). Moreover, rainwater can be polluted before it reaches to the ground surface (Vazquez et al. 2003). It is difficult to prevent stormwater runoff from polluting environments because surface runoff can be contaminated almost anywhere that rain falls (CRC for Catchment Hydrology 2005). When surface runoff occurs, it washes away and transports water quality parameters and soil sediment on its path till finally they are discharged into receiving water bodies (Bannerman et al. 1993; Novotny et al. 1985; Sartor et al. 1974) and deteriorates the quality of surrounding environments.

Since stormwater pollution leads to the significant deterioration of surrounding environments, identification of the specific characteristics and types of pollutants is critically important. To address water quality concerns adequately with stormwater runoff, it is important to understand types of pollutants which are present as well as their potential impact into receiving water bodies (Adams and Papa 2000). The common water quality parameters which deteriorate the quality of an aquatic environment are:

- Litter
- Suspended sediments/solids
- Nutrients
- Heavy metals
- Hydrocarbons
- Total organic carbon

2.4.1 Litter

Litter is the most obvious component of stormwater pollution. Over the last 30 years, pollution of the environment from the export of litter has intensified due to the production of easily disposable, non-biodegradable packaging materials, and commercial and industrial items (Wong et al. 2000). Shaheen (1975) found that 20% of the weight of pollutants which accumulate on road surfaces is litter.
Litter can be categorised as two types, i.e. artificial litter and natural litter. Packaging materials (paper, glass, metal, plastics etc.) are artificial litter, which mainly originates from construction or demolition activities. Litter originating from vegetative materials is called natural litter. Most of the urban catchments produce a significantly high volume and mass of litter.

The significance of litter as a water quality polluting agent is comparatively low. However, litter contributes to the blockage of urban drains and causes the unsightly appearance of receiving waters. During rainfall events, litter is washed-off from catchment surfaces and drains into streams and creeks. They interfere with the quality of water aesthetically and threaten animals, plants and fishes which live into waterways and water bodies (MW-Web 2011). In addition, Rahman et al. (2002b) found that leaf litter forms a significant component of SS in forested catchments.

### 2.4.2 Suspended Sediments/Solids

The term suspended sediment/solid (SS) is used to describe one of the major water borne pollutants. It refers to the mass or concentration of inorganic and organic materials suspended in water which is detached and transported (eroded) from their original sites. It is well recognised that SS is the most prevalent component in the deterioration of the quality of water, aquatic environments and their ecosystems. Therefore, an in-depth knowledge of the SS characteristics in stormwater runoff is critical. A clear understanding of SS especially particulate pollutants will aid watershed management authorities developing strategies to reduce the export of pollutants to surface water (Vaze and Chiew 2004).

A large portion of stormwater pollutants originating from a particular catchment surface is particulate matters (Vaze and Chiew 2004). During a storm event, some of the particulate matters are dissolved into rainwater. However, because of raindrop induced energy, a significant amount of particulate pollutants are kept in suspension in the overland flow (Shaw et al. 2010; Egodawatta et al. 2007) and transported to nearby streams with surface runoff. Hence, the concentration of SS in stream water increases during a storm event.
Excessive levels of SS in water bodies have the significant deleterious impact on physical, chemical and biological properties of nearby waterways and receiving water bodies (Bilotta and Brazier 2008). The physical alterations of water quality due to SS are associated with the undesirable aesthetic effects of polluted water, the higher water treatment costs, reduced navigational facilities of streams, and decreased longevity of dams and reservoirs (Butcher et al. 1993). The deposition of SS can block pipes, change in the flow conditions in open channels and disrupt habitats of aquatic invertebrates and fishes (Duncan 1999). In addition, the deposition of sediment particles raises the stream bed, lakes and ponds, reduces the discharge capacity of streams, impacts upon navigational facilities; and hence causes floods during high rainfall events (Lawrence and Atkinson 1998). The recreational uses of water, such as boating and swimming may be reduced due to turbidity of water. Turbidity from SS also makes water cloudy or muddy which leads to the reduction in light penetration through the water column, temperature changes, and infilling of channels and reservoirs by their deposition. Sedimentation problems of natural rivers arise from river management when there is an inadequate prediction of the sediment behaviour (Ali et al. 2010).

The chemical alterations caused by SS into waterways and water bodies include release of contaminants, such as heavy metals, pesticides and nutrients (Russel et al. 1998; Miller 1997). Furthermore, SS with a high organic content undergoes anaerobic breakdown and depletes the level of oxygen during their decomposition. This produces a critical oxygen shortage, which can leads to fish kills during low-flow conditions (Ryan 1991). Moreover, large amounts of bioorganic matters and heavy metals are bounded to SS (Deletic 2001).

The biological effects of high level SS on different groups of organisms are different. Increase in SS during a storm event can have the ecotoxic effects to aquatic organisms (Rossi et al. 2006). The growth rate of photosynthetic organisms is reduced by the high concentration of SS (Akan and Houghtalen 2003; Adams and Papa 2000; Duncan 1999). Deposition of SS can clogs fish gills and reduces spawning, resulting in lower fish populations or a shifting of fish species. Decline of fishery resources and serious ecological degradation of an aquatic environment are the results of excessive SS concentration.
Significant amounts of other pollutants are also transported as solid-bound contaminants with surface runoff. Zoppou (2001) noted that some of the main water quality parameters adhere to SS particles and conveyed along with soluble pollutants by surface runoff. Zhou et al. (2003) found that SS are closely linked to environmental problems due to the role of sediment particles in adsorbing and transporting contaminants. Vaze and Chiew (2004) found that particulates of TP and TN in stormwater runoff are attached to the sediment particles between 11 and 300 µm. Wong et al. (2000) found significant amounts of inorganic pollutants in SS. Herngren et al. (2005); Akan and Hougalten (2003) and Sartor et al. (1974) found a strong correlation between SS and heavy metals, nutrients and hydrocarbons. Sartor and Boyd (1972) reported that 50% of heavy metals are adsorbed to the particle size less than 43 µm. They further found that one third to one half of algal nutrients is attached to that sediment size. As the correlation between SS and other pollutants are strong, the presence of SS in water indicates the presence of other pollutants, such as heavy metals, nutrients.

2.4.3 Nutrients

Nutrients are chemical compounds that play important roles in all forms of life (Wong et al. 2000). They are important to maintain the productivity of an agro-ecosystem. Since these elements are essential to the life processes of aquatic organisms, they are referred to as nutrients (Bowie et al. 1985). Without nutrients, a healthy ecosystem cannot be considered. However, the presence of an excess concentration of nutrients into water bodies is considered as pollution.

An excessive amount of nutrients in the water environment can cause the detrimental effects to the health of an aquatic ecosystem (Wong et al. 2000). The increased concentration of nutrients into surface waters accelerates the eutrophication process (Laenen and Dunnette 1997). Eutrophication is the enrichment of water bodies with the dissolved nutrients, which causes the depletion of available oxygen in water and disturbs the biological equilibrium in water ecosystems. Eutrophication in surface water (e.g. rivers, dams, lakes and estuaries) in combination with other physio-chemical factors (i.e. increased water inflow from rainfall events and stratification) lead to the enhanced aquatic plant growth which is often referred as algal bloom (Robson and Hamilton 2003; Byun et al. 2005).
Algal bloom ultimately results in the de-oxygenation of water, reduces aesthetics and increases toxicity and contributes to the loss of diversity to aquatic environments (Drewry et al. 2006). The excessive growth of algae also alters the appearance of water bodies affecting the colour, odour, turbidity and floating matter water. Decaying algae cause undesirable odours and affects the taste of drinking water.

The major nutrients are nitrogen, phosphorus, carbon, calcium, potassium and silicon (Bowie et al. 1985). There are also some other micronutrients, such as iron, manganese, sulphur, zinc, copper, cobalt and molybdenum, which are essential for aquatic plants. However, these micronutrients are usually present only in trace amounts and they meet the biochemical requirements of organisms. For example, carbon is not considered to be a limiting nutrient in an aquatic system. In most situations, carbon does not limit algal growth; and hence the carbon simulation can be omitted in water quality modelling. Silicon is a limiting nutrient only for diatoms. Therefore, the biogeochemical cycle of silicon is generally modelled only when diatoms are simulated as a separate phytoplankton group (Bowie et al. 1985).

From the water quality perspective, nutrients that play an important role in water quality deterioration are: total nitrogen and total phosphorus (Ballo et al. 2009; Carpenter et al. 1998). Algal growth is typically limited by either of these two nutrients (Bowie et al. 1985). Total nitrogen ($TN$) is the sum of total kjeldahl nitrogen ($TKN$) and oxidised nitrogen. On the other hand, $TKN$ is comprised of organic nitrogen and ammonia nitrogen; while oxidised nitrogen is comprised of nitrite nitrogen and nitrate nitrogen. Both nitrite and nitrate in drinking water contribute to methemoglobinemia illness or blue baby syndrome. Total phosphorus ($TP$) is the sum of the dissolved and particulate phosphorus which can be sub-divided into reactive, acid-hydrolysable and organically bound according to its chemical availability (Duncan 1999).

### 2.4.4 Heavy Metals

In a large number of water quality studies (e.g. Wong 2006; Adams and Papa 2000; Sartor and Boyd 1972), the concentration of heavy metals were recorded. Depending upon the prevailing redox and pH condition, heavy metals can be organic or inorganic or complex and usually exist in a variety of ionic species (Ball et al. 2000).
The common heavy metals found in stormwater runoff in urban areas are: Zinc (Zn), Lead (Pb), Copper (Cu), Cadmium (Cd), Mercury (Hg), Arsenic (As), Nickel (Ni) and Chromium (Cr). The presence of heavy metals in stormwater runoff is of concern because most of these metals are toxic to aquatic lives (Akan and Houghtalen 2003; Davis et al. 2001; Ball et al. 2000). Environmental toxicity is depended largely on their concentration in the solution. Unlike most other water borne pollutants, heavy metals cannot be chemically destroyed or integrated into the environment (Davis et al. 2001). They may be transported attached to sediments or in a soluble or dissolved form. Most heavy metals are cumulatively accrued in an aquatic environment rather than the instantaneous flux (Ball et al. 2000). They have a strong affinity to attach smaller particle sizes due to the capacity of absorbing fine solids because of their relatively larger surface area (Deletic and Orr 2005). The conventional pollutant removal processes have little impact in reducing heavy metals from stormwater runoff (Miguntanna 2009).

2.4.5 Hydrocarbons

Hydrocarbons are important water quality parameters especially in urban environments. Generally, hydrocarbons refer to organic compounds which contain carbon and hydrogen. The main types of hydrocarbons which degrade the quality of water are total petroleum hydrocarbons (TPH) and polycyclic aromatic hydrocarbons (PAHs), which form a sub-group within TPH (Miguntanna 2009).

Hydrocarbons are crucially important in water quality study because some of their compounds are toxic to aquatic environments (Gobel et al. 2007; Akan and Houghtalen 2003). Herngren et al. (2005) found that there is a strong correlation between heavy metals and hydrocarbons. Ball et al. (2000) noted that similar to heavy metals and nutrients, hydrocarbons in stormwater runoff are mostly associated with the sediment particles. According to Datry et al. (2003), most of the hydrocarbons are attached to particulate matters. Wong (2006) found that many organic hydrocarbons can persist in sediment for long periods and they are dangerous at very low concentration. As a result, benthic organisms are particularly vulnerable to these organic toxicants. Therefore, an increased concentration of hydrocarbons leads to increase the toxicity of waterways and receiving water bodies (Polkowska et al. 2001).
2.4.6 Organic Carbon

Organic carbon is considered to be another major water quality parameter in urban stormwater runoff. It reflects the level of natural organic substances or humic minerals in water samples. Organic carbon is the measure of all carbon atoms covalently bonded in organic molecules (Wong 2006).

The common impact of organic matter in water is the reduction in dissolved oxygen \((DO)\) due to microbial oxidation. Decomposition of organic matter depletes \(DO\) into receiving water and affects aquatic environments (Akan and Houghtalen 2003). Hence, excess organic carbon means excessive oxygen demanding materials into receiving water and causes significant damage to aquatic lives (Warren et al. 2003). Moreover, Lin and Chen (1998) found a positive linear relationship between organic matter and heavy metals. Furthermore, a substantial level of organic matter leads to anaerobic conditions resulting in fish kills, foul odours and discoloration. Organic carbon in sediment solution leads to the solubility enhancement effect (Warren et al. 2003). Solubility enhancement is the reduction of solid-solution partition coefficient which reduces the total sediment sorbed amount, thereby increasing the soluble fraction (Goonetilleke and Thomas 2003). In addition, Wu et al. (2004) noted that organic matter is the main sorbent of hydrophobic pesticides in water systems.

2.4.7 Pathogens

Every stream contains some microorganisms many of which are harmless. However, some of them can cause diseases to human which are called pathogens (MW-Web 2011). The common pathogens associated with the water quality degradation, include bacteria, viruses and protozoa (Miguntanna 2009). They are disease causing organisms and are sometimes responsible for epidemics. Pathogens cause some fatal waterborne diseases, such as diarrhoea, dysentery, hepatitis, cholera or typhoid fever (Wong 2006).

Sources of pathogens in streams and creeks include human and animal faecal wastes (MW-Web 2011). They enter via stormwater drains, sewer overflows, animals in catchments, poorly maintained septic tanks, failing decentralised wastewater treatment systems and illegal sanitary sewer cross-connections. Some of the pathogens (e.g. enteric viruses) have the ability to replicate themselves within hosts (Herngren 2005).
2.5 Sources of Water Quality Parameters

Water quality parameters can originate from a variety of sources on catchment surfaces. Knowledge of the sources of pollutants allows decisions to be made to target the reduction of pollutants and evaluating the changes in pollutant loadings due to the modification in land-use and development. Proper knowledge of pollutant sources helps to estimate the effects of water system configurations on contaminant flows and their control. Bohemen and De Laak (2003) noted that the most effective way to control the flow of water quality parameters is to control them at the source.

There are a range of natural factors (climate, topography, geomorphology, geology, soil type and vegetation community type) as well as land-use and land management practices which determine the quality of water (O’Reagain et al. 2005). However, human activities are the prime reason for the accumulation of pollutant loads on catchment surfaces (Hoorman et al. 2008).

Urban expansion and consequent anthropogenic activities, agricultural activities, fertilizer applications, deterioration of forests, any physical changes on catchment surfaces and other human activities alter the natural condition of an existing environment. Due to these natural and anthropogenic sources, non-point pollutant loads are accumulated during the antecedent dry periods. Failures in urban infrastructures, such as sewer infiltration, leachate from landfills and direct connection of sanitary sewers to stormwater drains also increase the accumulation of pollutant loads (CRC for Catchment Hydrology 2005).

Most of the pollutants are either dissolve or otherwise transported with surface runoff and eventually end up in receiving water bodies. Therefore, surface runoff from different types of land-use is enriched with different types of pollutants. Hence, stormwater runoff is considered to be the major problem into nearby waterways and receiving water bodies. Different researchers have identified diverse sources of water quality parameters which impacts upon the quality of an aquatic ecosystem. The categories of important natural pollutants are geological, hydrological and climatic. Usually, surface water is impaired by water quality parameters input from the both point and non-point sources.
2.5.1 Point-source and Non-point Sources Pollutants

Pollutant loads coming from an identifiable source are called point source pollutants; while pollutants coming from many unknown sources are called non-point source pollutants. Point source pollutants can be predicted, controlled and reduced by imposing regulations. However, the prediction of the non-point source pollutants are difficult; and hence non-point source pollution, such as runoff from agricultural land, industrial and highly urbanised areas have the profound impact on both surface runoff and groundwater (Deletic 2001). Today, after years of extensive research in this area, it is clear that non-point source pollution (also called diffuse source pollution) is a significant problem to surrounding water environments (Novotny and Olem 1994). The common non-point source pollutants are categorised as sediments, nutrients and sewage. They can be originated from (Goonetilleke and Thomas 2003):

- transportation activities
- industrial/commercial activities
- construction and demolition activities
- corrosion of materials
- vegetation
- soil erosion
- atmospheric fallout
- spills

2.5.2 Transportation Activities

Transportation activities are considered to be one of the major contributing sources of water quality parameters. According to Puckett (1995), 38% of the atmospheric nitrogen emission may come from transport activities. Transport related pollutants are mainly generated on street surfaces. Therefore, street surfaces are considered to have the profound impact on water quality parameters which are mainly generated from vehicles. Although highways and freeways represent only a minor portion of catchment imperviousness, they are significant sources of water quality pollution (Wong et al. 2000). A summary of the sources of street surface contaminants, which are directly related to anthropogenic and traffic related activities is presented in Table 2.1.
Table 2.1: Typical road runoff contaminants and their sources (Ball et al. 1998)

<table>
<thead>
<tr>
<th>Contaminants</th>
<th>Primary source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sediment</td>
<td>Pavement wear, vehicles, atmosphere, maintenance activities</td>
</tr>
<tr>
<td>TN, TP</td>
<td>Atmosphere, roadside fertiliser applications</td>
</tr>
<tr>
<td>Lead</td>
<td>Auto exhaust, tyre and bearing wear, lubricating oil and grease</td>
</tr>
<tr>
<td>Zinc</td>
<td>Tyre wear, motor oil, grease</td>
</tr>
<tr>
<td>Iron</td>
<td>Automobile rust, highway steel structures (e.g. guard rails), moving engine parts</td>
</tr>
<tr>
<td>Copper</td>
<td>Metal plating, bearing and brush wear, moving engine parts, brake lining wear, fungicides, insecticides, pesticides</td>
</tr>
<tr>
<td>Cadmium</td>
<td>Tyre wear, insecticide application</td>
</tr>
<tr>
<td>Chromium</td>
<td>Metal plating, moving parts, brake lining wear</td>
</tr>
<tr>
<td>Nickel</td>
<td>Diesel fuel and petrol exhaust, lubricating oil, metal plating, brush wear, brake lining wear, asphalt paving</td>
</tr>
<tr>
<td>Manganese</td>
<td>Moving engine parts, automobile exhaust</td>
</tr>
<tr>
<td>Cyanide</td>
<td>Deicing compounds</td>
</tr>
<tr>
<td>Sodium/Calcium Chloride</td>
<td>Deicing salts</td>
</tr>
<tr>
<td>Sulphate</td>
<td>Roadway surfaces, fuels, deicing salts</td>
</tr>
<tr>
<td>Petroleum hydrocarbons</td>
<td>Spill, leakages or blow-by of motor lubricants, anti-freeze and hydraulic fluids, asphalt surface leachate</td>
</tr>
<tr>
<td>Polychlorinated biphenyls (PCB)</td>
<td>Background atmospheric deposition, PCB catalysts in synthetic tyres, spraying of right-of-way</td>
</tr>
<tr>
<td>PAHs</td>
<td>Asphalt surface leachate</td>
</tr>
</tbody>
</table>

Numerous research studies have identified street surfaces as significant contributors of water quality parameters (Egodawatta and Goonetilleke 2008; Zafra et al. 2008; Drapper et al. 2000; Bannerman et al. 1993; Sartor and Boyd 1972). Hoffman et al. (1985) found that over 50% of the total annual pollutants input of SS, polycyclic aromatic hydrocarbons and heavy metals of the Pawtuxet River, USA originated from street surfaces. This is due to the direct and continuous anthropogenic activities on street surfaces.
However, the accumulation of water quality parameters on street surfaces varies widely depending on a wide range of factors. Gobel et al. (2007) noted that the concentration of pollutants from street surfaces vary with the traffic density, wind drift, duration and intensity of stormwater events, the duration of dry weather periods and the state of the traffic technology. Sartor and Boyd (1972) identified pavement conditions and pavement materials as important sources of water quality parameters on street surfaces. They also found that asphalt pavements contribute 80% more pollutant loadings than concrete surfaces. Furthermore, the accumulation of water quality parameters on a street surface depends on the location of traffic lights, road layout, pavement surface roughness and driver’s habits (Goonetilleke and Thomas 2003).

2.5.3 Industrial/Commercial Activities

Industrial and commercial activities have the reflective impact on the accumulation of a wide range of water quality parameters on catchment surfaces. Due to the wide range of industrial activities and chemical use heavy metals are especially found in the urban areas. Numerous researchers identified different categories of water quality parameters in industrial and commercial areas. For example, the studies of Ghafouri and Swain (2004); Lau and Stenstrom (2005) and Brezonik and Stadelmann (2002) found that industrial and commercial areas have the relatively high concentration of water quality pollutants compared to other land-uses. Sartor and Boyd (1972) found that the accumulation of TP in a catchment surface was 14.3 g/m² for commercial areas and 51.8 g/m² for industrial areas. In addition, Bannermann et al. (1993) found that streets and parking lots are the critical source areas for the generation of water pollutants in industrial and commercial areas.

The major water quality parameters from various industrial/commercial activities contain a significant amount of nutrients, heavy metals and chemical toxins (Lee and Bang 2000). In addition, hydrocarbons, dust and other chemicals are also related to these activities (US EPA 1993). The industrial sources of pollutants are exposed storage, loading and unloading, equipment, spills and leaks, industrial materials and waste products (Miguntanna 2009). Water quality parameters in commercial areas are generated mainly from motor fluids from parked cars, large parking lots, auto service stations, gas stations, shopping centres and restaurants.
Sartor and Boyd (1972) found the highest metal concentration in road sweepings from industrial areas. Kelly et al. (1996) noted that due to the extensive burning of fossil fuel, heavy metals are accumulated in industrial areas. In addition, Herngren et al. (2006) found the highest heavy metal loadings coincided with areas of the highest sediment loadings which occurred in commercial areas. Latimer et al. (1990) found that particulate pollutants in industrial sites have the highest concentration of hydrocarbons. Armitage and Rooseboom (2000) noted that industrial and commercial activities significantly influence the accumulation of litter. Puckett (1995) found that 53% of the atmospheric deposited nitrogen in the north-eastern states in the US came from large industries, such as coal and oil burning, and electric utilities. Therefore, areas with industrial or commercial activities are considered to be one of the most significant sources of water quality parameters (Pitt et al. 1995).

2.5.4 Construction and Demolition Activities

Construction and demolition activities have the potential to contribute a significant amount of water quality parameters. In urban areas, construction activities are the most hazardous for sediment generation (Jartun et al. 2008). During a relatively short period, construction sites can contribute more solids into receiving water bodies than that can be deposited naturally over several decades (Miguntanna 2009). This is primarily attributed due to the higher soil erosion from construction sites.

Construction activities contribute most sediment on a unit area basis of a particular catchment (Nelson and Booth 2002). After investigating different land-uses in the USA, Line et al. (2002) found that the highest sediment export rate came from a construction site which was 10 times more than from other sites. They also found the highest amount of annual $TN$ export from construction sites. In addition, Sonzogni et al. (1980) noted that urban construction sites contribute more phosphorus than any other land-use.

Wind transports on-site pollutants from construction sites which are then accumulated on roofs, front yards and roads. High traffic volume close to construction and demolition activities forces pollutants to be accumulated on curbs (Brinkmann 1985). However, pollutant loads vary considerably with the amount of construction, catchment area, management of the site and maintenance activities (Miguntanna 2009).
2.5.5 Vegetation

Stormwater runoff contains not only inorganic pollutants but also organic vegetative pollutants. Waste vegetative matter from tree leaves and other plant materials, such as pollen, bark, twigs and grass are potential contributors of both organic pollutants and nutrients as they break down in catchments and in waterways. Line et al. (1997) found a significant amount of nutrients where a large amount of organic waste was present. They further noted that the concentration of TKN is increased during the periods of pollen deposition. Novotny et al. (1985) noted that a mature tree can produce 15 to 25 kg of organic leaf residues which contain a significant amount of nutrients. In addition, Dorney (1986) found that TP in urban street leaves are similar to TP from trees in natural ecosystems. Therefore, Cowen and Lee (1973) emphasised on the leaf pickup to minimise phosphorus content in urban drainages runoff.

However, Allison et al. (1998) have questioned the importance of leaf litter as a potential source of nutrients in urban stormwater. They found that potential nutrients contribution from the leaf litter is two orders of magnitude smaller than the measured total nutrient loads. Their findings were based on the outcomes of an urban catchment located in an inner city suburb of south-east Australia. Their observations confirm that water quality pollutants are dictated by the site-specific factors.

2.5.6 Soil Erosion

Soil erosion is the major contributor of SS in stormwater runoff. Nelson and Booth (2002) noted that the ubiquitous source of SS is the erosion of landscape areas either from natural or anthropogenic activities. Erosion starts with the impact of a raindrop, which can vary from 1/16 inch to 1/4 inch in diameter with maximum speed of 20 mph at the ground surface. The subsequent collisions between raindrops and the earth surface break the soil particles into their components (sand, silt, and clay). Surface runoff transports these smaller particles of silt and clay to the downstream. The force of flowing water also detaches the soil particles and transports them into receiving water bodies. The adsorbed pollutants with soil particles settle in the stream bed and in flood plains which causes physical, chemical and biological harm into receiving water bodies (US EPA 1993).
The most common factors affecting erosion of soil include the intensity and duration of rainfall, frequency of storm, snow cover and antecedent rainfall (Sonzogni et al. 1980). Soil types, topography, vegetation and climatic conditions also have a significant influence on soil erosion. Any land practice that exposes soil to erosive forces of rainfall and runoff represent erosion and pollutant hazards. During the active construction period, protective vegetative cover is removed from construction sites and the unprotected soil is left exposed to rainfall (USGS 2000). The loss of vegetative cover from the ground surface allows raindrops to strike with its full energy, which increases soil erosion and leads to increase the concentration of SS into stormwater runoff. Additionally, stream bank erosion increases the annual solid loads in an urbanised catchment (Nelson and Booth 2002).

2.5.7 Corrosion of Materials

In areas where metal roofs dominate, corrosion is a significant source of water quality pollution. Different metal elements, for example, Copper (Cu), Zinc (Zn), Aluminium (Al), Lead (Pb) and other materials are used as roof coverings, gutters and down pipes. All these materials release heavy metals in stormwater runoff as a corrosion product (Gobel et al. 2007). Forster (1996) also found that the areas having appreciable coverage of metal roofs are more liable to have the accumulation of heavy metals on catchment surfaces. Similar results were obtained by Davis et al. (2001) in a comprehensive study of urban water quality parameter sources. Their study was based on a variety of building sides and roofs among other surfaces. Further study undertaken by Bannerman et al. (1993) and Pitt et al. (1995) demonstrated that runoff from a galvanised roof surface contains higher heavy metals concentration than runoff from street surfaces.

Acid rain and aggressive gases contribute to a significant corrosion of roofs, gutters, paints and other metal surfaces (Brinkmann 1985). Due to the low pH value of rainwater, the corrosion process is enhanced (Gobel et al. 2007). During storm events, the corroded particles are transported with surface runoff into waterways and receiving water bodies. However, the amount of corroded pollutants on surface runoff depends on many factors.
Brinkmann (1985) identified the most common factors responsible for the corrosion of metal surfaces:

- availability of corrodible materials
- the frequency and intensity of exposure to an aggressive environment
- the drying-rewetting frequency of the exposed surface
- the character and the structure of materials
- maintenance practices

2.5.8 Atmospheric Fallout

In certain regions, atmospheric deposition is recognised as a significant non-point source of water pollution. Pollutants from atmospheric fallout include fine particulate materials, sulphur and nitrous oxides, PAHs etc. (Bohemen and De Laak 2003). According to Hoffman et al. (1984), most of the PAHs in the estuarine and coastal sediments originate from atmospheric fallout. Airborne pollutants are deposited on the earth surface and worsen the quality of water and water environments (Polkowska et al. 2001). Wang and Li (2009) found that the main contribution to the accumulation of pollutants on a roof surface is atmospheric dry deposition rather than roof materials.

The sources of atmospheric deposition include industry, traffic, refineries, power plants and waste processing companies (Bohemen and De Laak 2003). Emissions from vehicles initially contribute to pollution of atmospheric environments but return to the earth surface due to atmospheric deposition and they pollute stormwater runoff. Puckett (1995) noted that the atmospheric nitrogen is one of the important sources of nutrients. Davis et al. (2001) found that a significant amount of heavy metals from roof surface runoff came from atmospheric deposition.

Deposition of atmospheric pollutants occurs in two ways, i.e. dry deposition and wet deposition (Ball et al. 2000). Dry deposition is the direct transfer of dust, aerosol and gas from the atmosphere to the earth surface; and wet deposition is due to rain, snow, fog, dew and frost which contain substances leached-out from the atmosphere (Gobel et al. 2007). These two methods of deposition are the most important sources of water quality pollution (Brinkmann 1985).
2.5.9 Spills

This category of pollutants is difficult to define quantitatively either in terms of volume or composition (Goonetilleke and Thomas 2003); and it is even difficult to predict the occurrence of spills accurately. However, spills are concentrated in parking lots and near traffic lights. The major source of spills is vehicular transport. Leakages of fuel, motor oils and lubricants occur everywhere on road surfaces (Brinkmann 1985). In addition, Sartor and Boyd (1972) noted that vehicular transports, building constructions and industrial activities are considerable sources of spills. The adverse impact of spills on water quality is reduced through good maintenance and management practices.

2.6 Roles of Mathematical Models

Modelling is a commonly used tool in research and management of environmental systems. The ability to use a mathematical model to simulate the behaviour of a system has allowed environmental researchers and managers to predict how different scenarios and stimuli affect various environmental systems. The complex relationships between waste loads from different sources (point or non-point) and the resulting water quality responses of receiving water bodies are best described with mathematical models (Deksissa et al. 2004). Therefore, there exists a wide range of water quality models to be used in managing water quantity and quality with respect to a variety of environmental impacts. These models can predict physical, chemical and biological processes occurring during transportation of water quality parameters (Perk 2006). According to Perk (2006), the purposes of water quality models development and applications are:

- to forecast travel time of pollutants
- to assess the past, present, or future human exposure to pollutants
- to predict the future conditions under various scenarios of the environmental changes and management strategies, which are relevant to support decisions in policymaking processes
- to evaluate the effectiveness of possible management actions
- to reduce water quality monitoring costs by replacing or supplementing expensive measurements by cheaper model predictions
- to gain improved understanding of mechanisms, which control pollutant processes especially related to research and developing settings
2.7 Water Quality Modelling Approaches

Since 1970 a number of water quality models have been developed for the simulation of different water quality parameters of catchments and streams. Some of these models were developed specifically for and individual area and geographic conditions. Each of the models was developed in a unique way with the simulation characteristics and properties that influence how its output should be interpreted. These models vary in terms of their complexity, considered approaches, the personnel and computational requirements, and data requirements for calibration and validation (Merritt et al. 2003; Charbeneau and Barrett 1998). In addition, Radwan et al. (2003) and Zoppou (2001) noted that the essential difference in modelling approaches is the information that can be obtained from models, the sophistication of the analysis performed and the simulation period.

Modelling approaches start from the very simple conceptual type to the complex data intensive models. The simple models are unable to determine the controlling processes of pollutants because they do not employ a sufficient number of processes (Cox 2003). However, data requirements of the complex models prohibit their broader applications to the practical problems (Eatherall et al. 1998). In addition, more complex models need more computation effort and more data resources (Phillips and Yu 2001; Zoppou 2001; Goonetilleke 1998). Snowling and Kramer (2001) also noted that complex models are generally very sensitive and therefore it is difficult to predict water quality parameters. The financial costs of a complex model further enhance the difficulty of their wider application.

According to Lindenschmidt (2006), the most complex model is not necessarily the most accurate. Increasing model complexity increases the number of degree of freedom (i.e. more parameters and variables) which increases model sensitivity. Therefore, over-parameterisation makes the calibration procedure difficult and reduces the predictive capability of models. The users often try to use the simple models rather than the complex ones because they are easier to calibrate and therefore reliable (Pearl 1978). However, oversimplification of the catchments and pollutants behaviour may leads to gross errors. On the other hand, the processes of different model parameters are interrelated to each other, and hence increase the difficulty in pollutants prediction.
Snowling and Kramer (2001) demonstrated a hypothetical relationship among model sensitivity, error and complexity. According to them, model sensitivity increases with increasing model complexity due to the large number of parameters and their interactions with state variables. However, more complex models are able to better simulate the reality with more processes included and fewer simplifying assumptions; and hence modelling error decreases. For any system, consequences of choosing a model for a given complexity are illustrated in Figure 2.8. If a model is too complex, the sensitivity will be large (too far to the right), and if a model is not complex enough, the error will be large (too far to the left) as shown in Figure 2.8.

Figure 2.8: Model complexity, error and sensitivity relationship
(Snowling and Kramer 2001)

Depending upon the physical processes, model algorithms describing these processes and data requirements, models fall into three main categories (Merritt et al. 2003):

- Empirical or statistical/deterministic model
- Conceptual model
- Physics-based model

There are also other types of mathematical models, which fall within any of the three categories.
2.7.1 Empirical Model

The empirical model type is generally the simplest of all modelling types. This type of model is developed based on the general observation data. The computational effort and data requirements for such models are usually less than for conceptual and physics-based models (Merritt et al. 2003). The parameters of the empirical model are obtained by the calibration procedure from the observed data. However, the empirical model type is criticised for employing unrealistic assumptions of the physics. The advantage of the empirical model is that it can be frequently used in complex models, as it can be implemented in situations with the limited data and parameter inputs.

2.7.2 Conceptual Model

The conceptual model type is developed by incorporating underlying transfer mechanisms and their dynamic behaviour. Any model developed based on Darcy’s or Newton’s law is considered to be a conceptual model. The parameters of a conceptual model are usually derived by the calibration procedure against the observed data, such as stream discharge and concentration measurements. Therefore, this type of models tends to suffer from problems of identifying their parameter values. Furthermore, like the empirical model type, the parameters of the conceptual model have limited physical interpretability due to the lack of the unique value. However, simpler conceptual models have fewer problems than more complex models (Merritt et al. 2003).

2.7.3 Physics based Model

The physics-based model involves sophisticated approaches and solutions, which are based on the fundamental physical equations. This model type is developed based on the known sciences to simulate the processes on the basis of systems. The standard equations used in such models are the conservation of mass and momentum for flow and the conservation of mass for sediment (Merritt et al. 2003). Theoretically, the parameters of this model are measurable and so are known. However, in practice, due to a large number of factors involved, these parameters are often calibrated against the observed data (Merritt et al. 2003). Nevertheless, there are too many parameters to be quantified, i.e. they tend to be affected by over-parameterisation.
The derivation of mathematical expressions describing individual processes in the physics based model is subject to numerous assumptions that may not be relevant in many real world situations. The governing processes of the physics-based model are derived at small scale and under very specific physical conditions; and hence this type of model is useful for the short term simulation (Redwan et al. 2003).

2.7.4 Lumped and Distributed Models

Models can be considered as lumped or distributed based on applied areas. Models developed without considering the spatial variability are lumped models; while models reflecting the spatial variability of the processes are called distributed models. Most of the urban runoff models are deterministically distributed (Nix 1994). The distributed approach is also applicable to sediment transport modelling (Merritt et al. 2003). However, data requirements of the distributed models are larger compared with the lumped model. If the estimation at the catchment outlet is sufficient, then the lumped model is sufficient. The choice of the lumped model or distributed model depends on the desired output and the nature of management interactions.

2.7.5 Event Based and Continuous Models

In terms of the rainfall-runoff simulation, models can be categorised as event based model and continuous model. The model used for the simulation of an individual storm event is called event based model (Alley and Smith 1981); while the model developed for the simulation of a catchment’s overall water balance for a long period is continuous model. Continuous models are more advantageous than event based models (Tan et al. 2005; Boughton and Droop 2003). However, for the design of stormwater infrastructures, event driven models are more appropriate (Zoppou 2001).

2.7.6 Deterministic and Stochastic Models

Depending upon the nature of the prediction, models can be divided into two types, i.e. deterministic model and stochastic model. Deterministic model usually attempts to simulate the actual physical process associated with runoff quantity and quality; while stochastic model focuses on the probabilistic nature of pollutant processes (Barbe et al. 1996).
If any of the model variables is regarded as random variable having a probability distribution, then the model is called stochastic. Otherwise, the model is deterministic. For the same input, a deterministic model will produce the identical results (Zoppou 2001). However, a stochastic model will always produce different model responses as one or more variables are selected from the random distribution.

2.8 Model Selection
Although various modelling techniques are well developed, the distinction between models is not well-defined (Zoppou 2001). Kronvang et al. (2009) presented the ensemble modelling results of nutrients loads for 17 European catchments. They could not find any single model to perform the best across all of the catchments. Each of the model types serves a specific purpose and a particular model type cannot be recommended as appropriate for all the situations. Models are likely to contain a mix of modules from each model category. For example, while the rainfall-runoff component of a water quality model may be the physics-based or conceptual, empirical relationships are used to model the pollutant processes.

The ultimate decision for each modelling exercise depends upon the needs of the users and the purpose of modelling endeavour (Snowling and Kramer 2001). However, model should be easily applicable and usable for planning and management by water authorities (Laenen and Dunnette 1997). The choice of appropriate model depends on many factors. Tan et al. (2005) identified two main factors that define model selection, i.e. the objectives of the study and the availability of data and resources. According to Merritt et al. (2003), for the practical application, selection of the most appropriate model requires the consideration of the following factors:

- the intended use and objectives of the model users
- suitability of the model to the local conditions
- data requirements of the model, including spatial and temporal variation of the model inputs and the output
- complexity of the model structure
- various components of the model structure
- the capability of the model including the accuracy, validity and underlying assumptions
In addition, Akan and Houghtalen (2003) identified some important factors which affect the choice of a particular model for a practical application:

- the model is widely accepted (by engineers, consultants and regulators)
- the model is inexpensive (written for personal computers with good user support)
- the model is user friendly (preferably using Windows environment)
- the model is flexible and robust
- the model is technologically advanced

2.9 Hydrologic Modelling Approaches

The evaluation of hydrologically induced mobilisation of water quality parameters from watershed areas has received considerable attention in the recent hydrological modelling studies (Rusjan et al. 2008; Withers and Jarvie 2008). Hydrologic modelling is the technique which is used to estimate the quantity of runoff from a particular catchment area after a successive rainfall event. Hydrologic models were first developed for natural or rural catchments. With the increasing demand for urban hydrologic models, these types of models are modified to handle the urban characteristics (Ahyerre et al. 1998). At present, the same hydrologic model is used for both rural and urban catchments or partly rural and partly urban.

It is widely recognised that if runoff cannot be estimated accurately, pollutant loads cannot be predicted reliably. This is due to the two reasons. Firstly, pollutant loads cannot be determined without having the estimated flows. Hence, hydrological models form the basis of water quality models (White 1989). Secondly, the procedures to mitigate water quantity and quality are often complementary. Therefore, most of the water quality models are developed based on watershed hydrology or closely related to simulate diffuse pollutant loadings (Novotny and Olem 1994).

Over recent years, a large number of hydrologic models (RORB, XP RAFTS etc.) were developed to compute catchment runoff only. However, there are a number of models (e.g. SWMM, MUSIC) which were developed to estimate both water quantity and quality. Most of the models are based on the empirical relationships among drainage area, time of concentration ($t_c$), rainfall and other factors. They start from the very simple which calculate only the peak discharge, to the complex which are capable of
estimating runoff hydrographs. However, all of the developed models cannot simulate the continuous generation of surface runoff from catchment surfaces. Most of the models developed are based on the event based simulation and they can simulate a hydrograph for a single rainfall event only. The main types of hydrological modelling procedures described in many hydrologic texts and in manuals (such as Australian Rainfall Runoff) are as follows:

- Rational method
- Unit hydrograph method
- Time-area routing method
- Kinematic wave routing
- Artificial storage routing

2.9.1 Rational Method

The first and most enduring hydrological method for the calculation of storm runoff is the rational method (Gribbin 2007). The rational formula is the simplest form of hydrologic model to estimate the peak discharge from a small watershed area. The method was developed by Kuichling in 1889 for the measurement of discharges of small drainage basins in urban areas. Since then it has become the most widely used method for the design of small rural and urban watersheds structures (Viessman and Lewis 2003).

For the design of very small hydraulic structures, derivation of a complete runoff hydrograph is time consuming and becomes expensive. Only the peak discharge is sufficient enough for the design of such structures. For these cases, the rational method is very effective. The major advantage of the rational method is its simplicity. Moreover, the method requires comparatively less data and resources to perform calculations. The method is suitable for small (<25 km$^2$) to medium (<100 km$^2$) catchments where there are limited data.

The rationale of this method is based on the concept that steady, uniform rainfall intensity causes surface runoff at its maximum rate when all parts of a watershed area are contributing to flow at a point of interest (Viessman and Lewis 2003). However, there are certain assumptions which limit the use of the methods.
Dayaratne (2000) identified the fundamental assumptions adopted in the rational method which can be described as follows:

- rainfall is uniform all over a watershed area for the same period of time
- the duration of rainfall is equal to the time of concentration
- the peak flow is a fraction of overall rainfall rather than rainfall excess
- rainfall-runoff response is linear

Mathematical expression of the rational method can be written as follows:

\[ Q_p = C_u C_y I_y A_F \]  \hspace{1cm} (2.4)

Where, \( Q_p \) is the peak discharge of a watershed area \((L^3/T)\), \( C_u \) is the unit conversion coefficient, \( C_y \) is the dimensionless runoff coefficient corresponding to the return period ‘\( y \)’, \( I_y \) is the average rainfall intensity \((L/T)\) for a storm with the return period ‘\( y \)’ and storm duration \( t_c \), and \( A_F \) is the total area of watershed \((L^2)\).

Although the formula is widely used, care should be taken in selecting the appropriate runoff coefficient. The specific limitations of the method are as follows:

- the method is capable of estimating only the peak discharge
- the choice of the runoff coefficient is difficult
- uniform rainfall is rarely experienced over an entire catchment
- return period of both rainfall and runoff would rarely agree
- the catchment \( t_c \) many not be known
- applicable to small catchments only

2.9.2 Unit Hydrograph

The concept of the unit hydrograph was first introduced by L.K. Sherman in 1932 (Gribbin 2007). The unit hydrograph of a watershed area is defined as the direct runoff hydrograph resulting from one unit volume of excess rainfall of constant intensity and uniformly distributed over a particular drainage area. A flood hydrograph can be derived from a given rainfall excess hyetograph using the models developed based on the unit hydrograph method. The unit hydrograph is catchment responses for a unit of excess rainfall. The exact shape of the unit hydrograph depends on the characteristics of a particular watershed area being considered.
Basic assumptions considered in the use of the unit hydrograph include the followings:

- the effective rainfall has a constant intensity within the effective duration
- the effective rainfall is uniformly distributed over a particular watershed area
- the time base of a direct runoff hydrograph resulting from an excess rainfall of a given duration is constant
- ordinates of all direct runoff hydrographs of a common time base are directly proportional to the total amount of direct runoff

For a particular watershed area, if the generalised pattern of a rainfall event is known, a unit hydrograph is constructed for that area. Once the unit hydrograph is constructed, then the total runoff hydrograph can be estimated for any rainfall amount by multiplying the number of unit rainfall excess by the unit hydrograph. The superposition principle is used to construct the total hydrograph for a rainfall event greater than the unit rainfall. To do so, the rainfall excess is divided into a number of components, each equal to the unit rainfall. Since each unit rainfall excess produces a unit hydrograph, all of the unit hydrographs are combined to estimate the total hydrograph for that rainfall event. Figure 2.9 shows the construction of a resultant hydrograph from two unit rainfall events which occurred one after the other. Each of the unit rainfall produces a unit hydrograph of equal magnitude.

Figure 2.9: Unit hydrograph constructed from multiple units of rainfall
However, the second unit hydrograph shown in Figure 2.9 is separated by a time value equal to the unit time. The total hydrograph is constructed by plotting each of its ordinates as the sum of ordinates of the two unit hydrographs at each point along the time axis. In this way, the total runoff hydrograph is developed for any rainfall pattern longer than the unit time.

2.9.3 Time-area Method

The time-area (TA) method of rainfall-runoff transformation is one of the most widely used and suitable methods of hydrologic watershed routing (Shokoohi 2008; Saghaian and Shokoohi 2006; Ponce 1989). During a rainfall event, the entire area of a catchment is not contributing to surface runoff at the same time. The point close to the catchment outlet contributes to surface runoff almost immediately after rainfall. After that the contributing area increases which leads to an increase in surface runoff. Runoff contribution from upstream parts of a watershed arrives at the later time, adding to the contributing runoff from nearer points until flow arrives from all points of the watershed (Viessman and Lewis 2003). Therefore, in watershed modelling, to account partial area contribution of surface runoff, the TA method was developed.

The TA method was developed in the 1940’s (Shokoohi 2008). The method utilises a convolution of rainfall excess hyetograph with a time-area histogram (TAH) representing the progressive area contributions within a catchment in set time increments (Ponce 1989). The method was developed without considering the storage effects. In the TA method, the entire area of a catchment is deemed to contribute surface runoff after the \( t_c \) which is the time taken for a drop of water to reach at the outlet from the furthest point of a particular catchment. A detailed description of the method can be found in any hydrological hand book, such as Viessman and Lewis (2003). The following is a short description of the method.

Figure 2.10 shows a catchment area which is divided into five sub-areas \( A_1, A_2, A_3, A_4 \) and \( A_5 \). A rainfall event of the six different intensities shown in Figure 2.11(a) occurs over the catchment. The time taken for surface runoff to reach from one sub-area to another is \( \Delta t \). At the beginning of a rainfall event, no part of the catchment contributes to surface runoff and the initial flow \( Q_0 \) is zero.
After the first time step $\Delta t$, only the sub-area $A_1$ is contributing surface runoff to the outlet. Thus the flow rate at the end of the first time step is calculated by $Q_1 = A_1(RE)_1$. Where, $(RE)_1$ is the average rainfall excess for the first time step. At the end of the second time step $2\Delta t$, sub-areas $A_1$ and $A_2$ both are contributing flow to the catchment outlet and the runoff rate is calculated by $Q_2 = A_1(RE)_2 + A_2(RE)_1$. Where, $(RE)_2$ is the average rainfall excess for the second time step. Similarly, at the end of the third time step $3\Delta t$, sub-areas $A_1$, $A_2$ and $A_3$ contribute to surface runoff and the flow rate is determined by $Q_3 = A_1(RE)_3 + A_2(RE)_2 + A_3(RE)_1$. Where, $(RE)_3$ is the average rainfall excess for the third time step. After the $t_c$, all of the sub-areas contribute to surface runoff. In this way the process continues until rain stops.
In the TA method, catchment shape is reflected in the TAH and runoff hydrograph. The method can account for the temporal variation of rainfall intensity. In this method, the unrealistic assumption made in the rational method of uniform rainfall intensity over an entire catchment area during the $t_c$ is avoided. However, for very short rainfall events, it produces erroneous results (Saghafian and Shokoohi 2006). Furthermore, the success of the TA method mainly depends on the precision of isochrones mapping. Isochrones are the lines of equal flow time to the point where discharge is required. The most sensitive component of the TA method is isochrones mapping which divides a particular catchment into a suitable number of sub-areas to develop the TAH.

### 2.9.4 Time-area Histogram

The time-area histogram (TAH) can be defined as the contribution of surface runoff from consecutive watershed sub-areas as a function of time (expressed as a proportion of $t_c$) (Saghafian et al. 2002). This histogram shows the volume and time variation of contributing areas to the flow for a constant intensity of rainfall event.

To develop TAH, the catchment’s $t_c$ is divided into a number of equal time intervals $\Delta t$ (Ponce 1989). The cumulative time at the end of each time interval is used to divide a catchment into zones delimited by isochrones lines. Catchment sub-areas delimited by isochrones which contribute to runoff at each time interval are measured and plotted in histogram, which is known as the TA diagram or the TAH as shown in Figure 2.11(b). However, the time interval of effective rainfall hyetograph should be equal to the time interval of the TAH.

### 2.9.5 Time of Concentration

The most common definition of the time of concentration ($t_c$) is the amount of time required for a parcel of water to flow from the most remote hydraulic point of a watershed area to its outlet (Gribbin 2007; Garg 2001; McCuen 1984). It can also be expressed as the time taken from the start of a rainfall event until 100% of a catchment is simultaneously contributing to direct runoff at the outlet (IEAust 2001). For deterministic models like the TA method, the critical duration of a rainfall event is the time of concentration.
The $t_c$ is considered as the most widely used time parameter to estimate the peak discharge of a particular watershed area (Fang et al. 2007). However, it is neither possible to say that the $t_c$ starts exactly from any specific point of a watershed area nor if it can be measured exactly. Therefore, the estimation of the exact travel time for a water parcel from a watershed area is quite impossible. Nevertheless, it is a concept to describe the time responses of watershed’s runoff in hydrologic analysis.

The true value of the $t_c$ for a watershed area is influenced by so many parameters, including length and slope of the flow path, topographic setting (land-use classification), the rainfall characteristics, conveyance medium, channel shape etc. (Fang et al. 2008; Jonathan and Nelson 2002). Since it is difficult to determine a complete flow path and the necessary parameters, many equations were developed from basin average parameters to simplify the estimation of the travel time, i.e. the $t_c$ (Jonathan and Nelson 2002).

### 2.9.6 Kinematic Wave Modelling

Although several mathematical models were developed for hydrologic modelling, the actual behaviour and routing of flow in an irregular and complex catchment surface is not well established (Egodawatta 2007). For the modelling of flow through such complex surfaces, there are only some approximate models commercially available. An example of such a model is the simplified kinematic wave equation, which is one of the most popular forms of mathematical model formulations. The equation is a simplified form of the Saint Venant equation.

If all the terms of the momentum equation except $S_0$ (bed slope) and $S_f$ (energy slope) are omitted in the Saint Venant equation, it is reduced to a uniform flow equation. A general form of resistant equation can be used as follows (Shokoohi 2008):

$$Q_{str} = \alpha_u A_x^{\beta_u}$$  \hspace{1cm} (2.5)

Where, $A_x$ is the cross sectional area of flow, $\alpha_u$ is the coefficient for uniform flow and $\beta_u$ is the power of uniform flow.

For Manning’s equation, $\alpha_u$ and $\beta_u$ are obtained as follows:
\[ \alpha_w = \frac{1}{n} \cdot \frac{S_j^{1/2}}{P_w^{3/2}}, \ \beta_w = \frac{5}{3} \] (2.6)

Where, ‘\( n \)’ is the Manning’s roughness coefficient and \( P_w \) is the wetted perimeter.

By inserting a resistant equation into the continuity equation, one can obtain (Shokoohi 2008):

\[ \frac{\partial h}{\partial t} + c_k \cdot \frac{\partial h}{\partial x} = RE, \quad \frac{\partial q_w}{\partial t} + c_k \cdot \frac{\partial q_w}{\partial x} = c_k(RE) \] (2.7)

Where, \( RE \) is the excess rainfall intensity, ‘\( h \)’ is the depth of flow, \( q_w \) is the discharge per unit width of a channel reach and \( c_k \) is the kinematic wave celerity.

Although kinematic models are simplified and relatively accurate for shallow flow depths and steep slopes, these models need more computational efforts to reach the level of accuracy available from the storage routing model and the TA routing model. In addition, catchment sub-division is complex for this modelling technique (Egodawatta 2007). These restrict the use of water models developed using the kinematic wave equation in Australia.

### 2.10 Stream Hydrology and Hydraulics

The study of stream hydrology and hydraulics is important for the prediction of streamflow and stream water pollutants, and for the future development of water resources engineering. Since decision makers typically employ dilution ratios to assess the expected water quality, modelling the quantity of flow is generally more important than modelling the quality (Shanahan et al. 1998).

Generally, the estimation of water quantity is determined from hydrograph routing. Hydrograph routing is a mathematical procedure used to predict the temporal and spatial variations of runoff quantity, as water travels from one section to another through a particular stream reach (Viessman and Lewis 2003). Flood forecasting, reservoir design, watershed simulation modelling and comprehensive flood control planning studies require some form of hydrograph routing techniques. Stream hydrograph routing techniques are categorised into two types (Seybert 2006): hydrologic routing and hydraulic routing.


2.10.1 Hydrologic Stream Routing

The first reference to routing a flood hydrograph from one stream station to another was derived by Graeff in 1883 (Viessman and Lewis 2003). The technique was based on the use of wave velocity and a rating curve of stage-discharge relationship. A particular stream is modelled as a single reach or divided into several sub-reaches. When more than one reach is used, the calculation of the furthest upstream section is done first. Then the calculation proceeds in the downstream direction making the outflow from one stream to the inflow for the next downstream reach.

For a particular stream reach, water entering into the reach must either exit from that reach, or cause a change in reach storage. The simplest form of routing equation relies on the conservation of mass and can be written as: inflow – outflow = change in storage (Ladson 2008). Mathematical expression is:

\[ I_s - O_s = \frac{dS_{csv}}{dt} \]  

Where, \( I_s \) is the inflow rate to a stream reach at the upstream section (m³/s), \( O_s \) is the outflow rate from a stream reach to the downstream section (m³/s), \( dS_{csv}/dt \) is the rate of change of storage volume within the stream reach at time ‘t’ and \( S_{csv} \) is the volume of reach storage.

The most popular hydrologic stream routing methods are: the Muskingum method and the Muskingum-Cunge method.

2.10.1.1 Muskingum Method

A popular hydrologic stream routing method is the Muskingum method. The method was developed by G.T. McCarthy (1938) using studies associated with the U.S. Army Corps of Engineers Muskingum Conservancy District Flood Control Project in Eastern Ohio (Seybert 2006). This method is used for the calculation of an outflow hydrograph at the downstream end of a stream reach for a given inflow hydrograph at the upstream end. The basic assumption for the Muskingum method is that the stage-discharge relationship is one-to-one and it is strictly applied (Cunge 1969). In addition, the cross sectional geometry is assumed to be constant throughout a reach section.
The simplified storage relationship in the Muskingum method uses the geometric shapes of a prism and wedge to represent reach storage capacity. The prism represents reach storage volume where flow is steady, i.e. the inflow is equal to the outflow. The wedge represents reach storage volume above the prism which is caused by a passing a flood wave (Seybert 2006).

The Muskingum method is a finite difference solution of the storage equation. The storage within a particular stream reach at a given time can be expressed as (Chow 1959):

\[
S_{\text{stor}} = \frac{b_{SV}}{a_{SD}} \left[ X_p \times I_S^{m_{SV}/n_{SD}} + (1 - X_p)O_S^{m_{SV}/n_{SD}} \right]
\]  

(2.9)

Where, \( S_{\text{stor}} \) is the channel storage \((m^3)\), \( a_{SD} \) and \( n_{SD} \) are the stage-discharge characteristics of a control section, \( b_{SV} \) and \( m_{SV} \) are the mirror of the stage-volume characteristics of reach section and \( X_p \) is the relative weight of the inflow and the outflow for a particular stream reach.

The method assumes that a linear relationship exists between \( S_{\text{stor}} \), \( I_S \) and \( O_S \) (Akan and Houghtalen 2003). Equation 2.9 produces a linear relationship if \( m_{SV}/n_{SD} = 1 \). Assuming \( b_{SV}/a_{SD} = K_{\text{stor}} \) one can get the following equation:

\[
S_{\text{stor}} = K_{\text{stor}} \left[ X_p \times I_S + (1 - X_p)O_S \right]
\]  

(2.10)

Where, \( K_{\text{stor}} \) is the storage time constant for a particular stream reach.

Equation 2.10 states that channel storage is a function of the channel inflow and outflow. The application of Equation 2.10 has shown that \( K_{\text{stor}} \) is usually reasonably close to the wave travel time through a stream reach (Viessman and Lewis 2003). The value of weighting factor \( X_p \) varies between 0 and 0.5 with an average value of about 0.2. The behaviour of a flood wave due to the changes in the values of \( X_p \) is readily apparent as shown in Figure 2.12. The figure shows that when \( X_p = 0.5 \), the result is a perfect translation of a flood wave.

Equation 2.10 is straightforward, if the values of \( K_{\text{stor}} \) and \( X_p \) are known. Using subscripts 1 and 2 to denote the beginning and the end of the routing time interval \( \Delta t_R \) one can obtain:
\[
\frac{I_{S1} + I_{S2}}{2} - \frac{O_{S1} + O_{S2}}{2} = \frac{S_{stor,2} - S_{stor,1}}{\Delta t_R}
\]
(2.11)

The change in reach storage during the routing interval is written from Equation 2.10:

\[
S_{stor,2} - S_{stor,1} = K_{stor} \left[ X_p \left( I_{S2} - I_{S1} \right) + \left( 1 - X_p \right) \left( O_{S2} - O_{S1} \right) \right]
\]
(2.12)

Substituting this into Equation 2.11 results in the Muskingum routing equation:

\[
O_{S2} = C_{M0} I_{S2} + C_{M1} I_{S1} + C_{M2} O_{S1}
\]
(2.13)

in which

\[
C_{M0} = \frac{-K_{stor} X_p + 0.5 \Delta t_R}{K_{stor} - K_{stor} X_p + 0.5 \Delta t_R}
\]
(2.14)

\[
C_{M1} = \frac{K_{stor} X_p + 0.5 \Delta t_R}{K_{stor} - K_{stor} X_p + 0.5 \Delta t_R}
\]
(2.15)

\[
C_{M2} = \frac{K_{stor} - K_{stor} X_p - 0.5 \Delta t_R}{K_{stor} - K_{stor} X_p + 0.5 \Delta t_R}
\]
(2.16)
It should be noted that $K_{\text{stor}}$ and $\Delta t_R$ must be in the same unit and $C_{M0}$, $C_{M1}$ and $C_{M2}$ are dimensionless Muskingum routing coefficients. The sum of the value of $C_{M0}$, $C_{M1}$ and $C_{M2}$ is unit:

$$C_{M0} + C_{M1} + C_{M2} = 1 \quad (2.17)$$

Since $I_{S1}$ and $I_{S2}$ are known from a given inflow hydrograph and $O_{S1}$ is known either from the initial conditions or from the previous time step computations at any given time interval, the only unknown parameter in routing Equation 2.13 is $O_{S2}$.

With the predetermined values of $K_{\text{stor}}$, $X_P$ and $\Delta t_R$, routing coefficients are computed and a general routing equation in the form of Equation 2.13 is created. The solution begins by identifying the inflow and outflow conditions in a particular stream reach section.

In the Muskingum method, it is assumed that water surface is a uniform unbroken surface profile between the upstream and downstream ends of a reach section. Moreover, the routing parameters $K_{\text{stor}}$ and $X_P$ are best derived from streamflow measurement and are not related to the channel characteristics (Akan and Houghtalen 2003; Viessman and Lewis 2003). In addition, the values of $K_{\text{stor}}$ and $X_P$ are presupposed to be constant throughout ranged flows. These limitations are overcome by the Musking-Cunge method.

### 2.10.1.2 Muskingum-Cunge Method

The Muskingum-Cunge method is one of the attempts to overcome the limitations of the Muskingum method. It is based on the continuity of mass equation within a channel reach. For the general use, Cunge (1969) blended the accuracy of the diffusion wave method for the simplicity of the Muskingum method. The input parameters, such as length, roughness, slope and geometry allow for the computation of routing parameters $K_{\text{stor}}$ and $X_P$ for each routing interval. It is classified as a hydrologic method, yet it gives the comparable results with hydraulic methods (Viessman and Lewis 2003). The Muskingum-Cunge method is also a well-known method used to compute pollutographs of stream reaches.
According to Cunge (1969), the finite difference form of the Muskingum equation becomes the diffusion wave equation, if the parameters of the both methods are appropriately related. From Equations 2.8 and 2.10, the Muskingum equation becomes:

\[
K_{\text{stor}} \cdot \frac{d}{dt} \left[ X_p I_S + (1 - X_p) O_S \right] = T_S - \bar{O}_S
\]  

(2.18)

Substituting \( Q_i \) for \( I_S \) and \( Q_{i+1} \) for \( O_S \), and rewriting in finite difference form, we can obtain:

\[
\frac{K_{\text{stor}}}{\Delta t_R} \left[ X_p Q_{i+1}^{i+1} + (1 - X_p) Q_{i+1}^{i+1} - X_p Q_i^{i} - (1 - X_p) Q_{i+1}^{i} \right] = \frac{1}{2} \left( Q_i^{i+1} - Q_{i+1}^{i+1} + Q_i^{i} - Q_{i+1}^{i} \right)
\]  

(2.19)

The equation to be used for routing a flood wave is obtained from Equation 2.19 by solving for the un-known flow rate:

\[
Q_{i+1}^{i+1} = C_{\text{MC}0} Q_i^{i+1} + C_{\text{MC}1} Q_i^{i} + C_{\text{MC}2} Q_{i+1}^{i}
\]  

(2.20)

Where,

\[
C_{\text{MC}0} = \frac{\Delta t_R / K_{\text{stor}} - 2X_p}{2(1 - x_{SR}) + \Delta t_R / K_{\text{stor}}} \quad (2.21)
\]

\[
C_{\text{MC}1} = \frac{\Delta t_R / K_{\text{stor}} + 2X_p}{2(1 - x_{SR}) + \Delta t_R / K_{\text{stor}}} \quad (2.22)
\]

\[
C_{\text{MC}2} = \frac{2(1 - x_{SR}) - c_k \Delta t_R / \Delta x}{2(1 - x_{SR}) + \Delta t_R / K_{\text{stor}}} \quad (2.23)
\]

Where, \( C_{\text{MC}0}, C_{\text{MC}1} \) and \( C_{\text{MC}2} \) are dimensionless Muskingum-Cunge routing coefficients.

If \( K_{\text{stor}} \) is assumed to be \( \Delta x_{SR} / c_k \), where \( \Delta x_{SR} \) is the length of a sub-reach then Equation 2.20 is also the finite-difference of:

\[
\frac{\partial Q_{\text{str}}}{\partial t} + c_k \cdot \frac{\partial Q_{\text{str}}}{\partial x_{SR}} = 0
\]  

(2.24)
This is called a kinematic wave equation and can be derived by combining the continuity equation and the momentum (friction) equation. The parameter $c_k$ can be derived based on the kinematic wave theory. A discharge rating curve is necessary to compute the value of $c_k$ and a common approach to approximate the $c_k$ is by using the Manning’s equation (Seybert 2006). If the previous flood data are available the value of $c_k$ can be extracted by reversing routing calculations.

The value of celerity $c_k$ can also be calculated as a function of the average flow velocity by:

$$c_k = \beta_u u_f$$  

(2.25)

Where $u_f$ is the average velocity ($Q_{str}/A_x$) and $\beta_u$ is about 5/3 for wide natural channels. The coefficient $\beta_u$ comes from the uniform flow Equation 2.5. By taking the partial derivatives Equation 2.5 produces:

$$\frac{\partial Q_{str}}{\partial A_x} = m \cdot \frac{Q_{str}}{A_x} = \beta_u u_f$$  

(2.26)

Substituting these into the continuity equation:

$$\frac{\partial Q_{str}}{\partial x} + \frac{\partial A_x}{\partial t} = 0$$  

(2.27)

This gives Equation 2.20; if $c_k = \beta_u u_f$. If discharge data are available, $\beta_u$ can be estimated from Equation 2.25.

The parameter $X_P$ is a dimensionless weighting factor that has a value between 0 and 0.5 and generally between 0.1 and 0.3 for natural channels (Linsley et al. 1992). When $X_P = 0$, the volume of water in storage is purely a function of outflow alone, and $X_P = 0.5$ indicates that the inflow and the outflow have an equal weighting in determining the volume in storage and the routing equation produces translation without attenuation.

Cunge proposed the following equation for the determination of $X_P$:

$$X_P = \frac{1}{2} \left( 1 - \frac{q}{S_y c_k \Delta x_{SR}} \right)$$  

(2.28)

When $\Delta x_{SR} = 0$ (zero reach length), no translation or attenuation occurs.
When $\Delta t_R < K_{stor}$, the negative value at the outflow hydrograph can occur in the rising limb of a hydrograph. This negative value can be avoided by choosing $\Delta t_R$, $\Delta x_{SR}$ and $K_{stor}$ such that $(\Delta x_{SR}K_{stor})/(L\Delta t_R)$ lies below the curve shown in Figure 2.13 (Cunge 1969). Model for Urban Stormwater Improvement Conceptualisation (MUSIC) automatically sub-divides the reach length into a number of sub-sections $N = \Delta x_{SR}/L$, such that $(\Delta x_{SR}K_{stor})/(L\Delta t_R)$ lies below the curve shown in Figure 2.13, when the users set the values of $K_{stor}$ and $X_P$ for a stream reach. The values of $K_i$ for all sub-reaches are made equal such that $K_i = K_{stor}/N$, where $K_{stor}$ is the value defined by the users for the whole stream reach.

![Figure 2.13: $(\Delta xK_{stor})/(L\Delta t)$ vs. $X_P$ Curve](image)

In addition, when the value of $2K_i(1-X_P)$ for a sub-reach is less than $\Delta t_R$, the negative value is formed in the calculation of $C_{MC2}$ and ultimately in the falling limb of the outflow hydrograph which is unrealistic. Therefore, the parameters should be selected in such a way so that these negative values can be avoided. However, it is often difficult to satisfy both the conditions to stop the negative values in outflow hydrographs, especially when the value of $X_P$ is outside the range of 0.1 to 0.49 (CRC for Catchment Hydrology 2005). To avoid this discrepancy MUSIC limits the value of $X_P$ in between 0.1 and 0.49.
2.10.2 Hydraulic Routing

Hydraulic routing depends on the simultaneous solution of the continuity equation and the momentum equation for unsteady, gradually varied (non-uniform form) flow (Seybert 2006). Differential form of the continuity (conservation of mass) equation for the significant lateral inflow can be written as:

\[
A_{CF} \cdot \frac{\partial u_f}{\partial x_{SR}} + vB_T \cdot \frac{\partial h}{\partial x_{SR}} + B_T \cdot \frac{\partial h}{\partial t} = q_L, \tag{2.29}
\]

Where, \(A_{CF}\) is the channel flow area, \(B_T\) is the top width of a channel reach and \(q_L\) is the lateral inflow.

This equation is augmented with a conservation of momentum equation as presented by Henderson (1966) for unsteady flow in a stream reach:

\[
S_f = S_0 - \frac{\partial h}{\partial x_{SR}} \cdot \frac{v}{g} \cdot \frac{\partial u_f}{\partial x_{SR}} - \frac{1}{g} \cdot \frac{\partial u_f}{\partial t}, \tag{2.30}
\]

Where, \(S_f\) is the friction slope and \(g\) is the acceleration due to gravity.

These are nonlinear partial differential equations that require sophisticated numerical techniques for solution.

2.11 Water Quality Modelling Approaches

An accurate estimation of water borne pollutants transferred from upstream catchments to the downstream are crucial for the design of effective impact mitigation devices and management strategies. If we are able to predict water quality parameters at un-sampled locations or point with sufficient accuracy, we do not need to sample at these locations. Therefore, mathematical approaches have become prevalent for predicting water quality parameters of aquatic environments. The advancement of mathematical models as a tool for the simulation of water quality followed the historical development. Computer models are such water quality prediction tools which are widely used to simulate different water quality parameters. Computer models of urban stormwater runoff and pollutant loads are extremely useful in establishing whether various management strategies to produce water quality that conforms to legislation (Zoppou 2001).
In this context, a large number of water quality models have been developed for the prediction of water quality parameters from watershed areas. They are generally based on the similar principle. They first estimate the runoff volume using given rainfall data and geographical parameters. Then the quality of runoff is estimated by using the pollutant processes equations. These pollutant equations are either a simplified form of statistical relationships or the replication of empirical equations, such as the pollutant build-up and wash-off. These processes can be used either only for catchments or for streams or integrated.

2.11.1 Catchment Water Quality Modelling Approaches

The estimation of stormwater pollutants is typically obtained through the use of catchment water quality model, which simulates the water quality processes and influence both the quantity and quality of stormwater runoff. Storm Water Management Model (SWMM) is one of the first models developed for the simulation of stormwater runoff quantity and quality (Gaume et al. 1998). Since then many mathematical models have been developed by different researchers with different degrees of complexities and varying levels of accuracy based on different modelling approaches. However, most of the available models provide event based estimates of water quality parameters from specific sites.

As it is not possible to achieve a strictly physically based comprehensive operational model (Akan 1987), attempts were made to estimate the amount of transferred pollutants by using the simple event mean concentration (EMC) model. Since data requirements of the EMC model are less, it is easy to use. However, the EMC value can change between storms (Chiew and McMahon 1999; Butcher 2003); and hence the prediction of pollutant loads by the EMC model may be inaccurate for unmonitored storm events. To avoid this discrepancy arising in the EMC model, the sophisticated build-up wash-off (BUWO) models are formulated (Chen and Adams 2007). Pollutant build-up and wash-off is a continuous process which occurs on catchment surfaces during the $t_d$ and storm events respectively. These methods are most commonly used in catchment water quality models (Obropta and Kardos 2007). A hypothetical surface pollutant build-up and wash-off is shown in Figure 2.14.
SWMM is one of the BUWO models, which is a widely used model to simulate stormwater pollutants. However, the lack of data for the determination of the parameters of BUWO models can lead to the significantly biased results in the estimation of water borne pollutants. There is no standard form of pollutant build-up and wash-off formulation (Shaw et al. 2010). However, there are minor variations amongst the available formulations and their conceptual basis is the same.

The basic model for pollutant processes on a particular catchment surface can be written as follows (Shaw et al. 2010):

$$P_{t+\Delta t_{BW}} = P_t + \left\{ k_B \left( 1 - \frac{P_t}{P_{max}} \right) - \alpha_w P_t Q_t \right\} \Delta t_{BW} \quad (2.31)$$

Where, $P_t$ is the available pollutant mass (kg) accumulated on a catchment surface at time ‘t’, $P_{max}$ is the threshold at which additional pollutant do not accumulate on a catchment surface (kg), $k_B$ is the pollutant build-up coefficient (kg/time), $Q_t$ is the runoff rate (m$^3$/time) at time ‘t’, $\alpha_w$ is the wash-off rate constant (m$^3$) and $\Delta t_{BW}$ is the time increment for pollutant build-up or wash-off.
2.11.1.1 Event Mean Concentration

The event mean concentration (EMC) is a method used to characterise the concentration of pollutants from storm water runoff into nearby waterways and receiving water bodies. It is a statistical parameter used to present a flow weighted average concentration of a desired water quality parameter during a single storm event (Wanielista and Yousef 1992). An EMC model assumes a single flow weighted concentration and can be used across an entire storm event. The EMC model is the simplest model to calculate stormwater pollutants which is mostly useful for calculating annual pollutant loads (Charbeneau and Barrett 1998). This is the frequently used method to characterise stormwater loadings.

The EMC value can be determined by calculating the cumulative mass of pollutants and dividing it by the volume of storm runoff. The classical EMC model can be demonstrated as follows (Kim et al. 2004):

\[
EMC = \frac{\int_0^T P_c(t) dt}{\int_0^T V_{Ts}(t) dt}
\]  

(2.32)

Where, EMC is the event mean concentration, \( P_c(t) \) is the captured pollutant loads, \( V_{Ts} \) is the volume of runoff during the integration time interval and \( T_s \) is the duration of storm event.

However, in most of the cases, the monitored watershed area does not represent a single homogeneous land-use. Then the observed EMC is the runoff weighted sum of all EMCS from the individual land-use (Butcher 2003). For these cases, the mathematical expression of EMC can be written as follows:

\[
EMC = \frac{\sum_{i=1}^{n} (EMC)_i \{(R_D)_i(A_c)_i\}}{\sum_{i=1}^{n} \{(R_D)_i(A_c)_i\}}
\]  

(2.33)

Where, \( R_D \) is the depth of runoff and \( A_c \) is the contributing area.
2.11.1.2 Pollutant Build-up

Pollutant build-up is the accumulation of contaminants on catchment surfaces prior to rainfall events. However, during storm events, rainfall not only washes away pollutants from catchment surfaces but also deposits its own pollutants (James and Thompson 1997). This deposition is small enough to neglect in water quality modelling.

It is widely accepted that rain washable pollutants are engendered during the $t_d$. To the present, most of the build-up studies have been conducted by considering the $t_d$ as the most influential parameter (Egodawatta et al. 2009; Vaze and Chiew 2002; Chui 1997; Kibler 1982; Sartor et al. 1974). For example, the first build-up assumption justified by the work of Sartor and Boyd (1972) used the $t_d$ as the most important parameter. Alley and Smiths (1981) also developed a pollutant build-up model that included the $t_d$ as an input parameter. In addition, Soonthornnonda and Christensen (2008); Rossman (2004) found that the accumulation pollutant loads on a catchment surface is a function of the number of preceding dry weather days.

On the other hand, some other researchers criticised this assumption. Based on the experimental study at Belgrade in Yugoslavia and Lund in Sweden on impervious surfaces, Deletic and Maksimovic (1998) found that the $t_d$ had only a minor effect upon the accumulation of road surface pollutants. At Aberdeen, Scotland Deletic and Orr (2005) found that the $t_d$ had a weak negative influence on the accumulation of pollutant on catchment surfaces. Shaw et al. (2010) noted that the accumulation of pollutants on catchment surfaces is not deterministically related to the $t_d$. These contradictory results were due to the different geographical conditions and the time which had elapsed between data collections.

The common variables, such as surface type, roughness, slope, land-use and weather conditions affect the redistribution of pollutant loads; and hence the build-up rate. Initially available pollutant is another important factor in the accumulation of water quality pollutants which dominates the transport rate. Theoretically, it can be assumed that pollutants on catchment surfaces accumulate uniformly. However, the rate of pollutant build-up is significantly higher during the initial period (Ball et al. 1998; Sartor and Boyd 1972). Then the rate decreases as the dry days increases and eventually approaches zero due to land-use activities.
Egodawatta (2007) found that pollutant build-up is significantly higher during the first two days for road surfaces and seven days for roof surfaces. Vaze and Chiew (2002) noted that pollutant build-up on road surfaces may vary along the longitudinal direction depending upon the presence of traffic signals and bottlenecks. In addition, the regional and catchment management practices influence pollutant build-up and its composition (Ball et al. 1998). Therefore, different pollutant build-up functions are used to determine the accumulation of water quality parameters.

Usually, the accumulation of water quality parameters on a catchment surface is presented by two main build-up functions, i.e. the linear and the non-linear. At the very beginning, SWMM assumed that there was a linear increase of pollutants whereby a constant amount of pollutants increased during the $t_d$ (Novotny et al. 1985). The linear pollutant build-up with the $t_d$ can be written as follows:

$$\left( B_{t_d} \right)_D = k_L t_d$$  \hspace{1cm} (2.34)

Where, $\left( B_{t_d} \right)_D$ is the accumulation of pollutant on a particular catchment surface during $t_d$, $k_L$ is the linear build-up rate constant and $t_d$ is the antecedent dry time (days or hrs).

However, the linear build-up concept is not always adequate in depicting more realistic pollutant accumulation processes on catchment surfaces (Whipple et al. 1977). Ball et al. (1998) tested a range of models in different forms for pollutant build-up, and they found that the power function and the hyperbolic function are the best fit pollutant accumulation model. Therefore, Alley and Smith (1981) emphasised on the non-linear build-up model.

The non-linear pollutant build-up process was first identified from the field data collected by Sartor and Boyd (1972). From the non-linear build-up models, the power function, the exponential function and the Michaelis-Menton function are the main types found in water quality literature (Wang and Li 2009). Amongst them, the exponential function is the most widely employed build-up model (Chen and Adams 2006). The first published pollutant accumulation model was also exponential. Most of the available models accepted the accumulation of pollutants in the form of decreasing rate with increasing dry days.
The decreasing of pollutant accumulation with increasing dry days can be expressed as follows (Novotny et al. 1985):

\[
\frac{dB_{w_i}}{dt} = P_i - k_w B_{w_i}
\]  

(2.35)

Where, \( P_i \) is the sum of all the inputs and \( k_w \) is the wash-off coefficient.

The above Equations 2.34 and 2.35 are based on the assumption that every storm has the capacity to remove all the available pollutants from catchment surfaces for an adequate duration of rainfall event. However, from the literature review, it was easily understood that a single storm event can transport only a fraction of pollutants from a particular catchment surface (Egodawatta et al. 2007). For example, the experimental study by Vaze and Chiew (2002) showed that a significant rainfall event of 39.4 mm can remove only 35% of pollutant loads. Therefore, in practice, the amount of accumulated pollutants on catchment surfaces has two parts, i.e. pollutants build-up during the \( t_d \) and residual pollutants not washed-off by the previous storm events (Chen and Adams 2007). Hence, Charbeneau and Barrett (1998) proposed the following build-up model, which accounts for the mass not washed-off during the previous rainfall event.

\[
B_{w_d} = \left( B_{w_d} \right)_R + \left( B_{w_d} \right)_D
\]  

(2.36)

Where, \( \left( B_{w_d} \right)_R \) is the pollutant mass not washed off during the previous rainfall event.

2.11.1.3 Pollutant Wash-off

Pollutant wash-off is the transportation of the accumulated pollutants by surface runoff from catchment surfaces to nearby waterways and receiving water bodies (Temprano et al. 2006). An accurate estimation of water quality parameters entering into waterways and receiving water bodies depends on the accurate representation of the wash-off model (Sriananthakumar and Codner 1992). The complex process of pollutant wash-off is determined by a number of factors. Numerous research studies were conducted in identifying the governing variables of pollutant wash-off. However, most of the hypotheses were based on the four influencing rainfall runoff variables, i.e. rainfall intensity, rainfall volume, flow rate and runoff volume (Egodawatta et al. 2007).
According to Akan (1987), the characteristics of raindrops, overland flow and type of pollutants affect the pollutant wash-off rate. Moreover, Bannerman et al. (1993) found that the runoff energy has a significant influence on the pollutant wash-off process. In addition, Chen and Adams (2007) noted that types and conditions of street surfaces, the particle size, street cleaning and traffic density affect the pollutant wash-off rate. Berretta et al. 2007) also noted that the dynamics of the pollutant wash-off process is affected by hydrologic parameters, the catchment characteristics and the nature of pollutants. Duncan (1999) and Novotny et al. (1985) noted that the amount of pollutant washed-off from catchment surfaces is influenced by the amount of pollutant accumulated during the \( t_d \). Kibler (1982) noted that landscape modification affects the pollutant wash-off rate.

However, it is difficult to find out the relative importance of each parameter on the pollutant wash-off rate (Herngren et al. 2005). The interrelationships amongst different variables show the difficulty in understanding the degree of influence of individual variables. Therefore, different researchers proposed different pollutant wash-off models for the estimation of pollutant loads from catchment surfaces. Most of these studies focused on impervious surfaces, especially impervious road surfaces. There are only few studies of pollutant wash-off rates for residential and open land areas. Even these areas are major sources of water quality parameters in stormwater runoff (Soonthornnonda and Christensen 2008; Bannerman et al 1993).

The wash-off formulations used in most of the existing models are very similar to SWMM. However, these formulations are purely empirical and contain at least one parameter which has no physical basis. SWMM’s formulations are according to the algorithm of the first developed stormwater management model developed by Metcalf and Eddy Inc. et al. (1971). Metcalf and Eddy Inc. et al. (1971) proposed that the rate of pollutant wash-off from a catchment surface is proportional to the mass of pollutant which remains on that surface.

\[
\frac{dW_p}{dt_{BS}} = -k_w W_p \tag{2.37}
\]

Where, \( W_p \) is the pollutant wash-off from a catchment surface (mass) and \( t_{BS} \) is the time since the beginning of a storm event.
In urban stormwater quality modelling, Sartor et al. (1974) proposed a solution to this exponential equation as a function of rainfall intensity and duration. Their study concluded that the pollutant wash-off rate from an impervious surface area is proportional to the rainfall intensity and the mass of pollutant available on a particular catchment surface.

\[
W_p = \left(B_{t_0}\right) \left(1 - e^{k_w R_{t0}}\right)
\]  
(2.38)

However, Alley (1981) modified this exponential wash-off model and showed that pollutant wash-off is better predicted by the runoff rate instead of rainfall intensity. The expression can be written as follows:

\[
W_p = \left(B_{t_0}\right) \left(1 - e^{k_w R_{t0}}\right)
\]  
(2.39)

Where, \(R_{t0}\) is the average runoff rate of a storm event (mm/hr) during the time step \(t_{BS}\).

However, many other researchers noted that pollutant wash-off can be better predicted using the runoff volume, e.g., Deletic et al. (1997). Barbe et al. (1996) also suggested adopting a wash-off model which is related to the runoff volume. The expression of pollutant wash-off relationship with runoff volume can be written as:

\[
W_p = k_w V_{t0}^{E_p}
\]  
(2.40)

Where, \(E_p\) is the power of wash-off parameter.

On the other hand, Akan (1987) showed that the pollutant detachment rate at any point along a particular catchment surface is assumed to be proportional to the overland flow bottom-shear stress and the amount of pollutant available on that surface. Assuming the pollutant wash-off rate as proportional to the bottom-shear stress of the overland flow and the distribution density, Akan (1987) proposed a physically-based mathematical model. The expression can be written as follows:

\[
\frac{dP_p}{dt_{BS}} = k_w S_0 h B_{t_0, i}
\]  
(2.41)

Where, \(B_{t_0, i}\) is the mass of the pollutant per unit surface area.
However, there are many stormwater quality models which were developed based on the assumption that the rate of pollutant wash-off is proportional to the remaining pollutants and surface runoff (Chen and Adams 2007; Osuch-Pajdzinska and Zawilski 1998; Grottker 1987; Sartor et al. 1974).

\[
\frac{dW_p}{dt} = -\alpha_w B_t q_d
\]  \hspace{1cm} (2.42)

Where, \( q_d \) is the runoff rate per unit catchment area.

The primary boundary condition of these wash-off equations is the amount of pollutant available on a catchment surface (Barbe et al. 1996) to wash-out during surface runoff event. However, it is commonly recognised that pollutant wash-off is significantly greater at the beginning of a storm runoff compared to the later period, after rainfall has cleansed catchment surfaces (Novotny et al. 1985).

2.11.1.4 The First-Flush Phenomena

First-flush (FF) is a term used to refer to the higher concentration of pollutants in stormwater runoff during the initial period of a storm event. Generally, stormwater runoff containing a higher pollutant concentration is called FF. It is often noted that the highest intensity of rainfall are bursts at the initial period of a rainfall event, which could cause the higher pollutant wash-off during that period of storms and increases the occurrence of FF.

Numerous researchers considered the FF phenomena as an important factor in stormwater pollutant wash-off and transportation processes. They observed the highest amount of pollutant wash-off during the initial period of a storm event. Sartor et al. (1974) found that surface runoff from the first hour of a moderate to heavy storm would contribute more pollutant loads than sanitary wastes of the same area during the same time period. Egodawatta and Goonetilleke (2008) and Taebi and Droste (2004) observed that a higher fraction of pollutant from catchment surfaces is transported during the initial period of a storm. Therefore, Ballo et al. (2009) noted that the effects of FF are obvious in stormwater runoff. Hence, the existence of FF should be considered in stormwater management strategies.
The occurrence and the nature of FF can be influenced by a range of factors. It can vary with the rainfall intensity, the runoff volume and the catchment characteristics. Different hypotheses were tested to establish the relationships between FF and rainfall, runoff, catchment area, $t_d$, and collection network characteristics. If there is a large quantity of available pollutant then the transport rate from catchment surfaces is higher. Therefore, FF is increased in residential areas due to stormwater runoff from roof surfaces. Forster (1996) noted that roof surfaces produce a significant concentration of pollutants during the initial period of surface runoff. The imperviousness of a catchment area is also responsible for the FF phenomenon. Lee et al. (2002) observed the higher FF occurrence for small and impervious watershed areas.

Although the occurrence of FF is commonly reported, the FF effects appear only to limited number of pollutants and storm events (Batroney et al. 2010; Deletic and Maksimovic 1998). Deletic (1998) observed FF during large storm events when deposition of pollutants on catchment surface is small. In Melbourne, Australia Bach et al. (2010) did not observe any FF for all the catchments they studied. Therefore, the appropriateness of FF depends primarily on the nature and sources of pollution, in terms of drainage hydrology, pollutant mobility and pollutant supply. Therefore, identification of the existence or non-existence of the phenomenon is most critical.

FF is most important in stormwater treatment design. Pollutant collection systems during FF are employed to capture and isolate the most polluted runoff. Stormwater retention and detention basins are designed considering FF to treat the initial runoff which contains a higher concentration of pollutants (Egodawatta 2007). After the time of FF, the rest of the runoff is discharged without any treatment.

### 2.12 Stream Water Quality Modelling Approaches

Stream water quality modelling has become an integral part of water resources planning and water quality management options throughout the world. Stream water quality models inquire about the description of the spatial and temporal variations of water quality parameters. Moreover, these models are widely used to aid in water quality surveillance and to predict water quality conditions. They are extensively used in research as well as in the design and the assessment of water quality management measures.
Due to various studies on stream water quality, there are a large number of models available for the simulation of different water quality parameters. Since the initial study of Streeter and Phelps (1925), models are constantly refined and updated to meet the emerging problems of surface water pollution. Following the evolution of water quality problems, this starting model (Streeter and Phelps model) was developed with only two state variables. Since then more processes descriptions and variables have been gradually incorporated into it over the past seven decades in an attempt to improve the water quality simulation.

However, in most of the models the main processes considered are physical transport and exchange processes (such as advection and diffusion/dispersion), physical conversion processes, and chemical, biological and biochemical processes of water quality parameters (Rauch et al. 1998). In most of the water quality models, these processes are one-dimensional (1D), i.e. the changes in pollution constituents are only taken into account along the mean stream direction and are constructed by discretisation along stream. The dynamics of stream water quality parameters are described in the well-known three-dimensional (3D) partial differential equation (PDE) (Cox 2003; Chapra 1997; Thomann and Mueller 1987):

$$\frac{\partial \vec{c}}{\partial t} + u_x \cdot \frac{\partial \vec{c}}{\partial x} + u_y \cdot \frac{\partial \vec{c}}{\partial y} + u_z \cdot \frac{\partial \vec{c}}{\partial z}$$

$$= \frac{\partial}{\partial x}\left(\varepsilon_x \cdot \frac{\partial \vec{c}}{\partial x}\right) + \frac{\partial}{\partial y}\left(\varepsilon_y \cdot \frac{\partial \vec{c}}{\partial y}\right) + \frac{\partial}{\partial z}\left(\varepsilon_z \cdot \frac{\partial \vec{c}}{\partial z}\right) + f(\vec{c}, p_m) \quad (2.43)$$

Where, $\vec{c}$ is a multi-dimensional mass concentration vector for each of the water quality parameters, $x, y, z$ are spatial coordinates, $u_x$ is the average longitudinal velocity component, $u_y$ is the average transverse velocity component, $u_z$ is the average vertical velocity component, $\varepsilon_x$ is the turbulent diffusion coefficient in the longitudinal direction, $\varepsilon_y$ is the turbulent diffusion coefficient in the transverse direction, $\varepsilon_z$ is the turbulent diffusion coefficient in the vertical direction and $f$ is a term representing the rates of change of state variables due to biological, chemical and other conversion processes as a function of the concentration $\vec{c}$, and the model parameters $p_m$, which is subjected to calibration.
2.12.1 Available Stream Water Quality Models

Stream water quality models are developed by a simultaneous solution of a set of equations describing each of the pollutants involved. However, most of the equations require the users to provide the values of the rate parameters and to calibrate these parameters in order to achieve a fit with the observed data (Cox 2003).

Generally, there is not enough time or funding for the field investigation of water quality parameters (Cox 2003). Therefore, stream water quality models are crucially important for the prediction of water quality parameters within a stream reach. Numerous examples of commercial stream water quality models (SIMCAT, TOMCAT, QUAL2E, QUASAR, MUSIC, MIKE11 etc.) were developed for the simulation of water quality parameters with some ranged values of the model parameters.

SIMCAT (SIMulation of CATchments) is a one-dimensional, steady state, stochastic stream water quality model which represents the inputs from the point-source effluents. The model can simulate pollutants either conservatively or by having the first order decay. Although the model calculation is quick, the approach considered is over simplistic. This model is suitable for pollutants in freshwater where sediment interaction is negligible (Cox 2003).

TOMCAT (Temporal/Overall Model for CATchments) is essentially identical to SIMCAT, i.e. a steady state model. However, TOMCAT allows for more complex temporal correlations. Although the model has limited functionality in terms of processes included, the use of seasonal statistics does allow for the potentially greater accuracy than that could be achieved in the SIMCAT model. Nevertheless, the model is limited by over simplistic ascriptions of flow and water quality processes even with the excellent data. In addition, due to the inclusion of some seasonality, it is not a dynamic model and cannot examine short term variability, such as the diurnal effects.

The stream water quality model QUAL2E was developed and released by the United States Environmental Protection Agency (US EPA) in 1985. The QUAL2E is a steady state model for the conventional water quality parameters in 1D stream and well-mixed ecosystems. The model is similar to SIMCAT and TOMCAT models. However, the conceptualisation of QUAL2E is rather more advanced than these two models (Cox 2003).
The basic theory of QUAL2E is based on the assumption that major transport mechanisms, advection and dispersion are significant only along the main direction of flow. QUAL2E is considered as one of the most widely used stream water quality simulation models throughout the world (Brown and Barnwell 1987). Although the model is widely used, it did not get great exposure in the UK because it is a steady state model and there is no stochastic component. Moreover, the model is much complex and requires extensive data. Like any other water quality model, QUAL2E needs to be correctly calibrated and verified for a particular stream for the reliable prediction of water quality parameters. In addition, QUAL2E and other similar models do not address a number of practical problems, such as stormwater flow events (Shanahan et al. 1998).

MIKE 11 is a 1D (cross-sectionally averaged) dynamic stream water quality model which was developed by the Danish Hydraulic Institute (DHI). MIKE 11 is a widely used hydraulic as well as water quality model used to assess the impact of pollutant discharges into streams and estuaries. The model consists of a set of flow and water quality modules. The model is used for the simulation of water quantity and quality, such as sediment transport in estuaries, streams, irrigation channels and other water bodies. In addition, the model accounts for photosynthetic production, respiration, atmospheric re-aeration, BOD decay and nitrification. The number of water quality parameters included depends on the level of model being run. Although MIKE 11 is an advanced water quantity and quality model, a common problem for this complex model is the requirements for large amounts of data which may not always be available.

QUASAR (QUAlity Simulation Along River Systems) is a dynamic stream water quality model which describes the time varying (i.e. dynamic) transport and transformation of solutes in branched river systems using 1D, ordinary and lumped parameters for a differential equation of the conservation of mass (Cox 2003).

MUSIC (Model for Urban Stormwater Improvement Conceptualisation) was developed by the CRC for Catchment Hydrology, Australia. Although it is a catchment water quality model, the Muskingum-Cunge theory was used for routing streamflow and water quality parameters. The model is stochastic and can simulate water quality parameters with some treatment options. However, the model does not consider internal conversions of the reactive pollutants within stream reach.
All of the developed models contain similar assumptions and limitations that need to be understood for a meaningful interpretation of the model simulation (Cox 2003). However, it is difficult to identify a single model that is good enough to simulate all of the pollutant processes. Some of them were developed for specific sites and are not useful for other areas. For example, due to the stochastic components and the lack of commercial exposure, SIMCAT and TOMCAT are not generally used outside the UK (Cox 2003). QUAL2E incorporated degradation of organic materials, growth and respiration of algae, nitrification (considering nitrite as an intermediate product), hydrolysis of organic nitrogen and phosphorus, reaeration, sedimentation of algae, organic phosphorus and organic nitrogen, sediment uptake of oxygen and sediment nutrients release. MIKE 11 was developed on the basis of modules, e.g. rainfall-runoff component, a hydrodynamic module, a water quality module and a sediment transport module which are mixed of the conceptual and physics based models (Merritt et al. 2003). The modules can be combined and simulate advection-dispersion, water quality, sediment transport, eutrophication and rainfall-runoff.

Although the processes formulation of MIKE 11 is similar to QUAL2E, there are some remarkable differences. The most important difference is the division of organic matter into dissolved and suspended fractions in MIKE 11. The description of phosphorus cannot be compared because in MIKE 11, phosphorus is estimated in another module. Neither MIKE 11 nor QUAL2E makes an attempt to describe populations of bacteria. MUSIC considers water quantity as well as pollutant loads (SS, TN, TP and gross pollutants) as conservative substances which are unusual to other models. However, MUSIC is widely used in Australia for predicting stormwater quality pollutants. There are variations amongst the available stream water quality models in terms of their complexity, number of parameters, computation time and processes involved.

**2.12.2 Sedimentation Processes**

Water quality of seas, lakes and streams depends on the concentration of fine sediments (especially clay and silt particles) brought from upstream catchments with surface runoff (Rimkus et al. 2007). During the period of intense rainfall events, the sediment particles are transferred to nearby waterways and settled on the stream bed and on floodplains especially during the low flow conditions.
From the primary view point of erosion, deposition, navigation and flood defence, the mechanism of sedimentation has become considerable interest to hydraulic engineers, coastal engineers, geologists, hydrologists and geographers (Yang 2005). The sedimentation phenomena further play an important role in the evaluation of earth surfaces topology especially in river beds, reservoirs, estuarine and coastal regions (Zhou et al. 2003).

The transported sediment affects not only the cross sectional geometry of waterways, but also the hydrodynamic conditions of fluid in motion. The suspended load changes the fluid density, which is generally taken as constant in the hydrodynamic modelling. If suspended transport and deposition is not taken care properly, further remedy is needed. Lin and Namin (2005) noted that long-term morphological processes are involved with the transport of heavy metals and toxic wastes via the adsorption on sediment particles. Therefore, sufficient understanding of sedimentation processes is essential for integrated river management and river engineering. Depending on the size and density of bed materials and flow conditions, sediment particles are divided into two main categories, i.e. bed load and suspended load. However, the scope of this thesis is limited to suspended load.

Over recent years, a number of SS models ranging from 1D to 3D equations were developed. This is because of the sedimentation rate depends on a large number of factors which form the flow structure. The volume and distribution of the deposited sediment depends on the distribution and the duration of flow, vegetation on stream banks, the concentration of incoming sedimentation, the hydrological and other conditions (Vaikasas and Rimkus 2003).

The process of sediment can be illustrated from the well-known advection-diffusion Equation 2.43. For the purpose of water quality modelling, some simplifications of the equation can be made by reducing the number of spatial dimensions. This is reasonable because for relatively shallow streams, the mixing depth is short and thus a depth integrated (i.e. 2D) form can be applied (Cox 2003).

\[
\frac{\partial \tilde{c}_{ss}}{\partial t} + u_x \cdot \frac{\partial \tilde{c}_{ss}}{\partial x} + u_y \cdot \frac{\partial \tilde{c}_{ss}}{\partial y} = \frac{1}{h_x} \frac{\partial}{\partial x}\left(h_x \frac{\partial \tilde{c}_{ss}}{\partial x}\right) + \frac{1}{h_y} \frac{\partial}{\partial y}\left(h_y \frac{\partial \tilde{c}_{ss}}{\partial y}\right) - D \quad (2.44)
\]
Where, \( \bar{c}_{ss} \) is the depth average SS concentration vector and ‘D’ is a term representing the rate of SS deposition or erosion (kg/m³/s).

In the calculation of SS deposition, the term ‘D’ is of great importance which is introduced through the bed boundary conditions. For steady flow problems, the dispersion is often negligible and can be omitted for the simplification. Then it produces an ordinary differential equation (ODE) which is easy to solve and analyse (Cox 2003; Rauch et al. 1998).

Vaikasas and Rinkus (2003) determined the SS deposition rate \( D_{ss} = D_{sed} h \) of sand particles by the concept of suspension transport capacity of a particular stream:

\[
D_{ss} = \frac{q_{ss,in} - q_{ss,tr}}{q_w} \cdot w_s
\]

(2.45)

Where, \( D_{sed} \) is the decrease in sediment flow rate due to sedimentation (kg/m³/s), \( D_{ss} \) is the deposition of SS per unit area of stream bottom (kg/m²/s), \( q_{ss,in} \) is the SS flow rate per unit width (kg/m/s), \( q_{ss,tr} \) is the transport of SS per unit width (kg/m/s), \( q_w \) is the streamflow rate per unit width of reach (m²/s) and \( w_s \) is the settling velocity or fall velocity of particles (m/s).

When applying this Equation 2.45, one must know the effective transport capacity \( q_{ss,tr} \) of that flow. However, SS transport is a complex phenomenon with the sediment undergoing a series of processes such as erosion, deposition, and advective and diffusive transportation. Numerous formulas were proposed for the calculation of \( q_{ss,tr} \). In most of the cases, the model for the estimation of SS transportation were developed by applying the most common methods proposed by van Rijn (1984a, 1993) and Bagnold (1966).

When the bed-shear velocity exceeds the particle fall velocity, upward turbulent forces may be higher than the submerged weight of the particles and as a result suspension of particles occurs (van Rijn 1993). However, the concentration of SS is higher close to the bottom and gradually decreases with the distance up from the bed. The depth integrated SS transportation proposed by van Rijn (1984a, 1993) can be expressed as follows:

\[
q_{ss,tr} = F_{DS} \bar{u} h c_s
\]

(2.46)
Where, \( c_\delta \) is the reference level concentration of SS at height ‘\( \delta \)’ above bed level, \( \bar{u} \) is the depth average flow velocity (m/s) and \( F_{DS} \) is the dimensionless shape factor, which is expressed as follows (van Rijn 1984a, 1993):

\[
F_{DS} = \int_a^h \frac{u_z}{u} \cdot \frac{c}{c_\delta} \cdot d(z/h)
\]  

(2.47)

Bagnold (1966) also proposed a SS transport model based on the energy balance concept relating to the power of a particular stream. The equation can be expressed as follows:

\[
q_{ss,p} = p_{str} \cdot \frac{e_s u_s}{W_s} \cdot (1 - e_b)
\]  

(2.48)

Where, \( p_{str} \) is the available stream power per unit bed area, \( e_s \) is the suspension efficiency factor (suspension work rate per unit stream power), \( e_b \) is the bed-load transport efficiency factor (bed-load work rate per unit stream power) and \( u_s \) is the transport velocity of SS (m/s).

There are also other models available for the estimation of SS transportation rate. However, all of the models proposed by different authors are based on experiments performed in different flow conditions; therefore they produce different results. In addition, there are only a few numbers of models available for the generic application of the SS simulation. The lack of accepted fundamental principles and the demand for the practical application stimulated the engineers and scientists to improve existing sediment transport equations.

### 2.12.3 Reactive Pollutants Processes

The solute transport equations described in the previous section 2.12.2 are sufficient for simulating substances which do not undergo any transformation or reaction within a stream reach. However, for certain pollutants (e.g. nutrients, BOD, DO), these equations need modification, because they are affected by a number of factors. Without advection, dispersion and external inputs, internal transformations of these pollutants occur within stream.
Similar to the SS processes, equations for the reactive pollutants can be derived from the advective-diffusive Equation 2.43. For 1D flow after neglecting the diffusion term, the advective-diffusive equation becomes a simple ordinary differential equation (Marsili-Libelli and Giusti 2008).

\[
\frac{dC_R}{dt} = -K_R f(C_R)
\]  

(2.49)

Where, \(C_R\) is the concentration of reactive pollutant (mg/L), \(K_R\) is coefficient for the reaction parameter or rate coefficient and \(f(C_R)\) is the general reactive term for the pollutant concentration \(C_R\).

This is the simplest form of the commonly used equation in water quality models for the reactive pollutants (Cox 2003). For the prediction of reactive pollutants, this \(f(C_R)\) is highly significant because it represents the pollutant processes.

For modelling the nitrogen processes, Equation 2.49 can be used as a first order decay function. However, most of the widely used models (e.g. QUAL2E, MIKE 11) present more detailed processes of nitrogen. The nitrogen processes can be presented by organic nitrogen, ammonium \((NH_4)\), ammonia \((NH_3)\), nitrite \((NO_2)\) and nitrate \((NO_3)\). Nevertheless, in general, the last one \((i.e. NO_3)\) is present in the highest quantity. In the nitrogen cycle, organic nitrogen is produced by algae, and removed by hydrolysis into ammonium and by the settling process. Ammonium is reduced from the water column by the nitrification process. Ammonium is also released by algal respiration and reduced during photosynthesis. Again the denitrification process generates ammonium and releases it to water. On the other hand, the nitrate concentration is affected by nitrification and denitrification. Nitrite is produced from ammonium in the first stage of denitrification and removed by oxidation of nitrite to nitrate. Finally, nitrate in the water column is consumed by algae. Since the action of micro-organisms and chemical reactions convert these forms sequentially, it is convenient to consider \(TN\) in modelling (Boorman 2003).

Similar to the nitrogen processes, the phosphorus processes can be described from Equation 2.49. Phosphorus can exist as either organic or dissolved fractions. Organic phosphorus is produced by algae and removed by the normal decay and settling.
However, the dissolved phosphorus is produced due to the decay of organic phosphorus and its release from sediment, and removed by algal photosynthesis. Since these processes are very complex and difficult to include in modelling due to the lack of rigorous data, it is convenient to consider TP in water quality modelling instead of fragmental components.

Other than SS and nutrients, BOD and DO are the most important water quality parameters for the degradation of stream water quality. The process of BOD can be derived by the oxidation procedure and by the settling procedure. In stream water, BOD processes are simulated as being removed by biological oxidation and sedimentation; and accumulated due to algal death. However, some models (e.g. MIKE 11) include the contribution of BOD from re-suspension. The total process of DO can be derived by the reaeration, algal photosynthesis and respiration, sediment oxygen demand, nitrification and use of oxygen to satisfy BOD within the water column.

However, most of the rate parameters of reactive pollutants are temperature dependent which itself is modelled as a conservative variable. The function to represent the temperature effect on rate coefficients is:

\[ k_{Te} = k_{ref} \theta_{ref}^{Te-20} \]  

(2.50)

Where, \( k_{Te} \) is the process rate at \( (Te) \) °C (1/day), \( k_{ref} \) is the process rate at a reference temperature, always 20 °C and \( \theta_{ref} \) is the temperature correction factor of the reactive pollutants at reference temperature.

### 2.13 Integrated Models

From the evolution of environmental pollution, many processes of the environment were developed and modelled. The detailed models were developed to describe individual needs and objectives of environmental problems. However, currently adopted stormwater quality models are unable to reproduce the historical pollutographs accurately and reliably (Cheah and Ball 2007). Therefore, decision makers at all levels are feeling dissatisfied with the outcomes resulting from ‘narrowly-focused incremental and disjointed’ environmental management (Margerum and Born 1995).
Many of the present researchers are arguing that earlier approaches failed to deal with many interconnections. Therefore, Parker et al. (2002) noted that environmental problems of the 21st century cannot be considered in isolation. The integration is essential to address the current and future environmental problems.

The integrated modelling approach will set priorities and implement natural resource management options in a better way. According to Jakeman and Letcher (2003), the integrated approach will allow us to develop a better understanding of the interdependencies of the processes involved and may lead to the identification of the outcomes which are considered better tradeoffs than pre-specified optima. Furthermore, Rotmans and Asselt (1996) stated that integrated models are useful: (i) to analyse the dynamic behaviour of complex systems, (ii) to show interrelations and feedback amongst the various issues, (iii) to make uncertainties explicit and analyse the accumulation of uncertainties and (iv) to develop end-to-end strategies. In addition, the integrated approach will allow us to represent complexities and interactions within and between different environmental systems effectively. Hence, the integrated modelling approach will become an objective of government policy internationally.

Although the integrated modelling approach is known, there is no generally agreed upon the definition of what constitutes integration. Different disciplines focus on different scales for the integration of modules from different sources (Parker et al. 2002). Different researchers described various forms of integration. For example, linking models with GIS (Geographic Information System), integrating with software, integration of more than one model, or even stakeholder participation are different forms of integration.

Rotmans and Asselt (1996) provided a definition of integrated assessment: “Integrated Assessment is an interdisciplinary and participatory process of combining, interpreting and communicating knowledge from diverse scientific disciplines to allow a better understanding of complex phenomena”. Risbey et al. (1996) stated that models that attempt to integrate information by linking mathematical representations of different components of natural and social systems is one way of creating an integrated model. In addition, the integration of different disciplines is required to gain insights into complex processes.
In the case of water quality, the integrated modelling approach can be defined as modelling of interactions between two or more physical systems (Rauch et al. 2002), e.g. catchment hydrology and water quality, stream hydrology, hydraulics and water quality. An integrated water quality model is the combination of more than one model to observe the changes in water quality parameters passing through nearby waterways into receiving water bodies, such as the integrated catchment-stream water quality model.

The process of integrated modelling approach should be considered as an important product for any particular project. According to Parker et al. (2002), by recognising the approach of integrated model, earlier forms of system models can be replaced with integrated models which will incorporate different dimensions of the environment and facilitate scenario generation and decision support functions. However, the development of an integrated model is a challenging task (Rauch et al. 2002). The main constraint is the complexity of systems that prevents existing simple deterministic models. Furthermore, data availability is a severe constraint for obtaining more informed and confident decision support through an integrated model.

2.14 Conclusions

This chapter reviewed the current state of knowledge with respect to water quality parameters, their sources, hydrology, current water quality modelling approaches and the related issues. A list of considerations which are appropriate for model selection was also presented in this chapter. Finally, the chapter reviewed the issues related to the integrated modelling approach. From the discussion of the literature review, important conclusions relating to hydrologic, hydraulic and water quality regime can be summarised.

The primary water borne pollutants identified in the literature include litter, SS, nutrients, hydrocarbons, heavy metals and organic carbon. Among them SS and nutrients are mainly responsible for the degradation of the quality of waterways and receiving water bodies. Most of the other pollutants are chemically adsorbed to the particulate pollutants.
Although human existence is impossible without water, excesses in human activities are responsible for the degradation of water quality and water environments. Due to the various anthropogenic activities, road surfaces are recognised as the largest contributor of water quality parameters, which are mainly generated from vehicles. All other impervious and pervious surfaces are the dominant sources of pollutants under certain conditions. The extent of pollutant contribution varies depending on the characteristics of catchment surfaces. Types of land-use also play an important role in the accumulation of water quality parameters.

Pollutant loads from different sources are transported into waterways and receiving water bodies and deteriorate the quality of aquatic ecosystems. To address water quality concerns adequately, it is important to measure the amount of pollutants present on catchment surfaces and their transportation during storm events. Proper estimation of pollutant loads helps to control their transportation by guiding watershed management authorities for the implementation of appropriate management options. However, management strategy based on the test results are costly, time consuming and sometimes impossible. The use of mathematical models has become a common practice of analysis, aimed at understanding of the cause-effect relationships between emissions and water body quality, assessing the impact of the changes and to design as well as to assess the effectiveness of mitigation measures.

The modelling technique plays an important role for the development of appropriate watershed management strategies and to support decision making processes. The models are used for the analysis of various scenarios and for the evaluation of alternative management operations to achieve desired objectives. The goal of a model is to reduce the complexity and effort spent on hand computation and analysis.

At present, numerous excellent and elegant water quality models are available and new models are appearing regularly to provide tools for water quality managers and regulators for the protection and improvement of aquatic environments from the impact of pollution and for the adoption of appropriate management options. However, there are no adequate guidelines available to select appropriate modelling options which can be used to simulate various processes. The lack of user friendliness and proper selection of the model parameters further hinder the application of water quality models.
Different algorithms are employed in different models to estimate water quality parameters. These differences among sub-models are due to the different levels of complexity of systems modelled and are likely to be due to the uncertainty of differently measured data. Few models exist which are appropriate for providing the accurate pollutant loads prediction and identification of the critical pollutant sources. A range of problems including over parameterisation, data requirements, inappropriate structure and unsuitability of model assumptions often constrains the use of a complex model. Therefore, more complex models are often no more successful.

There are two major components of water quality models, i.e. the quantity and quality components. The estimation of the quantity characteristics is a well-developed field. There are various tools and established methods for the estimation of water quantity. However, the estimation process of quality has only been developed recently. The lack of understanding of the pollutant processes and underlying variables, the use of a quality model can often leads to gross errors. An extensive amount of data is needed to calibrate the water quality component of a model.

This literature review showed that no one’s model could provide all of the functionality, which is essential in the prediction of water quality parameters. The models were developed based on the experimental study and for a particular situation. Therefore, the generic application of a particular water quality model is often difficult. In addition, detailed models of catchments and stream water describe the performance according to the individual needs and objectives. Since most of the models were developed for a particular purpose, it is unfair to set one model against another in terms of broad applicability. Due to the large variability of errors, it is difficult to draw specific conclusions on superiority of one model over another.

The literature review also identified that no water quality models were developed to simulate catchment and stream water quality parameters integratively. However, the growing demand for the integrated water quality model originates from current surface quality policy programs. The earlier approaches to environmental management usually failed to deal with interconnections, complexities, multiple perspectives, multiple uses and resulting cross-cutting externalities. Today’s challenge is to move from such individual considerations of system performances to an integrated approach.
Environmental problems of the 21st Century are public and non-exclusive. To achieve an integrated and sustainable future, instead of a fragmented disciplinary future, there is a call for the integrated modelling approach. For any particular project, the process of integrated assessment and modelling is important. Therefore, the literature review highlighted the necessity for guidelines or improved methods and also the need for the further development of integrated models for the prediction of water quantity and quality. There is no doubt that in-depth knowledge of the integrated modelling approach is crucially important to implement BMPs and to safeguard water quality, such as the design of pollutant control structures.
Chapter 3
Model Development

3.1 Introduction

Chapter 2 illustrates the stipulations for the development of an integrated water quality model. This chapter demonstrates the development of an integrated catchment-stream water quality model which continuously simulates different water quality parameters. The integrated model is comprised of two individual models, i.e. the catchment water quality model and the stream water quality model. These models can be used separately or in combination to predict water quality parameters. The output from the catchment water quality model can be used as the input for the stream water quality model.

Each of the models was developed with two basic components, i.e. the water quantity component and the water quality component. The quantity model estimates hydrographs generation at the point of interest; while the quality model estimates the amount of transported pollutants into waterways and receiving water bodies. The quantity model was incorporated with the quality model to be able to continuously simulate different water quality parameters. The main aim was to determine the quantity and quality of surface runoff discharged from catchment surfaces into waterways and receiving water bodies during rainfall events by using an integrated modelling approach.

The three basic water quality parameters were included for the development of the integrated model, i.e. SS, TN and TP. However, for the stream water quality model, the BOD process was also included. The specific rational for the selection of these pollutants is not available, however they can be summarised. Water is the most important vector for transporting pollutants from upstream catchments into waterways and receiving water bodies, and therefore its inclusion is essential. SS is an important agent for the transportation of adsorbed pollutants, such as heavy metals, pesticides and other pollutants. They also play a significant role in evolving the shape of estuaries and coastlines (Boorman 2003). Moreover, the growth rate of photosynthetic organisms is reduced by the high concentration of SS (Akan and Houghtalen 2003).
Nitrogen has many sources within catchments including the deposition, fertilizer application, discharge from sewage treatment plants etc. In the form of ammonia, nitrogen is highly toxic to fish and aquatic organisms (Bowie et al. 1985). In addition, nitrogen is generally a limiting nutrient to algal growth. Like nitrogen, too much phosphorus also leads to the excessive plant growth. Both of these nutrients (nitrogen and phosphorus) are the critical components of eutrophication. BOD is considered to be one of the most important water pollutants for the degradation of stream water quality.

These pollutants are often used as the indicators of the quality of water as well as their impact on receiving water. On the other hand, the estimation of $TN$ and $TP$ from the direct proportioning to the SS loadings increases errors due to the uncertainty in the prediction of SS loads (Butcher 2003). In addition, for water quality management strategies, Russel et al. (1998) noted that the consideration of only the dissolved portion of pollutants will underestimate the total nutrients flux. Therefore, $TN$ and $TP$ were considered in developing a meaningful model and a water quality prediction tool. Overall based on data availability and other important factors in water quality modelling, these water quality parameters were selected for the development of the integrated model. However, other pollutants can be incorporated with the model.

This chapter starts with the construction of a catchment water model, including catchment hydrology and pollutant processes. After that, the chapter focuses on the development of a stream water quality model, including the quantity and quality of stream water and their progressions. Finally, the chapter discusses the integration of the catchment water quality model with the stream water quality model.

### 3.2 Catchment Water Quality Model

General strategies and programs for watershed management always depend on the catchment water quality modelling results which involve both the runoff and the pollutants processes (Chang et al. 2007). Therefore, over the years, various catchment water quality models have been developed for the estimation of surface runoff and pollutant loads. A comprehensive catchment water quality model facilitates the simulation of both surface runoff and pollutant loads simultaneously (Chiew and McMahon 1997). Hence, Amoudry (2008) noted that modelling the quality of water required describing both pollutant and ambient water motions and their interaction.
The catchment water quality model was developed for the continuous simulation of stormwater quantity and quality during the period for which rainfall data are given to the model. The model is comprised of two sub-models, i.e. the runoff model and the pollutant model. The runoff model also known as the hydrologic model enables the simulation of catchment hydrological processes via a range of commonly applied rainfall loss and routing methods. The pollutant model estimates the amount of pollutant accumulated during the $t_d$ and the amount washed-off during surface runoff event. The model also recognises the influence of effective impervious area of catchment surfaces.

The basic functions of the model are similar to the comprehensive hydrological and water quality simulation model, SWMM. However, SWMM does not consider the rainfall kinetic energy which is considered as an important factor for the pollutant wash-off as found in recent studies, such as Shaw et al. (2010); Egodawatta et al. (2007). The model has the option to consider the rainfall intensity from which the kinetic energy can be calculated directly. The final output of the model is the quantity and quality of stormwater runoff at a catchment outlet.

The major advantage of the model is that it can be used for either the continuous or the event based simulation of hydrology and water quality parameters associated with surface runoff during rainfall events. In addition, the model is capable of considering both the pollutant loads and concentration of catchment surfaces. The model thus improves the estimation of pollutant loads entering into waterways and receiving water bodies.

### 3.2.1 Runoff Model

It is well recognised that stormwater pollutants cannot be predicted accurately without proper estimation of the runoff quantity. Usually, the runoff quantity from a storm event is measured by the traditional rainfall-runoff models (Kokken et al. 1999). However, the output of such models is used to investigate wider environmental problems, such as the water quality issues (Christophersen and Wright 1981). Therefore, rainfall-runoff transformation is considered to be the key component of any stormwater quality model because it characterises the runoff generation mechanism and determines the degree of model complexity (Chen and Adams 2007).
The area contributing to stormwater runoff is either impervious surface or pervious surface or from mixed land surfaces. Therefore, the model was developed by dividing an entire catchment into impervious and pervious areas to consider proper catchment responses. The structure of the conceptual rainfall-runoff model is shown in Figure 3.1.

![Figure 3.1: Structure of the developed rainfall-runoff model](image)

The rainfall-runoff transformation was modelled using two main processes simultaneously, i.e. rainfall loss and overland flow routing. The IL-CL procedure was used to account for rainfall loss from sub-catchments. The routing model was developed based on the TA runoff estimation method. The important input parameters for the runoff model are the rainfall hyetograph, daily average evaporation, IL from both impervious and pervious surfaces and the CL parameters. The output from the runoff model is the quantity of surface runoff at the catchment outlet.

Modelling runoff from an impervious surface is easy, because for this surface, infiltration loss is negligible and can be assumed to be zero. Since surface runoff occurs immediately after a rainfall event from impervious surface, there is no CL for this surface area. Only the contributing loss for impervious surface is Evp and IL. Therefore, in estimating the runoff from an impervious surface, the model considers Evp and IL only. On the other hand, for pervious surface, there is the Evp loss, infiltration loss, IL and watershed leakages. The model subtracts the Evp loss, IL and CL for the estimation of surface runoff from pervious area as shown in Figure 3.1.
The runoff model can predict surface runoff based on different hydrologic considerations. The runoff calculation can be done from impervious and pervious surfaces separately for any given duration of rainfall. The total runoff at the catchment outlet is assumed to be the combination of area-weighted runoff from impervious and pervious surfaces of a particular catchment. General equation of the surface runoff calculation can be written as follows:

\[
Q_S = \begin{cases} 
0 & R_a \leq L_{impv} \\
A_T IMPV \left( R_a - L_{impv} \right) & L_{impv} < R_a \leq L_{pev} \\
A_T \left( R_a - L_{pev} - IMPV \left( L_{impv} - L_{pev} \right) \right) & R_a > L_{pev}
\end{cases}
\]  

Where, \( Q_S \) is the surface runoff, \( IMPV \) is the percent imperviousness of a particular catchment, \( R_a \) is the rainfall, \( L_{impv} \) is the rainfall loss from impervious surface area and \( L_{pev} \) is the rainfall loss from pervious surface area.

3.2.1.1 Rainfall Loss Model

The governing rainfall abstraction/losses estimation is difficult over an entire catchment. To overcome difficulties in the spatial variability of rainfall, the topography, and the characteristics of catchments (such as vegetation and soils), simplified lumped conceptual models are used. In this thesis, rainfall loss is divided and modelled into the three categories, i.e. the \( Evp \) loss, \( IL \) and \( CL \). Since the \( IL-CL \) method is a widely used model in Australia, the current model was developed based on \( Evp \) and the \( IL-CL \) model. The users have the option to provide the average value of the \( Evp \) loss.

The part of rainfall loss which occurs at the early part of a storm event is called \( IL \). After the initial time period, this loss disappears. The value of \( IL \) can vary depending upon the characteristics of a particular catchment and weather conditions. Tularam and Ilahee (2007) noted that the rainfall duration is also an important factor in the calculation of \( IL \). However, this loss is considered as fixed and most of the time, the value of \( IL \) is derived from literature. Therefore, in the developed model, the users have an option to provide the value of \( IL \) as a constant input value. It was mentioned earlier that the model estimates impervious surface runoff and pervious surface runoff separately. Hence, there are two different options to provide the value of \( IL \) for impervious surface and for pervious surface.
Besides *IL*, all other rainfall losses that could occur through the remaining storm period are usually categorised as *CL*. The values of *CL* depend on the type of surface areas of a particular catchment. The usual practice of *CL* consideration is constant (Rahman et al. 2002a). However, Ilahee and Imteaz (2009) presented a detailed study using the *IL-CL* model for many eastern Australian catchments. Their study concluded that the values of *CL* are higher at the beginning of a rainfall event and gradually decreases with the time as rainfall continues.

To calculate the values of *CL*, the developed model allows the users in using any of the available three different methods, i.e. constant, exponentially decreasing or linearly decreasing. By allowing the different options for *CL* estimation, the model becomes flexible and is improved. Equations 3.2 to 3.4 represent *CL* models for constant, exponential decreasing and logarithmic decreasing respectively. The choice and validity of a model depends on the availability of data and the runoff calculation processes.

\[
CL = \delta_{CL} \quad (3.2)
\]

\[
CL = A_{CL} - B_{CL}e^{-t_{BS}} \quad (3.3)
\]

\[
CL = C_{CL} - Log(t_{BS}) \quad (3.4)
\]

Where, *CL* is the continuing loss (*mm/hr*), *A_{CL}* is the parameter for exponential decreasing *CL*, *B_{CL}* is the coefficient for exponential decreasing *CL*, *C_{CL}* is the parameter for logarithmic *CL*, *t_{BS}* is the time since commencement of a rainfall event (*mins*) and \(\delta_{CL}\) is the constant value of *CL*.

### 3.2.1.2 Routing Model

While watershed runoff models can be formulated with various degrees of complexity based on different methodological considerations, the catchment hydrologic model for this research study was developed using the *TA* method to represent the partial area runoff contribution of a particular catchment. It was recognised that a fundamental part of a runoff hydrograph development is the use of the *TA* method. Since the *TA* method is one of the most widely used and suitable model for watershed routing, it was used in the generation of catchment runoff.
Over intervening years to present, ILSAX, DRAINS are widely used models in Australia. They also utilise the basic \(TA\) method to estimate both impervious surface and pervious surface runoff. In addition, Kull and Feldman (1998) found that 40% of the US Corps of Engineers projects use the \(TA\) technique in the rainfall-runoff model. The general equation of the \(TA\) method which gives a net hydrograph due to rainfall excess can be written as follows:

\[
(Q_s)_j = \sum_{k=1}^{j} (RE)_k (A_{sub})_{j-k+1}
\]  

(3.5)

Where, \(j\) is the time step and \(A_{sub}\) is the area between two consecutive isochrones.

Using Equation 3.5, the developed model generates separate hydrographs for impervious and pervious surfaces. These hydrographs are combined to calculate the total runoff hydrograph. According to Equation 3.5, the watershed area should first be divided into sub-areas before applying the \(TA\) method.

### 3.2.1.3 Catchment Sub-division

Since the entire catchment area does not contribute to surface runoff at the same time during a rainfall event, the area is divided into a suitable number of sub-areas to describe the proper watershed responses. Various sub-catchment flows are combined and routed through the system to a catchment outlet. However, it is unfeasible to sub-divide a catchment into an infinitive number of sub-catchments. In this study, the fundamentals of catchment sub-area division were based on the \(t_c\) because it is assumed that after the \(t_c\) the entire area of a catchment contributes to surface runoff. The model was developed by dividing a catchment into sub-catchments along watershed lines (isochrones lines). The developed model has the two different options for the division of a particular catchment area into sub-areas:

- Sub-area division by the user
- Sub-area division by the model

The users have the choice to select any of these two procedures based on available data. When isochrones data are available for a particular catchment, the users can divide the entire catchment area into sub-areas from these isochrones data. The developed model has an option to account sub-areas and corresponding imperviousness.
However, where isochrones data are not available, several empirical methods were developed for the delineation of isochrones. For this thesis, the empirical technique from the HEC-1 was used for the automatic sub-division of a particular catchment by the model. Since this technique gives the better results for watershed area sub-divisions (Shokoohi 2008), it was adopted in the hydrological model development. In the HEC-1 user manual, it was noted that this empirical technique is useful for watershed areas, where physiographic data are not available. For this technique, the watershed area is assumed to be a parabola. Where sub-catchment areas are not available, the program utilises a dimensionless \( TA \) curve as follows:

\[
A_D = 1.414 t_{DC}^{1.5} \quad \left(0 \leq t_{DC} < 0.5\right) \tag{3.6}
\]

\[
A_D = 1 - 1.414 \left(1 - t_{DC}\right)^{1.5} \quad \left(0.5 \leq t_{DC} < 1.0\right) \tag{3.7}
\]

Where, \( A_D \) is the dimensionless catchment area, which is the proportion of the considered area to the total area, and \( t_{DC} \) is the dimensionless time meaning the proportion of the considered area cumulative time to the \( t_c \). The whole procedure can be described as follows. From Equation 3.6

\[
A_D = 1.414 t_{DC}^{1.5}
\]

\[
\Rightarrow \frac{A_i}{A_T} = A = 1.414 t_{DC}^{1.5}
\]

\[
\Rightarrow A_i = 1.414 t_{DC}^{1.5} \times A_T
\]

\[
\Rightarrow A_i = 1.414 \left(\frac{\Delta t}{t_c}\right)^{1.5} \times A_T
\]

\[
\Rightarrow A_1 = 1.414 \left(\frac{1 \times \Delta t}{t_c}\right)^{1.5} \times A_T
\]

\[
\Rightarrow A_2 = 1.414 \left(\frac{2 \times \Delta t}{t_c}\right)^{1.5} \times A_T
\]
\[ \Rightarrow A_3 = 1.414 \left( \frac{3 \Delta t}{t_c} \right)^{1.5} * A_T \]

........................................................

........................................................

\[ \Rightarrow A_4 = 1.414 \left( \frac{i \Delta t}{t_c} \right)^{1.5} * A_T \]

From Equation 3.7

\[ A_D = 1 - 1.414(1-t_{DC})^{1.5} \]

\[ \Rightarrow \frac{A_1}{A_T} = A_D = 1 - 1.414(1-t_{DC})^{1.5} \]

\[ \Rightarrow A_1 = \left(1 - 1.414(1-t_{DC})^{1.5}\right) * A_T \]

\[ \Rightarrow A_1 = \left\{1 - 1.414 \left(1-\frac{\Delta t}{t_c}\right)^{1.5}\right\} * A_T \]

\[ \Rightarrow A_1 = \left\{1 - 1.414 \left(1-\frac{1 \Delta t}{t_c}\right)^{1.5}\right\} * A_T \]

\[ \Rightarrow A_2 = \left\{1 - 1.414 \left(1-\frac{2 \Delta t}{t_c}\right)^{1.5}\right\} * A_T \]

\[ \Rightarrow A_3 = \left\{1 - 1.414 \left(1-\frac{3 \Delta t}{t_c}\right)^{1.5}\right\} * A_T \]

........................................................

........................................................
\[ A_i = \left\{ 1 - 1.414 \left( 1 - \frac{i \cdot \Delta t}{t_c} \right)^{1.5} \right\} \cdot A_T \]

Where,

\[ A_T = \sum_{i=1}^{n} A_i \]  

\( \Delta t \)  Travel time of water parcel between two isochrones

3.2.1.4 Time of Concentration

Recognising the importance of the \( t_c \) in the hydrologic design and calculation, the hydrologists have developed numerous methods to calculate the \( t_c \) for a particular point (generally the furthest) within a particular watershed area to the outlet (Fang et al. 2007). However, these methods are not based on the theory of fluid mechanics. They were developed based on the analysis of one or more data sets and called empirical methods. Although these developed equations are empirical, many of them generally require the use of lumped basin parameters, such as basin area, flow path length, average slope, Manning’s ‘n’, channel factors, runoff coefficient, rainfall intensity, hydraulic radius. Some of the \( t_c \) computation methods are single, lumped parameter empirical equations and others divide the flow path into different hydraulic flow conditions and then sum the travel time from separate segments of flow. The developed model allows the users to select any of the three established methods for the calculation of the \( t_c \) for a particular catchment. These methods are widely used in Australia and can be discussed as follows.

Where a complete procedure based on the observed data is not available, the Bransby Williams formula was adopted as an arbitrary but reasonable approach. The method is described as follows:

\[ t_c = \frac{58L}{A_T^{0.1} S_e^{0.5}} \]  

Where, \( t_c \) is the time of concentration (minutes), ‘\( L \)’ is the length of the main channel (km), \( A_T \) is the area of the watershed in \( km^2 \) and \( S_e \) is the equal area slope of the main stream projected to the catchment outlet (m/km).
The Pilgrim and McDonald formula developed in 1982 is based on the typical conditions in eastern New South Wales, Australia. This method is also used in Victoria for the estimation of the peak discharge of a rainfall event. The formula was developed for the estimation of critical rainfall duration as the $t_c$. Pilgrim and McDonald (1982) recommended using this formula for a catchment area of up to $250 \text{ km}^2$. The formula is expressed as:

$$ t_c = 45.6 A_{r}^{0.38} \quad (3.10) $$

In the south-western part of Western Australia, the Flavelt (1983) formula is recommended. This area includes jarrah forest with lateritic and loamy soils, low jarrah forest with sandy soils and karri forest with loamy soils. The expression for the Flavelt formula is:

$$ t_c = 138.6 A_{r}^{0.54} \quad (3.11) $$

### 3.2.2 Pollutant Model

Over recent years, water resources management authorities emphasised the need for the development of strategies for water quantity and water quality together (Fulazzaky et al. 2010). In this study, the pollutant component of the catchment water quality model was developed and integrated with the developed hydrologic model to present a meaningful catchment water quality response. The pollutant model consists of two main functional sub-components, i.e. the pollutant build-up and the pollutant wash-off. The build-up model estimates the accumulation of water quality parameters within a catchment surface during the $t_d$. The wash-off model estimates the amount of transferred pollutants into nearby waterways and receiving water bodies with surface runoff.

According to the research literature, there are various techniques available for the estimation of pollutant build-up and wash-off from a particular catchment surface. Ball et al. (1998) suggested that instead of using one model, alternative models with different degrees of complexity and computation effort should be employed for the prediction of water quality parameters. The best selected model depends on the information needed which differs for each water ecosystem, individual management alternatives, and different institutional objectives and constraints (Laenen and Dunnette 1997).
Most of the currently available stormwater quality models were developed for impervious surface only. Stormwater quality models for pervious area are very limited. However, water quality parameters from pervious area also play an important role in polluting aquatic environments (Deletic and Maksimovic 1998; Grottker 1987). Therefore, water quality parameters from pervious surface area should be considered in any water quality study. In this study, the catchment water quality model was developed by considering the three different types of surface areas: (i) impervious surface, (ii) pervious surface and (iii) mixed surfaces (partly impervious and partly pervious).

The model allows the users to use different build-up and wash-off coefficients for impervious and pervious surface of a particular catchment area. These coefficients for build-up and wash-off models are determined by the land-use of those areas from observed or experimental water quality data. Finally, based on the calculated coefficients, the model estimates pollutant loads or concentrations at the catchment outlet.

For the practical application of the model, the minimum allowable value of the $t_d$ was assumed to one day (24 hrs). This means that if the $t_d$ was less than 24 hrs, pollutant build-up on catchment surface was negligible and assumed to be zero. Keeping in mind our objectives, it seemed logical to take the model similar to the widely used model SWMM as the basis for the catchment water quality model. The concept of SWMM was considered because adoption of a widely used equation gives confidence to the practical application of any water quality model (Boorman 2003). The main mathematical processes adopted in developing both pollutant build-up and wash-off models is described as follows.

3.2.2.1 Pollutant Build-up Model

The $t_d$ is expected to have a significant influence on the accumulation of water quality parameters on catchment surfaces. However, pollutant build-up on catchment surfaces continues during rainfall events (Bouteligier et al. 2002). The effect of build-up at this time period is negligible; and hence can be discarded for the simplicity. Therefore, Chui (1997) noted that the most important parameter for modelling pollutants build-up in a particular catchment surface is the $t_d$. 

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In this research project, the explicit relationship between the pollutant accumulation and the $t_d$ was used for the development of the build-up model. The build-up model was developed based on the assumption that on a clean surface, water quality parameters accumulate until a maximum value is attained. Considering the $t_d$ as the key variable including other hypotheses, the developed catchment water quality model described pollutant build-up by the three separate functions for impervious and pervious surfaces of a particular catchment:

- Power function
- Exponential function
- Saturation function

With the nature of the pollutant accumulation, an efficient, reliable and convenient model can be selected from the above mentioned functions.

3.2.2.1.1 Power Function Build-up Model

The power function pollutant build-up equation is the best generic form for the accumulation of water quality parameters on catchment surfaces (Owusu-Asante and Stephenson 2006; Ball et al. 1998). The power function build-up model considers that the pollutant build-up on a catchment surface increase with the increasing number of dry days until a maximum limit is reached (Egodawatta 2007; Rossman 2004; Ball et al. 1998). The power function pollutant build-up model is described according to the following Equation 3.12:

$$B_{iy}(p,s) = \text{Min} \left[ \frac{A_i(s) \times F_{imp}(s) \times C_1(p,s)}{A_i(s) \times F_{imp}(s) \times C_2(p,s) \times t_d C_3(p,s) + \left( B_{iy}(p,s) \right)} \right]$$

(3.12)

Where, $B_{iy}(p,s)$ is the accumulation of the pollutant ‘$p$’ (kg) on the land surface ‘$s$’ during the $t_d$, $A_i(s)$ is the contributing area of the catchment (km$^2$), $F_{imp}(s)$ is the impervious or pervious fraction of the land surface ‘$s$’, $C_1(p,s)$ is the maximum amount of pollutant that can be accumulated on the land surface ‘$s$’ (kg/km$^2$), $C_2(p,s)$ is the coefficient for the pollutant build-up parameter (kg/km$^2$.day$^{-C_3}$), $C_3(p,s)$ is the exponent for the pollutant build-up parameter and $t_d$ is the number of antecedent dry days.
3.2.2.1.2 Exponential Function Build-up

The exponential pollutant build-up model assumes that the accumulation of pollutants on a catchment surface occurs as an exponential function with an upper limit asymptotically (Rossman 2004). Many catchment water quality models employ the exponential build-up representation because it is simple and can be derived as a first order process (Kim et al. 2006). In this thesis, the exponential build-up model was developed as a function of build-up rate coefficient and the \( t_d \) as follows:

\[
B_{t_d}(p, s) = \text{Min} \left[ \left( A_x(s) \times F_{imp}(s) \times C_i(p, s) \right), \left( A_x(s) \times F_{imp}(s) \times C_i(p, s) \times \left( 1 - e^{-k_p t_d} \right) \right) + \left( B_{t_d} \right)_d(p, s) \right] \tag{3.13}
\]

Where, \( k_p \) is the pollutant accumulation rate coefficient (1/day).

3.2.2.1.3 Saturation Function Build-up

The saturation build-up function assumes that the accumulation of pollutants on a particular catchment surface occurs as a linear rate and continuously declines until the saturation value is reached (Rossman 2004). The saturation build-up function is presented as follows:

\[
B_{t_d}(p, s) = \text{Min} \left[ \frac{A_x(s) \times F_{imp}(s) \times C_i(p, s) \times \left( t_p(p, s) + t_d \right)}{t_p(p, s) + t_d} + \left( B_{t_d} \right)_d(p, s) \right] \tag{3.14}
\]

Where, \( t_p(p, s) \) is the half saturation constant (the days to reach half of the maximum build-up) for the pollutant ‘p’ to the land surface ‘s’.

3.2.2.2 Pollutant Wash-off Model

The pollutant wash-off model simulates the removal of the accumulated pollutants from a particular catchment surface into nearby waterways and receiving water bodies during rainfall events. However, in the areas with larger rainfall events, the accumulated pollutants can be washed-off completely by more frequent storms. As a result, the succeeding storm may produce the same or even more runoff as the preceding storm but may produce considerably smaller storm pollutant loads (Barbe et al. 1996).
From the literature review of the thesis, it was understood that the amount of pollutant wash-off is significantly influenced by the available pollutants on a particular catchment surface. Therefore, the pollutant wash-off model of this research project was developed by the integration of the rainfall-runoff transformation and pollutant build-up model. The wash-off process follows a suitable depletion law and thus influences the actual pollutant concentration that is transported through a catchment outlet into nearby waterways and receiving water bodies. The pollutant wash-off approaches used in this study are based on the model developed by Rossman (2004). Similar to the pollutant build-up model, the wash-off model was presented by three different functions as follows:

- Power function wash-off
- Rating curve wash-off
- Exponential function wash-off

These three wash-off functions represent the pollutant transport capacity of surface runoff from a particular catchment area (Barbe et al. 1996). These functions were developed to consider the wash-off from impervious and pervious surfaces separately. These functions consider the three influential variables, i.e. the rainfall intensity, the rainfall duration and the runoff volume. Due to their highly correlation with each other, the individual influence cannot be clearly discerned (Chiew and McMahon 1999; Chui 1997).

3.2.2.2.1 Power Function Wash-off

The power function wash-off model assumes that the amount of pollutants removed by stormwater runoff from a particular catchment surface is proportional to the product of the runoff rate per unit area raised to some power and the available pollutants on that catchment surface.

In this study, the power function pollutant wash-off model was developed by assuming that the removal of stormwater pollutant as proportional to the amount available at a catchment surface and directly related to surface runoff discharged from that catchment. According to this assumption, mathematical expression of the power function wash-up model is expressed as follows:
\[ W_t(p,s) = \frac{E_1(p,s)\left(q_{A,s}(s)\right)^{E_2(p,s)} B_{V_t}(p,s)}{1000V_{T_5}(s)} \] (3.15)

Where, \( W_t(p,s) \) is the wash-off rate (mg/l) for the pollutant ‘p’ from the land surface ‘s’ within time ‘t’ during surface runoff event, \( E_1(p,s) \) is the pollutant wash-off coefficient \( \left( \frac{mm}{hr} \right)^{E_2} \), \( q_{A,s}(s) \) is the runoff rate per unit area (mm/hr), \( E_2(p,s) \) is the pollutant wash-off exponent \( \text{dimensionless} \) and \( V_{T_5}(s) \) is the volume of surface runoff \( (m^3) \) within time ‘t’.

3.2.2.2.2 Rating Curve Wash-off

The rating curve wash-off model estimates the transportation of water quality parameters as a function of surface runoff discharged from a particular catchment surface. A number of previous studies e.g. Charbeneau and Barrett (1998); Barbe et al. (1996) found that pollutants wash-off can be better estimated by using runoff volume. Chiew and McMahon (1999) also established that the pollutant concentration in relation to the runoff rate provides a better prediction for some catchments. In addition, Deletic and Maksimovic (1998) and Chui (1997) found that pollutants wash-off from catchment surfaces strongly depends on the overland flow. Furthermore, Huber and Dickson (1988) noted that in the catchment water quality models, the relationship between the pollutant wash-off and the runoff volume is the most convenient and easiest equation. Another thing is that data for water quality parameters and runoff volume are easily available.

According to the rating curve wash-off function, the amount of transported pollutant from a particular catchment surface can be expressed as proportional to the surface runoff rate raised to some power. The rating curve wash-off model employed in this this thesis is expressed as follows:

\[ W_t(p,s) = \frac{E_3(p,s)\left(Q_t(s)\right)^{E_4(p,s)}}{1000V_{T_5}(s)} \] (3.16)

Where, \( E_3(p,s) \) is the coefficient for the pollutant wash-off parameter \( \left( kg \cdot m^3/s \right)^{E_4} \), \( E_4(p,s) \) is the exponent or power of the wash-off parameter \( \text{dimensionless} \) and \( Q_t(s) \) is the runoff rate \( (m^3/s) \) from the land surface ‘s’ at time ‘t’.
3.2.2.2.3 Exponential Wash-off

The exponential wash-off model estimates the decreasing pollutant concentration of a given accumulated pollutant with increasing time since the start of a rainfall event. Different derivations of this equation are used in the various stormwater quality models. The most common form of the exponential function assumes that the rate of pollutant wash-off from a particular catchment surface is proportional to the product of available pollutant which remains on that surface and the rainfall intensity. The expression can be written as follows.

\[
\frac{dW}{dt} = -\alpha_w W
\]  

(3.17)

Where, ‘W’ is the pollutant wash-off and \( \alpha_w \) is the wash-off rate constant.

Assuming \( \alpha_w \) varies with the direct proportion to the rainfall intensity, \( I(s) \) i.e. \( \alpha_w = E_5(p,s)I(s) \), Equation 3.17 becomes:

\[
\frac{dW}{dt} = -E_5(p,s)I(s)W
\]

Integrating the equation with respect to ‘t’:

\[
\int dW = -E_5(p,s)I(s)W dt
\]

\[
\Rightarrow \int \left( \frac{dW}{W} \right) = -E_5(p,s)I(s) \int dt
\]

\[
\Rightarrow \ln W = -E_5(p,s)I(s)t + C_{\text{const}}
\]  

(3.18)

When the time, \( t = 0 \), then \( W = B_{i_s} \) and the value of the constant become:

\[
C_{\text{const}} = \ln(B_{i_s})
\]

Substituting the value of the constant in Equation 3.18 one can obtain:

\[
\ln(W) = -E_5(p,s)I(s)t + \ln(B_{i_s})
\]

\[
\Rightarrow \ln(W) - \ln(B_{i_s}) = -E_5(p,s)I(s)t
\]
\[
\Rightarrow \ln \left( \frac{W}{B_{t_i}} \right) = -E_z(p,s)I(s)t
\]

\[
\Rightarrow \frac{W}{B_{t_i}} = \exp\left[ -E_z(p,s)I(s)t \right]
\]

\[
\Rightarrow W = \exp\left[ -E_z(p,s)I(s)t \right]B_{t_i}
\]

Therefore, the amount of pollutants removed from a particular catchment surface at any time ‘t’:

\[
W_t = B_{t_i} - W
\]

\[
\Rightarrow W_t = B_{t_i} - \left[ \exp\left[ -E_z(p,s)I(s)t \right]B_{t_i} \right]
\]

\[
\Rightarrow W_t = B_{t_i} \left[ 1 - \exp\left[ -E_z(p,s)I(s)t \right] \right]
\]

More specifically the equation can be written as follows:

\[
\Rightarrow W_t = \frac{B_{t_i} \left[ 1 - \exp\left[ -E_z(p,s)I(s)t \right] \right]}{\delta_{conv}}
\]

Where, \( W_t \) is the pollutant wash-off within time ‘t’ and \( \delta_{conv} \) is the conversion factor.

\[
\delta_{conv} = \begin{cases} 
1000V_{Ts} & \text{When } W_t \text{ is in concentration} \\
1 & \text{When } W_t \text{ is in loads}
\end{cases}
\]

In this research study, the exponential wash-off model is expressed as follows:

\[
W_t(p,s) = \frac{\left(1 - e^{-E_z(p,s)I(s)t}\right)B_{t_i}(p,s)}{1000V_{Ts}(s)}
\]  

(3.19)

Where, \( E_z(p,s) \) is the wash-off exponent (1/mm) for the pollutant ‘p’ from the land surface ‘s’ and \( I(s) \) is the rainfall intensity (mm/hr) on the land surface ‘s’.
3.2.2.3 More than one Catchment

The above models have been developed only based on single catchments. However in reality, there might be more than one catchment contributing to surface runoff and pollutant loads. For these cases, the integrated model was developed based on the superposition theory which estimates the final runoff and the pollutant concentration at the catchment outlet.

For example, at any time $t_1$, two catchments are contributing to surface runoff and pollutant loads. The ordinates of the hydrograph and pollutograph for the first catchment are $Q_1$ and $C_1$ respectively. The ordinates of the hydrograph and pollutograph for the second catchment are $Q_2$ and $C_2$ respectively. For the time $t_1$, the combined runoff and pollutant concentration are expressed as follows:

\[
Q_{combined} = Q_1 + Q_2
\]  
\[
C_{combined} = \frac{C_1 Q_1 + C_2 Q_2}{Q_1 + Q_2}
\]  

(3.20)  
(3.21)

To estimate the ordinates of the final hydrograph and pollutograph, the developed integrated model uses these Equations 3.20 and 3.21 respectively.

3.3 Stream Water Quality Model

Streamflow is the major contributor for the transportation and interaction of water quality parameters along a particular stream reach (Ali et al. 2010). Therefore, the primary objective of the developed stream water quality model was to produce a tool that has the capability to simulate the water quantity and water quality of a particular stream system. The developed model is one-dimensional and simulates concurrently two main processes, i.e. the streamflow process and the water quality processes. The flow model estimates the rate of stream discharges in a particular stream reach and the quality model estimates the concentration of stream water quality parameters in that stream reach. In the developed model, a particular stream reach can be divided into subsections at which the geometric characteristics of the stream bed and the hydraulic characteristics of flow are held to be constant. The outputs from one sub-reach are used as the input for the next one.
The stream water quality model was developed with the same water quality parameters (i.e. SS, TN and TP) of the catchment water quality model. However, for the survival of fish species, and other aquatic plants and organisms, DO is significantly important parameter for all waters. In addition, within streams biochemical oxygen demand (BOD) is another important parameter for the simulation of accurate DO in stream water (Boorman 2003). Therefore, these two parameters (i.e. DO and BOD) were incorporated in the development of the stream water quality model with the catchment water quality model’s pollutants.

A conflicting issue in the current water quantity and quality modelling techniques is that the hydraulic radius is assumed to be equal to the flow depth. This assumption is valid for the wide streams. For the streams with small width, this assumption leads to the formation of gross error. To avoid discrepancy in this study, the hydraulic radius of a particular stream reach is determined from the depth of flow by introducing a compensation factor.

3.3.1 Streamflow Model

The streamflow model was developed by considering the well-known Muskingum-Cunge method of stream routing. The major advantage of the Muskingum-Cunge method is that it allows better definition of the physical characteristics of a particular stream reach (Seybert 2006). Since the method is simple, efficient and relatively accurate, it has been widely accepted and used in practice (Ponce et al. 1996; Barry and Bajracharya 1995). The method was theoretically derived by Cunge (1969) by assuming and strictly applying one-to-one stage-discharge relationship. A brief description of the Muskingum-Cunge method is described as follows.

The equation of continuity (or storage equation) for unsteady flow through a stream reach can be presented as:

\[ Q_{in} - Q_{out} = \frac{dS_{csv}}{dt} \]  \hspace{1cm} (3.22)

Where, \( Q_{in} \) is the inflow rate into a stream reach at the upstream section (m³/s) and \( Q_{out} \) is the outflow rate from that reach to the downstream section (m³/s).
The routing procedure begins by dividing the routing time into a number of equal time intervals $\Delta t_R$, and expressing Equation 3.22 in the finite difference form. Using the subscripts $j'$ and $j+1$ for the beginning and the end of the time step $\Delta t_R$, one can obtain the following Equation 3.23:

$$\frac{Q_{in}^j + Q_{in}^{j+1}}{2} - \frac{Q_{out}^{j+1} + Q_{out}^j}{2} = \frac{S_{csv}^{j+1} - S_{csv}^j}{\Delta t_R} \quad (3.23)$$

The volume of water storage within a stream reach can be expressed as a function of flow rate entering and leaving that reach. The mathematical expression of the storage volume is given by Equation 3.24:

$$S_{csv} = K_{stor} [X_p \times Q_{in} + (1-X_p)Q_{out}] \quad (3.24)$$

Combining these two Equations 3.22 and 3.23:

$$\Delta t_R \left( Q_{in}^{j+1} - Q_{out}^{j+1} + Q_{in}^{j} - Q_{out}^{j} \right)$$

$$= 2K_{stor} \left[ X_p \times Q_{in}^{j+1} + (1-X_p)Q_{out}^{j+1} - X_p \times Q_{in}^{j} - (1-X_p)Q_{out}^{j} \right] \quad (3.25)$$

Rearranging this Equation 3.25 and introducing Muskingum-Cunge coefficients, we will get the following expression:

$$Q_{out}^{j+1} = C_{MC0} Q_{in}^{j+1} + C_{MC1} Q_{in}^{j} + C_{MC2} Q_{out}^{j} \quad (3.26)$$

Where, Muskingum-Cunge coefficients $C_{MC0}$, $C_{MC1}$ and $C_{MC2}$ are expressed as follows:

$$C_{MC0} = \frac{\Delta t_R - 2K_{stor} X_p}{2K_{stor}(1-X_p) + \Delta t_R} \quad (3.27)$$

$$C_{MC1} = \frac{\Delta t_R + 2K_{stor} X_p}{2K_{stor}(1-X_p) + \Delta t_R} \quad (3.28)$$

$$C_{MC2} = \frac{2K_{stor}(1-X_p) - \Delta t_R}{2K_{stor}(1-X_p) + \Delta t_R} \quad (3.29)$$

At any time step $j'$, all terms on the right hand side of Equation 3.26 are known. Hence, the ordinates of the outflow hydrograph at the next time step $j+1$ can be determined.
In some cases, the continuously varying storage delay time with discharge is awkward. However, this difficulty is overcome if the delay time is thought of as reach length divided by the wave celerity which varies continuously with the discharge. This concept is useful in understanding the non-linear routing processes. For the estimation of the variable $K_{stor}$, the developed model adopted the following equation:

$$K_{stor} \approx \frac{\Delta x_{SR}}{c_k} = \frac{\Delta x_{SR}}{\sqrt{gh}}$$

(3.30)

### 3.3.2 Stream Pollutant Model

The quality component of the stream water quality model was developed to describe and predict the observed effect of the change in water quality parameters within a stream system. The model was developed by considering the physical, chemical and biological changes of stream water quality parameters. The idea was to help in developing an effective stream water quality management tool for the protection and improvement of aquatic ecosystems of stream environments from the impact of water pollution.

The construction of the stream pollutant model started with the development of the $SS$ processes modelling. The main processes involved in the $SS$ simulation are the deposition and transportation. Since other pollutants are related to $SS$, the model for $SS$ was developed first. After the development of the $SS$ model, the $BOD$, $TN$ and $TP$ models were developed for the continuous simulation of these water quality parameters. There are two different options for the input parameters of the model. In the first option, the input pollutant loads are coming from the developed catchment water quality model; while in the second option, the input pollutants data are given by the users for an upstream section of a particular stream reach. Moreover, the temperature effect of the rate parameters was also considered in the development of the stream water quality model because the temperature has a moderating effect on many stream water quality parameters. The model was developed for the prediction of the mentioned water quality parameters to guide management authorities for the implementation of appropriate management options.
3.3.2.1 Suspended Sediment Model

It is widely recognised that the water quality parameter, suspended sediment (SS) is considered to be one of the major water pollutants of streams. Therefore, measurement of SS is important for the management of stream environments. However, the spatial patterns of the hydraulic variables (e.g. flow depth and velocity) and the sedimentological variables (SS concentration and deposition) are extremely complex to an accurate estimation of SS (Nicholas and Walling 1998). Now, there are only empirical models available and their solutions are performed based on the certain assumptions. Therefore, further research on the SS processes is still necessary to better understand these processes.

In this study, the SS model was developed based on the concept of streamflow, the bottom boundary condition, the near-bed reference concentration, the transportable sediment concentration, the critical bed-shear stress for deposition, and critical flow velocity. The model is one-dimensional and simulates concurrently two main processes, i.e. the sediment transport and the particle deposition. For the development of the SS model, widely used equations were modified and used for the simulation of SS transportation and deposition. The transport model estimates the transportation rate of SS for a particular stream reach under the given characteristics of that reach. The transport model proposed by van Rijn (1984a, 1993) was modified and used in this study. The deposition model calculates the deposition of SS based on the characteristics of sediment particles.

The model was developed based on the following assumptions: (1) width of the stream is not too large compared to the depth of flow (i.e. it is not wide channel), (2) the inflow sediment is coming only after rainfall events and (3) there is no re-suspension of particles already deposited onto the stream bed. The first assumption was realistic to the Australian conditions; because most of the creeks/streams in Australia have a relatively small width. The second assumption was again based on the Australian conditions. Most of the streams in Australia have no base flow and there is flow only during a rainfall event. Hence, no sediment is transported during the dry periods, i.e. days without rain. The third assumption was made to make the model structure simple and user friendly.
The advantage of the model is that it can be applied to all streams for the prediction of non-cohesive SS transportation and deposition along a particular stream reach. The model was developed as a tool for the routine morphological computations in daily water resources engineering practices. Practical problems such as sedimentation in dredged channels, trenches, reservoirs and basins can be benefited through the use of such a model. For a known particle size of the inflow sediment, the developed model is capable of predicting the transportation and deposition rate of SS to a particular stream reach.

3.3.2.1.1 Deposition of SS

The main factors determining the deposition of SS are low flow velocity and the amount transported from upstream catchments (Rimkus et al. 2007). There is high concentration of SS close to the bottom of stream and the thickness of this layer increases when the sediment concentration of the entire flow increases. The saturation degree of the sediment at the bottom layer regulates the deposition rate. However, high flow velocity and turbulence hinder the sediment particles from settling onto the stream bottom. The deposition begins when the concentration of sediment in the bottom layer becomes high enough to dampen the turbulence to a necessary degree. Decrease in the turbulence at the bottom layer allows sediment particles to fall to the bed of stream. Therefore, the critical velocity and the transportable sediment concentration are used for the calculation of the SS deposition (Rimkus and Vaikasas 1999).

The deposition of SS in a stream bed can be derived from the concept of the transportable sediment concentration, the critical bed-shear stress, or critical flow velocity (Vaikasas and Rimkus 2003). The mathematical expression of SS deposition can be written as follows:

\[
(SSD)_t = \frac{(q_{ss,in})_t - (q_{ss,tr})_t}{(q_w)_t} \cdot w_s, \tag{3.31}
\]

Where, \((SSD)_t\) is the deposition rate of SS per unit of bottom area (kg/m²/s) at time ‘t’, \((q_{ss,in})_t\) is the inflow rate of SS (kg/m/s) per unit width at time ‘t’, \((q_{ss,tr})_t\) is the SS transport rate per unit width (kg/m/s) at time ‘t’ and \((q_w)_t\) is the water flow discharge per unit width (m²/s) at time ‘t’. 

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When applying the above Equation 3.31, one must know the stream reach width because the input parameters \((q_{ss,in})_t\), \((q_{ss,tr})_t\), and \((q_w)_t\) should be given per unit width of a stream reach. However, the usual technique for the measurement of \((q_{ss,in})_t\) is volumetric, i.e. in \(\text{kg/m}^3\). According to van Rijn (1993), the behaviour of the SS particles is described in terms of the sediment concentration which is solid volume \((m^3)\) per unit fluid volume or the solid mass \((kg)\) per unit fluid volume \((m^3)\). Nevertheless, the transport rate \((q_{ss,tr})_t\) can be determined directly for per unit width from the well-known equations, such as van Rijn (1984a, 1993), Bagnold (1966). On the other hand, the runoff rate is usually measured volumetrically, e.g. in \(\text{m}^3/\text{s}\). Hence, for the calculation of \((q_{ss,in})_t\) and \((q_w)_t\), the estimation of stream reach width is mandatory. Therefore, it is difficult to apply the above Equation 3.31 with the available sediment and runoff measurement techniques without estimating the channel width. However, the measurement of a channel width is not so easy because the width of a particular stream is not fixed along its length. It can vary from the upstream to the downstream depending upon the characteristics of stream reach. Hence, the above equation needs modification to use with available data so that it can be used without estimating the stream width. For the modification of Equation 3.31, it is desirable to have a better understanding of the transport process of sediment in suspension and the capability of models to simulate this process as accurately as possible (Lin and Namin 2005).

Multiplying by the channel width to the numerator and denominator of the right side of Equation 3.31 and rearranging we can get:

\[
(SS_D)_i = \frac{[\{(q_{ss,in})_t - (q_{ss,tr})_t\} \times w_s \times B_w}{(q_w)_t \times B_w}
\]

\[
(SS_D)_i = \frac{\{(Q_{ss,in})_t - (Q_{ss,tr})_t\} \times w_s}{(Q_w)_t}
\]

\[
(SS_D)_i = \left[\frac{(Q_{ss,in})_t}{Q_w} - \frac{(Q_{ss,tr})_t}{Q_w}\right] \times w_s
\]

\[
(SS_D)_i = \{(inSS)_i - (trSS)_i\} \times w_s \tag{3.32}
\]
Where, $B_W$ is the width of a particular stream reach, $(inSS)_t$ is the concentration of inflow sediment (kg/m$^3$) at time ‘$t$’ and $(trSS)_t$ is the SS transport rate (kg/m$^3$) at time ‘$t$’.

Again, multiplying by the width of the stream and considered length of the reach section to the both sides of Equation 3.32:

$$(SS_{D,t})_t \times B_W \times \Delta L = \{(inSS)_t - (trSS)_t\} \times w_s \times B_w \times \Delta L$$

$$\Rightarrow (SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times B_w \times \Delta L \times \frac{(Q_{str})_t}{(Q_{str})_t}$$

$$\Rightarrow (SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times B_w \times \Delta L \times \frac{(Q_{str})_t}{Q_{str}}$$

$$\Rightarrow (SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times \Delta L \times \frac{(Q_{str})_t}{q_t}$$

$$\Rightarrow (SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times \Delta L \times \frac{(Q_{str})_t}{\bar{u}_t h_t} \quad (3.33)$$

Where, $(SS_{D,S})_t$ is the sediment deposition rate per second (kg/s) at time ‘$t$’, $\Delta L$ is the considered length of the stream reach (m), $(Q_{str})_t$ is the streamflow rate (m$^3$/s) at time ‘$t$’, $\bar{u}_t$ is the depth average flow velocity (m/s) of the stream at time ‘$t$’ and $h_t$ is the depth of flow (m) at time ‘$t$’.

Applying the Chezy formula for the calculation of the velocity component ($\bar{u}_t$) of the model in Equation 3.33 one can obtain:

$$(SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times \Delta L \times \frac{(Q_{str})_t}{C_t (h_t S_0)^{0.5} h_t}$$

$$\Rightarrow (SS_{D,S})_t = \{(inSS)_t - (trSS)_t\} \times w_s \times \Delta L \times \frac{(Q_{str})_t}{C_t h_t^{0.5} S_0^{0.5}} \quad (3.34)$$

Where, $C_t$ is the overall Chezy coefficient (m$^{0.5}$/s) at time ‘$t$’.
The above modified Equation 3.34 estimates the sediment deposition rate without knowing the stream width. For the application of this Equation 3.34, the incoming SS concentration \((inSS)\) is determined from SS data measured at the catchment outlet.

Applying the Stricker formula for Chezy coefficient and introducing the new term compensation factor for the calculation of the hydraulic radius we get:

\[
(SSD,t) = \left[ \frac{(inSS) - (trSS)}{w_t \times \Delta L \times Q} \right] \times \left[ \frac{1}{(CF)^{\frac{1}{6}} h^{0.5}} \right] S_0
\]

\[
(SSD,t) = \left[ \frac{(inSS) - (trSS)}{w_t \times \Delta L \times Q \times n} \right] \times \left[ \frac{1}{(CF)^{\frac{1}{6}} h^{0.5+CF/6}} S_0^{0.5} \right] \tag{3.35}
\]

Where, \(CF_c\) is the coefficient of the compensation factor and \(CF_e\) is the exponent of the compensation factor.

3.3.2.1.2 Transportation of SS

An essential part of the morphological computations is the estimation of the transported SS along a particular stream reach. However, the direct determination of SS from the experimental results on a wide scale is labour intensive, time consuming and expensive. Hence, modelling of the SS transportation has great interest to the engineers, scientists who are involved in the hydraulic engineering and management of river, estuarine and coastal water. Modelling of the SS transport rate is also important in the prediction of other pollutants, such as nutrients and heavy metal fluxes (Zhou et al. 2003).

A number of simulation models (van Rijn 1984a, 1993; Bagnold 1966) have been developed to estimate the SS transport rate of a particular stream reach, i.e. to estimate \((q_{ss,tr})\) of Equation 3.31; because it depends upon a number of factors that forms the flow structure. The modelling approaches vary from the simple to the complex. However, the simple modelling approaches with the sufficient accuracy are preferred by the engineers. Nevertheless, there are only limited numbers of simple models which are available for the generic application of the SS simulation. Due to the lack of accepted fundamental principles and the demand for the practical application, the engineers and scientists have been greatly stimulated to improve the existing modelling approaches.
Keeping in mind our objectives, it seemed logical to take the SS transport model similar to van Rijn’s (1984a, 1993) model as the basis of our approach. Since van Rijn’s (1984a, 1993) SS transport formulas are the most common and well-known, they were used in the present study. However, these mathematical expressions proposed by van Rijn calculates the SS transportation rate per unit area of the stream bed. Hence, the practical application of the van Rijn’s expressions requires the estimation of stream reach width which is difficult to determine. For the purpose of this research project, the van Rijn’s formulas were modified and used to be able to continuously simulate SS along a particular stream reach.

The SS transport model was developed based on the streamflow, the bottom boundary conditions and the near-bed reference concentration. The mathematical expression of modified van Rijn’s (1984a, 1993) formula which was used in the development of the SS transport rate can be written as follows:

\[(trSS) = 1000(F_{DS})(c_s), \rho_s\]  

(3.36)

Where, \((trSS)\) is the transport of SS (mg/l) at time ‘t’, \((F_{DS})\) is the shape factor (dimensionless) for the sediment particles at time ‘t’ and \(\rho_s\) is the density of SS particles (kg/m³).

3.3.2.1.3 Shape factor

The practical application of Equation 3.36 requires the estimation of the shape factor \((F_{DS})\), for a particular flow rate. The direct determination of the shape factor estimation by analytical solution is difficult (van Rijn 1993). The calculation of the shape factor can be done from the approximate solution as a function of the flow depth, reference level height and modified suspension number. In this study, this approximate solution was used to develop the SS transport model.

\[(F_{DS}) = \frac{(\delta_i/h_i)^Z - (\delta_i/h_i)^2}{(1 - \delta_i/h_i)^Z(1.2 - Z')}\]  

(3.37)

Where, \(\delta_i\) is the reference level height (m) at time ‘t’ and \(Z’\) is the modified suspension number.
3.3.2.1.4 The Reference Level Height

For the calculation of the shape factor from Equation 3.37, determination of a reference level height is mandatory. Celik and Rodi (1989) also noted that for the determination of the actual SS concentration, a reference level height must be specified. It is assumed that below the reference level height, the transport of all sediment is bed-load transport (van Rijn 1984b). Hence, the determination of a reference level height is one of the essential parameter in sedimentation modelling.

The natural choice of the reference level height is the edge of the bed-load layer. However, this is difficult to determine because there is no clear defined boundary between the bed-load and the suspended load regions. On the other hand, data for the determination of the reference level height is not easily available. Sometimes, the reference level height is assumed to be equal to the roughness height \( k_s \) with a minimum value \( 0.01h_t \). From the detailed experimental research, van Rijn (1986) found that this assumption is reasonable. Moreover, van Rijn (1984b) found that the reference level height smaller than \( 0.01h_t \) leads to the large error in the concentration profile of SS. Therefore, in this study, the value was determined from the depth of flow and Chezy coefficient with a minimum value by the following equation:

\[
\delta_t = \text{Max}\left\{ 12h_t 10^{-C'_t/18}, 0.01h_t \right\} \quad (3.38)
\]

Where, \( C'_t \) is the grain related Chezy coefficient at time ‘t’ which is determined as a function of the depth of flow and the particle properties as follows:

\[
C'_t = 18 \log \left( \frac{12000(CFC_t)h_{t}^{CF_e}}{3D_{90}} \right) \quad (3.39)
\]

Where, \( D_{90} \) is the sediment diameter for which 90% of materials are finer (m). For this study, the value of \( D_{90} \) is given by the user as an input parameter. If data are not available, it can be assumed to 1.5 times of \( D_{50} \) (van Rijn and Walstra 2003). The value of \( h_t \) is determined from the stage-discharge relationship for a particular stream reach section.
3.3.2.1.5 Modified Suspension Number

Similar to the reference level height, the modified suspension number should be determined for the estimation of the shape factor. For the calculation of the modified suspension number van Rijn (1984b) proposed an equation as a function of the suspension number and the stratification correction factor. This van Rijn (1984b) modified suspension number equation was used in the developed SS model. The expression is:

\[ Z' = Z + \phi \]  

(3.40)

Where, ‘Z’ is the suspension parameter (number) and \( \phi \) is the stratification correction factor. The suspension parameter reflects the influence of upward turbulent fluid forces and downward gravitational forces acting on SS and is expressed as follows (van Rijn 1993):

\[ Z = \frac{w_r}{\beta k u_s} \]  

(3.41)

Where, \( \beta \) is the ratio of sediment and fluid mixing coefficients (coefficient related to sediment diffusion of sediment particles), ‘k’ is the Von Karman constant (0.4) and \( u_s \) is the bed shear velocity (m/s).

Due to the inability of sediment particles to respond fully to the turbulent velocity fluctuations, some investigators suggested \( \beta < 1 \); while others suggested \( \beta > 1 \). In the turbulent flow, the centrifugal forces on the sediment particles would be greater than those on fluid particles, thereby causing sediment particles to be thrown outside of the eddies with a consequent increase in the effective mixing length and diffusion rate resulting an increase in the \( \beta \)-factor. However, Coleman (1970) found that the sediment diffusion coefficient is nearly constant in the upper half of the flow for each particular value of \( w_r/u_s \) (van Rijn 1984b). Using the Coleman’s (1970) results, van Rijn (1984b) computed the \( \beta \)-factor as follows:

\[ \beta = 1 + 2 \left( \frac{w_r}{u_s} \right)^2 \text{ for } 0.1 < \frac{w_r}{u_s} < 1 \]  

(3.42)
According to the following expression, the $\beta$-factor is always greater than unity indicating the dominance of the centrifugal force. However, van Rijn (1993) advised that the $\beta$-factor should not be more than 2.

The stratification correction factor ($\phi$) also known as the overall correction factor represents all the additional effects (volume occupied by particles, reduction in particle fall velocity and damping of turbulence) (van Rijn 1984b). The $\phi$-factor expresses the influence of the sediment particles on the turbulence structure of fluid (van Rijn 1993). The effect of this factor is extremely important in the upper regime with the high concentration of SS (> 10 kg/m$^3$) because this affects the damping of turbulence (van Rijn 1993). Analysing the $\phi$ values, van Rijn (1984b) found a simple relationship as:

$$
\phi = 2.5 \left( \frac{w_s}{u_*} \right)^{0.8} \left( \frac{C_s}{C_0} \right)^{0.4} \text{ for } 0.01 \frac{w_s}{u_*} < 1 \text{ and } 0.01 \frac{\delta_*}{h_i} < 1
$$

(3.43)

Where, $c_o$ is the maximum volume concentration (0.65) (Garde and Raju 2000).

3.3.2.1.6 Reference Level Concentration

An essential part of the SS transport computation in an open channel flow is the use of a reference concentration as a bed boundary condition (van Rijn 1984b). However, the most challenging task in SS transport modelling is the determination of the near-bed reference level concentration (Lin and Namin 2005). The reference level concentration also known as the bed level concentration can be prescribed by a known function that relates to the local near-bed hydraulic parameters, sediment diameter and particle Reynolds number.

There are a variety of relationships available in literature for the prediction of the near-bed reference level sediment concentration. Garcia and Parker (1991) carried out a detailed comparison of seven empirical relationships for the prediction of the near-bed sediment concentration. Using one set of common data for which direct measurement of the near-bed sediment concentration were available, they found that two formulae were predicting SS accurately. However, Lin and Namin (2005) noted that for another set of data, other formulae might perform well.
For this research project, the basic idea for the calculation of the SS concentration at the reference level was gained from the research study conducted by van Rijn (1984a, 1984b). The concentration of SS at the reference level was demonstrated in terms of the particle characteristics as follows:

\[
(c_a)_i = 0.000015 \frac{D_{50}}{\delta_i} \cdot T_i^{1.5} \cdot R_{ep}^{0.2} \tag{3.44}
\]

Where, \(D_{50}\) is the median particle diameter of SS (m) which is determined from the field measurements of SS particles, \(T_i\) is the bed shear (transport stage) parameter \((\text{dimensionless})\) at time ‘\(t\)’ and \(R_{ep}\) is the particle Reynolds number.

3.3.2.1.7 Bed-shear Stress Parameter

For the determination of the reference level SS concentration using Equation 3.44, it is necessary to calculate the bed-shear stress parameter \(T_i\). The developed model employed the van Rijn’s (1993) equation for the calculation of the bed-shear stress parameter. The expression can be written as follows:

\[
T_i = \frac{(\tau_{b,c})_t - \tau_{b,cr}}{\tau_{b,cr}} \tag{3.45}
\]

Where, \((\tau_{b,c})_t\) is the current related effective bed-shear stress \((N/m^2)\) at time ‘\(t\)’ and \(\tau_{b,cr}\) is the critical time averaged bed-shear stress \((N/m^2)\) according to Shields.

The current related effective bed-shear stress was determined as a function of grain friction efficiency and grain related bed-shear stress. The mathematical expression can be written as:

\[
(\tau_{b,c})_t = \eta_i (\tau_{b,c})_t \tag{3.46}
\]

Where, \((\tau_{b,c})_t\) is the grain related bed-shear stress at time ‘\(t\)’ and \(\eta_i\) is the grain friction efficiency which is determined from the ratio of the current related Chezy coefficient to the grain related Chezy coefficient and determined as follows:

\[
\eta_i = \left(\frac{C_i}{C_i'}\right)^2 \tag{3.47}
\]

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The current related Chezy coefficient can be expressed as a function of compensation factor and the depth of flow. By introducing the compensation factor and applying the Strickler formula, the mathematical expression of current related Chezy coefficient can be written as follows:

\[ C_t = \frac{1}{n} \left( \frac{(CFc)h_t^{CFc}}{n} \right)^{\frac{1}{6}} \]  

(3.48)

The grain related Chezy coefficient is determined according to Equation 3.39.

3.3.2.1.8 Grain related Bed-shear Stress

The grain related bed-shear stress in Equation 3.46 is an important factor which measures the power of streamflow to extricate sediment particles along a particular stream reach (Henderson 1966). In this research project, the grain related bed-shear stress was determined according to the van Rijn’s (1993) equation. The mathematical expression proposed by van Rijn (1993) was modified and used for this study as follows:

\[ \tau_{b,t} = \rho_w g(CFc)h_t^{CFc}S_0 \]  

(3.49)

Where, \( \rho_w \) is the density of water (kg/m³).

For the calculation of the bed-shear stress parameter \( (T_t) \) by Equation 3.45, the estimation of the critical time-averaged bed-shear stress is mandatory. For this thesis, the critical time-averaged bed-shear stress was determined according to the following Equation 3.50:

\[ \tau_{b,cr} = 0.001(\rho_s - \rho_w)gD_{50}\theta_{cr} \]  

(3.50)

Where, \( \theta_{cr} \) is the particle mobility parameter at the initiation of motion.

The particle parameter \( (\theta_{cr}) \) depends on the hydraulic conditions, the particle shape and particle position relative to the other particles. The hydraulic condition near to the bed can be expressed as the Reynolds number \( (R_{ep}) \). Thus, \( \theta_{cr} = f(R_{ep}) \). The mathematical of the particle parameter expression can be written as follows:
\[
\theta_{cr} = \begin{cases} 
0.24 R_{ep}^{-2/3} & 1 < R_{ep} \leq 8 \\
0.14 R_{ep}^{-32/75} & 8 < R_{ep} \leq 32 \\
0.04 R_{ep}^{-1/15} & 32 < R_{ep} \leq 90 \\
0.013 R_{ep}^{-29/350} & 90 < R_{ep} \leq 1837 \\
0.055 & R_{ep} > 1837 
\end{cases}
\] (3.51)

The particle Reynolds number was determined by the commonly used formula:

\[
R_{ep} = \frac{\sqrt{R_S g D_{50}}}{\nu / D_{50}}
\] (3.52)

Where, \( R_S \) is the specific weight of sediment particles (dimensionless) and \( \nu \) is the kinematic viscosity coefficient (m\(^2\)/s).

The specific weight of the \( S_S \) particles was determined as follows:

\[
R_S = \frac{\rho_s}{\rho_w} - 1
\] (3.53)

The kinematic viscosity coefficient was determined as a function of temperature according to the van Rijn (1993) equation:

\[
\nu = \left[1.14 - 0.031(Tem - 15) + 0.00068(Tem - 15)^2\right] \times 10^{-6}
\] (3.54)

Where, \( Tem \) is the water temperature (ºC).

3.3.2.1.9 Particle Fall Velocity

The particle fall velocity is one of the main controlling hydraulic parameters for the transportation as well as the deposition of \( S_S \) (van Rijn 1984b). The difference in the behaviour of \( S_S \) particles can be estimated by calculating their fall velocity (van Rijn 1993; Maidment 1993). It is therefore considered as a behavioural property of the sediment particles. However, there are many factors affecting the fall velocity of the sediment particles; amongst them the effects of size, shape, and density of sediment particles, the effects of fluid density and turbulence are the most important in the water quality study (Maidment 1993).
Based on the diameter of the sediment particles, the fall velocity can be calculated in different ways. A number of relationships are found in literature for different particles diameter. All of the mathematical formulations have been developed based on the characteristics of SS particles under different flow conditions (van Rijn et al. 1990). In this study, the developed model employed three different equations to calculate the particle fall velocity of the representative sediment particles. The users have the option of providing the median particle diameter as an input parameter.

In clear water, for the particles smaller than 100 $\mu m$ in diameter, the fall velocity mainly depends on the particle size and fluid properties (Garcia 2008). For these particles, the fluid drag force on the particle is in equilibrium with the gravity force and the sediment particles follow the well-known Stokes Law (van Rijn 1984b). In the Stoke range particles Reynolds number is less than one and the particle fall velocity can be calculated according to van Rijn (1984b) and Garcia (2008). The expression used in the model is written as follows:

$$w_s = \frac{1}{18} \cdot \frac{R_s g D_s^2}{\nu} \quad \text{when} \quad D_s < 100 \mu m$$  (3.55)

Where, $D_s$ is the representative diameter of SS particles.

However, the suspended particles with the diameter greater than 100 $\mu m$, fall out of the Stokes region and the above simple expression (Equation 3.55) for particle fall velocity is no longer valid. In that case, for the particles with a diameter ranging from 100 to 1000 $\mu m$, the fall velocity equation is proposed by Soulsby (1997) which is very similar to the equation proposed by van Rijn (1984b). The mathematical expression of the equation can be written as follows:

$$w_s = 10 \cdot \frac{\nu}{D_s} \left[ 1 + \frac{0.01 R_s g D_s^3}{\nu^2} \right]^{0.5} - 1 \quad \text{when} \quad 100 > D_s < 1000 \mu m$$  (3.56)

For particles larger than about 1,000 $\mu m$ the following simple fall velocity of van Rijn’s (1982) equation was used in this study. The expression is as follows:

$$w_s = 1.1 \left[R_s g D_s\right]^{0.5} \quad \text{when} \quad D_s > 1000 \mu m$$  (3.57)
3.3.2.2 Dissolved Oxygen

Dissolved oxygen \((DO)\) is the most important parameter observed in stream water. It represents the amount of molecular oxygen dissolved in water which is essential for the plant or animal processes. The prolonged periods of oxygen depletion can result in the death of fish or other animals; while too much oxygen is a sign of increased plant/algal biomass due to the nutrients enrichment.

Since \(DO\) is the primary reason for the deterioration of the quality of stream water, its inclusion in stream water quality modelling is crucially important (Bowie et al. 1985). During storm events, algal activity is negligible for relatively shallow and small streams (Mannina and Viviani 2010). Therefore, the contribution of \(DO\) due to algal biomass can be avoided throughout rainfall events. Since this study is concentrated on the pollutant transportation during rainfall events for relatively shallow streams, it can be assumed that there is the saturation level of \(DO\) in a stream.

The most frequently used method for the estimation of saturated \(DO\) is the polynomial equation developed by Elmore and Hayes (1960) for distilled water (Bowie et al. 1985). Elmore and Hayes (1960) have summarised the works of different researchers who measured \(DO\) at the saturation level. All of these equations were developed as functions of the temperature. Using the method of Elmore and Hayes (1960), the \(DO\) model of the stream water quality model is presented as follows:

\[
DO = DO_{sat} = 14.652 - 0.41022Tem + 0.0079910(Tem)^2 - 0.00077774(Tem)^3 \quad (3.58)
\]

Where, \(DO\) is the concentration of the dissolved oxygen and \(DO_{sat}\) is the concentration of \(DO\) at the saturation level.

3.3.2.3 Biochemical Oxygen Demand

Biochemical oxygen demand \((BOD)\) is a measure for determining the amount of oxygen required by aerobic biological organisms in a water body for the decomposition of organic matter at a certain temperature over a specific time period. It is the most important parameter which controls the quality of water. Therefore, the estimation of \(BOD\) is most important in water quality modelling.
Usually, the processes of BOD decay are described by assuming first order kinematics. These processes are well-recognised and widely employed approaches in the estimation of BOD for stream water. Although these techniques are widely used as an organic quality of water, they are empirical methods. In this research project, the main equation employed for the simulation of the BOD decay rate can be expressed according to Equation 3.59 as follows:

\[
BOD_t = \frac{Q_{in,t} \left[ BOD_{t-1} \left( 1 - e^{-\left( k_{BOD} + k_{BOD,settling} - k_{s fed} \right) \Delta t} \right) \right]}{Q_{out,t}}
\]  

(3.59)

Where, \( BOD_t \) is the concentration of BOD at time \( t \), \( BOD_{t-1} \) is the concentration of BOD at time \( t-1 \), \( k_{BOD} \) is the rate of oxidation of BOD (i.e. BOD decay rate) (1/day), \( k_{s fed} \) is the rate of BOD loss due to settling (1/day), \( k_{s fed} \) is the re-suspension rate of BOD (1/day), \( \Delta t \) is the time taken for a pollutant to reach from the upstream to the downstream of a stream reach section (day), \( Q_{in,t} \) is the inflow discharge at time \( t \) (m³/s) and \( Q_{out,t} \) is the outflow discharge at time \( t \) (m³/s).

In a column of water, the BOD decay rate is influenced by a number of factors. The influences of these factors are described by using the both theoretical and empirical formulations. Amongst the factors which influence the BOD decay rate, the temperature is the most important (Bowie et al. 1985). The BOD decay rate occurs at a rate, which increases with the increasing temperature up to the point, where protein denaturation begins. Therefore, Equation 3.59 of BOD decay rate should be modified for the inclusion of the temperature effects. The following mathematical expression of Equation 3.60 was used in the current model development to simulate the temperature effects of BOD.

\[
k_{BOD(Te)} = k_{BOD(20)} \theta_{BOD}^{T_20} \frac{DO}{\delta_{BOD} + DO}
\]  

(3.60)

Where, \( k_{BOD(Te)} \) is the rate of oxidation of BOD at \( (Te) \) °C temperature, \( k_{BOD(20)} \) is the rate of oxidation of BOD at temperature 20 °C, \( \theta_{BOD} \) is the temperature correction factor for the organic decay and \( \delta_{BOD} \) is the half saturation constant for BOD decay.
3.3.2.4 Total Nitrogen

Since nutrients availability is one of the main factors in controlling algal bloom, the nutrients dynamics are essential in any water quality model (Bowie et al. 1985). For the development of proper watershed management strategies, information on the mechanisms of nutrients mobilisation into waterways and water bodies is essential (Mathers et al. 2007).

Bowie et al. (1985) noted that in a stream system, the nitrogen dynamics are modelled in complex manner, because of their substantial biogeochemical role, important oxidation-reduction reactions and other important water quality variables. In natural water, the denitrification process reduces the amount of \( TN \). On the other hand, the mineralisation of BOD by the bacteria releases \( TN \) in the water. Again, the settling of SS reduces the amount of adsorbed \( TN \) in the stream processes. In this study, three major processes were employed in the development of the \( TN \) model, i.e. denitrification, mineralisation from BOD and sedimentation. Rusjan et al. (2008) found that the concentration of nitrogen in rainfall is less; and hence the atmospheric input of nitrogen to rainwater is significant. Therefore, it is not included in the current modelling.

All the processes of \( TN \) that occur in a stream reach are described assuming the first order kinematics. The governing equation for the transformation of \( TN \) employed in the model is described as follows:

\[
\Delta TN_t = Q_{in,t} \left( TN_{t-1} (1-e^{-k_{ext} \Delta t}) + k_{TN} BOD_{t-1} e^{-k_{BOD} \Delta t} - k_{sTN} SS_{SET} \right) - Q_{out,t} \]

Where,

- \( TN_t \) is the concentration of \( TN \) (mg/l) at time ‘\( t \’\), \( TN_{t-1} \) is the concentration of \( TN \) (mg/l) at time \( t-1 \), \( k_{DEN} \) is the denitrification coefficient of \( TN \) (1/day), \( k_{TN} \) is the coefficient for the mineralisation of \( TN \) from BOD, \( k_{sTN} \) is the release rate of \( TN \) from sediment and \( SS_{SET} \) is the SS which settles on the bottom.

The temperature influences the rate parameters of all nitrogen transformation processes. Therefore, all of the rate coefficients are temperature dependent. A reference temperature of 20 °C is usually assumed when specifying each rate coefficient (Bowie et al. 1985). The governing equations for the temperature correction factor are described as follows:
Where, \(k_{DEN(Te)}\) is the denitrification coefficient of TN at \((Te)\) °C temperature \((1/day)\), \(k_{DEN(20)}\) is the denitrification coefficient of TN at 20 °C temperature \((1/day)\), \(\theta_{DEN}\) is the temperature multiplier for the denitrification coefficient, \(k_{TN(Te)}\) is the coefficient for the mineralisation of TN from BOD at \((Te)\) °C temperature \((1/day)\), \(k_{TN(20)}\) is the coefficient for the mineralisation of TN from BOD at 20 °C temperature \((1/day)\), \(\theta_{TN}\) is the temperature multiplier for the mineralisation of TN from BOD, \(k_{sTN(Te)}\) is the release rate of TN from sediment at \((Te)\) °C, \(k_{sTN(20)}\) is the release rate of TN from sediment at 20 °C and \(\theta_{sTN}\) is the temperature multiplier for sediment TN release.

### 3.3.2.5 Total Phosphorus

In stream water, TP operates like TN in many respects. However, there is no reduction of TP in stream water except the settling with SS. In this research, two major processes were considered for the simulation of TP. Mathematical processes of TP that occur in a stream reach were described assuming the first order kinematics. The governing transformation of TP along the stream is as follows:

\[
TP_t = \frac{Q_{in\ t\ TP} \left[TP_{t-1} + k_{TP} BOD_{t-1} e^{-k_{BOD} \Delta t} - (k_{SS} SS_{SET} + \delta_{TP})\right]}{Q_{out\ t}}
\]  

(3.65)

Where, \(TP_t\) is the concentration of TP at time ‘t’, \(TP_{t-1}\) is the concentration of TP at time \(t-1\), \(k_{TP}\) is the coefficient for the mineralisation of TP from BOD, \(k_{sTP}\) is the release rate of TP from sediment and \(\delta_{TP}\) is the adjustment factor for sediment TP release.

Since the rate coefficients are temperature dependent, they require correction for the stream temperature. These coefficients are input at 20 °C and are then corrected to temperature using the Streeter-Phelps (1925) type formulation (Brown and Barnwell 1987). The governing equations for the temperature correction factors of TP processes are described as follows:
\[ k_{TP(Te)} = k_{TP(20)} \theta_{TP}^{Te-20} \]  \hspace{1cm} (3.66)

\[ k_{sTP(Te)} = k_{sTP(20)} \theta_{sTP}^{Te-20} \] \hspace{1cm} (3.67)

Where, \( k_{TP(Te)} \) is the coefficient for the mineralisation of TP from BOD at \((Te) \) °C, \( k_{TP(20)} \) is the coefficient for the mineralisation of TP from BOD at 20 °C, \( \theta_{TP} \) is the temperature multiplier for the mineralisation of TP from BOD, \( k_{sTP(Te)} \) is the release rate of TP from sediment at \((Te) \) °C, \( k_{sTP(20)} \) is the release rate of TP from sediment at 20 °C and \( \theta_{sTP} \) is the non-dimensional temperature multiplier for sediment TP release.

### 3.4 Integration of the Models

The need for the integrated assessment and modelling has grown extensively as the severity of environmental problems in the 21st Century worsens and to meet the challenges of sustainability. The potential benefits of mathematically predicting and analysing water quality parameters through the integrated modelling concept are widely recognised because of the time and cost effectiveness as well as the improved performance and understanding of the actual behaviour of the whole system. Therefore, the main objective of this thesis was to improve understanding of the tools and water quality parameters for integrated management of aquatic ecosystems of waterways and receiving water bodies.

In the literature review, the rationale of the need for the integrated modelling approach and the general consideration of practices through integrated management were discussed. However, the review lacks a detailed argument of how one can put the integrated modelling approach into practice. Risbey et al. (1996) noted that there are a variety of integrated models which contain a variety of structures. Many of these structures share a number of core approaches, assumptions, or component models, which are based on earlier models or pioneer works.

There was no formal equation for the integrated catchment-stream water quantity and quality simulation. However, it was well recognised that water quality pollutants of a particular catchment transfers to a stream through the catchment outlet via surface runoff. Therefore, in this study, the catchment water quality model and the stream water quality model were developed individually.
The models of the subsystems were formulated in a consistent way and run in parallel to consider the interactions in both ways or to use information from the downstream system to take control actions in the upstream system. Finally, both of the models were integrated to be able to continuously simulate the mentioned water quality parameters. The output from the catchment water quality model was used as the input for the stream water quality model. In each case, the integration was proposed as a means to overcome the separation and fragmentation.

Manual calculation of runoff and pollutant loads is time consuming and expensive. Therefore, a computer program was developed by the author for the formulation of the catchment and stream water quality models and their integration. The modelling codes were developed by using the Visual Basic 6.0. The source code is not included with this thesis to reduce the volume.

3.5 Conclusions
Due to the multiplicity of water quality constituents, the quality of natural water varies in time and among locations. The high cost of water quality samples collection from every location and their laboratory analysis is the greatest hindrance in monitoring water quality at the field level. Therefore, water quality modelling plays an important role for the assessment of physical, chemical and biological changes in waterways and receiving water bodies. However, the need for the integrated modelling approach has become prevalent over recent years as decision makers feel dissatisfied with the results from ‘narrowly-focused’ isolated models. The integrated modelling approach recognises the complexity of the natural systems and their interactions. This approach further provides a better control of pollutant build-up and wash-off from catchment surfaces.

This chapter has described the development of an integrated catchment-stream water quality model which continuously simulates different water quality parameters. The integrated model is capable for simulating both single event and continuous runoff and pollutant loads. To develop an integrated water quality model, it was necessary to obtain an improved understanding of the relevant models. Therefore, the integrated model comprised of two individual models, i.e. the catchment water quality model and the stream water quality model.
Considering the TA method, a rainfall-runoff model was developed. Then processes of water quality parameters were incorporated with the rainfall-runoff model which represents the catchment water quality model. Water quality parameters of catchments transfer to a stream through the catchment outlet with surface runoff and are discharged into receiving water bodies. Therefore, a stream water quality model was developed with the same water quality parameters of the catchment water quality model. However, for the stream water quality model, the process of BOD was also included. Finally, the catchment water quality model and the stream water quality model were integrated for the continuous simulation of the mentioned water quality parameters.

The model was developed to create a scientific and technical base from which standardised integrated catchment-stream water quality modelling can be established. The model provides a mathematical framework for the phenomena occurring during the $t_d$ and storm event to catchment surfaces. The equations represent the dominant processes affecting water quality parameters but include empirical terms and coefficients. The model algorithms are based on the known performance characteristics of common stormwater quality improvement measures.

The major advantage of the model is the continuous simulation of water quality parameters by an integrated modelling approach. The integrated model eliminates the inconsistent results which arise from isolated models. The model will help to avoid the split responsibilities for management and planning of catchments and streams, and can be used for mixed land-use areas. The model can be used to extend water quality surveillance and to predict the future water quality/quantity conditions.

The developed model has the potential to give the reliable prediction results of water quality parameters under the practical and complex conditions. The preparation process of the input data for the model is simple. The capability of the model to simulate surface runoff from a wide range of intensities make the model useful to assess the impact of water quality parameters to waterways and receiving water bodies and to the design effective stormwater treatment measures.
Chapter 4
Data Collection and Study Catchments

4.1 Background

It is well recognised that the prediction of water quality parameters for the purpose of watershed management strategies rely on the mathematical modelling output. Water quality models are important for understanding and explaining the pollutants processes and predicting the impact of the future changes (such as environmental policy) on water resources (Glavan et al. 2011). However, the application of any water quality model depends on calibration and validation with sufficient and reliable field data (Bouteligier et al. 2002). The need for data on background or base-line conditions is the essential requirements for watershed management strategies. All of the empirical equations of any model should be calibrated and validated against adequate data. Water quality predictions through any surface runoff model are useless without local data. Due to the lack of local data for calibration of any water quality model leads to the significantly biased results for the precise estimation of pollutant loads.

The quality of water in any drainage area depends on the land-use of that particular area. According to Chen and Adams (2006), the water quality model parameters vary not only spatially (i.e. catchment to catchment), but also temporally (i.e. differ among different rainfall events). Although SWMM is used in many countries, Leinster and Walden (1999) discouraged the generic application of water quality models not only from overseas countries but also from other parts of the same country. Tsihrintzis and Hamid (1998) noted that due to the absence of measured data, water quality models calibrated for other similar sites can be used only for screening purposes. Because depending upon the catchment characteristics (permeability of soils, initial pollutant loads), the impact of the actual land-use and management changes (Volk et al. 2009). Therefore, Puckett (1995) noted that watershed management plan needs to be developed based on individual watershed information.
This chapter of the thesis demonstrates the description of necessary data and the catchment characteristics which were used as case study for this research. The description has a particular focus on the characteristics of the physical environment that influence the quality of water. The primary focus was to obtain both water quantity and quality data which were observed at the catchment outlet. Water quality data collection was performed in two stages, i.e. the experimental data collection and the observed field data collection. The experimental data were collected from published literature. The measured field data were collected from GCCC, Australia. The experimental catchments were only impervious and the observed catchments were mixed land-uses.

4.2 Data Requirements for the Model
Since the developed model is comprised of many empirical equations, they need to be calibrated and validated against sufficient and reliable observed data. The integrated model was developed by comprising two individual models. Therefore, depending upon the choice of the users, the data requirements of the integrated model are different. Specifically, the model requires data for both the catchment water quality model and for the stream water quality model. Overall, the model requires hydrologic, hydraulic and water quality data.

For the calibration of the hydrologic component of the catchment water quality model, precipitation data is perquisite. During a storm event, rainfall undergoes the $Evp$ loss (mentioned in the literature review). The $Evp$ loss also occurs from surface water of the uncovered land surfaces. Hence, the model requires $Evp$ data to account this loss. Moreover, the model needs data for the calculation of $IL$ and $CL$ parameters. Finally, the model needs the catchment characteristics data. If there is more than one catchment contributing to surface runoff, the model needs lag time for each of the catchments. For calibration of the quality component of the catchment water quality model, the integrated model requires observed water quality data which are collected at the catchment outlet. All the mandatory information is given as the input parameters during the runtime of the model. The stream water quality model can be used either as a separate model or integrated with the catchment water quality model. For both cases, the stream water quality model needs hydraulic data, initial streamflow data and water quality data.
4.3 Data Collection

Good quality hydrologic, pollutants and physical data are required for the calibration and estimation of the water quality model parameters (Marsden et al. 1973). According to Boorman (2003), specifying water quality parameters to be modelled without fully understanding data availability is clearly a weakness in a study that has no data collection program. Due to the complex nature of pollutants processes, significant array of data is required for calibration and validation of any water quality model. Moreover, sufficient and reliable data are essential for improving existing models and creating a new one (Bertrand-Krajewski et al. 1993).

However, available and reliable data collection is the major challenge in water quality study (Nicholas and Walling 1998). It was understood that water quality data are not readily available. Because in most of the cases, data are inadequate and fragmented; and this put potentially severe limitations to the sustainable management of the nation’s water (Smith 1998). Zoppou (2001) and Rauch et al. (2002) argued that the lack of data is the greater hindrance for the development of water quality model than the lack of suitable algorithms. Due to the scarcity of measured data, the calibration and parameters estimation are impossible even for the simplest water quality model (Gaume et al. 1998).

In Australia, limited water quality data are available for the calibration and parameters estimation of sophisticated water quality model (Leinster and Walden 1999). There are also lots of deficiencies for the currently available data. Deletic and Maksimovic (1998) identified the common deficiencies of the currently available water quality data. The major paucity of available water quality data are:

- Water quality data on complete storms are very rare.
- The capacity of the samplers is limited to collect continuous data throughout storm events.
- Data is recorded at inadequate intervals and the frequency of data collection is generally low.
- Data is usually collected at the outlet of watershed area.
- Data is not always reliable.
4.3.1 Precipitation Data

For all hydrologic and water quality systems, precipitation data is the key information because it activates watershed responses in hydrologic systems (Chaubey et al. 1999; Newham 2002). In this study, one of the major purposes of precipitation data collection was to calibrate the developed catchment water quality model for the estimation of surface runoff and to determine the $t_d$. Therefore, it was necessary to collect continuous rainfall hyetographs for a long period of time. Both rainfall simulator data and natural precipitation data were collected in this research.

Experimental precipitation data of this study was collected from available literature. Egodawatta (2007) performed experimental study to observe the transferred water quality parameters from impervious catchment surfaces by using simulated rainfall. These rainfall simulator data was collected and used for this research project. For the estimation of the model parameters from the observed water quality data, actual hourly precipitation data for 2004 - 2008 was used from a nearby rainfall station of the study catchments. Precipitation data was measured by the weather station ‘Gold Coast Seaway’ (station no. 40764). Historical rainfall data for the study catchments were supplied by the Bureau of Meteorology (BoM). Duration of the simulation period and the number of storm events used in the simulation is shown in Table 4.1. Only those storm events were selected for the simulation for which water quality measurements were conducted. As the research project was intended to estimate the amount of pollutants transferred during storm events, rainfall data was chosen when the water quality measurement was undertaken.

<table>
<thead>
<tr>
<th>Storm No.</th>
<th>Duration date</th>
<th>No. of Events</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10/01/2004 – 19/01/2004</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>02/02/2007 – 15/02/2007</td>
<td>6</td>
</tr>
<tr>
<td>3</td>
<td>04/07/2008 – 09/07/2008</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>04/12/2004 – 06/12/2004</td>
<td>2</td>
</tr>
<tr>
<td>5</td>
<td>21/04/2005 – 04/05/2005</td>
<td>5</td>
</tr>
</tbody>
</table>
4.3.2 Experimental Water Quality Data

For the estimation of the model parameters from the experimental data, small impervious catchments from Gold Coast, Australia were selected. The experimental data obtained by Egodawatta (2007) was used to determine the build-up and wash-off models parameters from impervious surfaces. The experimental study was done only for impervious surfaces using simulated rainfall. These data on defined and limited surfaces were selected to obtain more specific and detailed knowledge on pollutant processes suggested by other researchers, e.g. Herngren et al. (2005); Vaze and Chiew (2002). The data was collected from road and roof surfaces.

4.3.3 Observed Water Quality Data

Water quality data collection programs have been initiated in Queensland by a number of local authorities (Leinster and Walden 1999):

- Brisbane City Council
- Cairns City Council
- Ipswich City Council and
- Gold Coast City Council

The observed water quality data used for this research was collected from Gold Coast City Council (GCCC), Australia. To investigate specific pressures on the health and environmental quality to nearby waterways and receiving water bodies, runoff water quality monitoring programs was started by GCCC. Water quality monitoring data observed by GCCC were studied to obtain insight to the nature of water quality parameters accumulation and washed-off, and to help improving the long-term continuous simulation of the catchment water quality model. The same data was also used for calibration and validation of the stream water quality model.

GCCC has been monitoring water quality parameters at different locations within catchments. Details of the data monitoring program are contained in the CMU (2002) report. The observed field data was collected for pervious dominated and mixed surfaces catchments. For this thesis, water quality data from two different catchments were used. These catchments were selected after careful consideration where the detailed water quality monitoring programs were undertaken.
It should be noted that for calibration of the catchment water quality model, upper most sub-catchments were selected to get unaltered pollutants transported only from catchments, i.e. to avoid water quality samples with pollutant degradation within creek. The output from the catchment water quality model was used as the input of the stream water quality model for the integrated simulation of water quality parameters.

4.3.4 Collection of Catchment Characteristics Data

The physical data of the catchment characteristics was collected for the simulation of the integrated model. These data include catchment areas, percent impervious and pervious, slopes and roughness parameters. These data were compiled from the several sources, such as contour maps, aerial photographs and other available literatures.

Generally, every catchment encompasses of two different surface areas, i.e. impervious surface and pervious surface. Each of the surface area designates different responses to surface runoff hydrographs and pollutographs. Therefore, any catchment water quality model requires knowing impervious and pervious surface areas separately. In this study, the separation was done from percent impervious data. The degree of imperviousness from the selected catchments was determined from aerial photograph. Pervious surface area was determined by the model from percent impervious data and represents differing responses from impervious and pervious surfaces contribution. It was mentioned earlier that catchment sub-division is important to represent the partial area contribution for surface runoff quantity and quality. Data for catchment sub-division was obtained from topographic image maps. If topographic maps were not available, catchment sub-division data was obtained by using the empirical method of HEC-1.

4.4 Description of the Study Catchments

Since the quality of water represents the health of watershed areas, it is important to monitor watershed conditions to obtain reliable data for the mitigation of water quality problems (Dai et al. 2004). Tsihrintzis and Hamid (1998) suggested that for the prediction of water quality parameters with the higher degree of confidence, the site specific input parameters are mandatory. Therefore, the selection of study area was one of the most important factors in this study.
The Gold Coast area was chosen to collect both experimental and field observation water quality data. This area is one of the few places in Australia where a comprehensive catchment monitoring program has been established. Gold Coast is called the capital of Queensland, Australia. The city is located in the south-east Queensland, about 78 km south of the Brisbane. The Gold Coast City is bounded by Logan city, Redland city and Moreton Bay in the north, Coral Sea in the east, the New South Wales border in the south and the Scenic Rim Region in the west. A location map of the Gold Coast area is shown in Figure 4.1.

Gold Coast is the sixth largest city of Australia. The total area of the city is about 1,406 km², featuring 40 km of coast line. Based on the city’s estimated resident population, GCCC is Australia’s second largest local government. The population of the city is approximately 515,157 and is expected to reach about 600,000 in 2021. Gold Coast is one of the most famous tourist spots in Australia with approximately 12% of the population being visitors. There are seven tourist parks and camping ground in the city.
Gold Coast has the largest government park estate in Australia and encompasses more than 2,254 parks covering 22,071 ha of land. There are more than 3,500 ha open spaces for sports and recreation. Beaches and waterways in particular are most important tourist attractions (GCCC-Web 2010).

Gold Coast city has a sub-tropical climate with average 287 days of sunshine annually. The city is with moderate temperature and summer dominated precipitation. The average temperature in summer ranges from 19 °C to 29 °C (66 to 83 degrees Fahrenheit) and winter temperature ranges from 9 °C to 21 °C (48 to 69 degrees Fahrenheit). The city’s stormwater drainage system features many natural and artificial waterways. These waterways include the five major rivers and numerous creeks, many of which are connected to artificial lakes and canals. Total length of rivers and streams run through the city is about 480 km. Navigable waterways of the city are around 260 km which is 9 times more than the Venice. Wetland reserves, lakes, dams and canals are more than 1,000 ha. The major watershed basins of the city are shown in Figure 4.2.

Figure 4.2: Major watershed basins of Gold Coast city (GCCC)
The waterways of the Gold Coast city flow from the surrounding west ward hilly area towards the Pacific Ocean coastline. Natural vegetation occupies a significant fraction of the regional land particularly west of the city. High density urban areas are located close to the coastline. Most of the residential settlements have developed adjacent to the city’s integrated waterways which promote luxurious waterside living. There are about 3,139 km of roads, more than 600 km of bikeways, and 923 km of paths and walkways in the Gold Coast city. The city has a protected natural area more than 25,000 ha. However, extensive urban development alongside the different waterways of the city influences the key environmental values of waterways ecology (GCCC-Web 2010).

4.4.1 Description of the Experimental Catchments

To investigate pollutants build-up and wash-off, Egodawatta (2007) conducted experimental study on the two different types of impervious surfaces, i.e. road and roof surfaces for SS. For the accumulation of pollutant on catchment surfaces, the only variable considered was the $t_d$, while for the wash-off investigation, rainfall intensity and duration was considered. The area of all experimental catchments (road and roof surfaces) remained the same, i.e. 3 $m^2$.

4.4.1.1 Road Surfaces Catchments

Along the many catchments monitored by GCCC, Egodawatta (2007) performed his experimental study within Highland Park area, Gold Coast, Australia, to investigate pollutant build-up and wash-off on road surfaces. Highland Park area is predominantly a residential area, and the area is situated within the boundary of the Nerang River catchment. The experimental study was performed to the three small road surfaces at: (a) Lauder Court, (b) Gumbeel Court and (c) Piccadilly Place. Road sections were straight, about 50 $m$ in length, mild slope with various traffic volumes. Most of the roads were access road and mainly used by the local traffics. The selected road sections were wide enough so that the local traffic was not affected during the experimental period. The roads were in good conditions (Egodawatta 2007). However, there were ongoing construction activities to the upstream portion of the catchments. The characteristics summaries of road surface catchments are shown in Table 4.2.
Table 4.2: Characteristics summary of the road surfaces catchments (Egodawatta 2007)

<table>
<thead>
<tr>
<th>Site</th>
<th>Description</th>
<th>Slope (%)</th>
<th>Surface texture depth (mm)</th>
<th>Number of households</th>
<th>Area (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lauder Court</td>
<td>Single detached housing area</td>
<td>10.00</td>
<td>0.66</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>Gumbeel Court</td>
<td>Duplex housing area</td>
<td>7.20</td>
<td>0.92</td>
<td>25</td>
<td>3</td>
</tr>
<tr>
<td>Piccadilly Place</td>
<td>Single detached housing area</td>
<td>10.80</td>
<td>0.83</td>
<td>41</td>
<td>3</td>
</tr>
</tbody>
</table>

4.4.1.2 Roof Surfaces Catchments

For the experimental study of pollutant build-up and wash-off processes on roof surfaces, Egodawatta (2007) selected the two roof materials at Southport Deport, Gold Coast, Australia. The two roofing materials used in the experimental study were: (a) corrugated steel and (b) concrete tile. The roofing materials selection was done so that they were representative for the study region. The two study models for the roofs were designed at an angle of 20°. To reduce the human influence and traffic induced turbulence, roofs were kept at a typical single story roofing height. The maximum height of the roofs was 2.5 m. Surrounding land-uses of the experimental sites were commercial and residential (Egodawatta 2007).

4.4.2 Description of the Observed Catchments

The catchment studied in this research work for the collection of observed water quality data are also located at Gold Coast, Australia. The two catchments were selected as case study: (a) Hotham creek catchment (HTCC) and (b) Saltwater creek catchment (SWCC). These two catchments were selected to ensure that different catchment characteristics and hydraulic systems are represented. As most of the other studies were based on impervious area, the selected catchments were pervious dominated and mixed. GCCC is monitoring water quality data for these two catchments. Data from these catchments were used for calibration of water quality parameters of the developed integrated model. A detailed map of the creeks running through the study catchments is shown Figure 4.3.
4.4.2.1 Hotham Creek Catchment

Hotham creek is the main tributary of the Pimpama River. It is a small creek with approximately 15 km in length. The headwater of the creek runs through residential areas near Willowvale, onto floodplain near the Pimpama River. A map of HTCC indicating area sub-division is shown in Figure 4.4.
Hotham is basically a freshwater creek. In this study, considered portion of the creek drains water from around 302.35 ha. The surface area of the catchment was pervious dominated. Land-use of the catchment included sugarcane, pasture, quarry and degraded wetland (Gavine et al. 2000). The upper portion of the catchment was dominated by farming practices, including dairy and beef cattle, banana and various other crops. For these farming activities large area of the upper portion was cleared.
In addition, to harvest water for irrigation and livestock, large riparian areas was cleared for access to the creek. Due to these clearing of surfaces, the authority was bound to establish various environmental weeds including Camphor Laurel. Soil disturbance within the lower catchment had the potential to change the physio-chemistry of the aquatic environment rapidly (GCCC-Web 2010). The summary of the characteristics of HTCC is shown in Table 4.3. As watershed delineation is important in hydrological modelling, the area was divided into eight sub-catchments. The detailed information of each sub-catchment is shown in Table 4.4. This information for the sub-area division was obtained from the aerial photograph.

### Table 4.4: Geographical parameters for each sub-catchment in HTCC

<table>
<thead>
<tr>
<th>Sub-catchments</th>
<th>Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Area (ha)</td>
<td>% Impervious</td>
</tr>
<tr>
<td>A₁</td>
<td>22.29</td>
<td>0</td>
</tr>
<tr>
<td>A₂</td>
<td>53.63</td>
<td>0</td>
</tr>
<tr>
<td>A₃</td>
<td>44.63</td>
<td>0</td>
</tr>
<tr>
<td>A₄</td>
<td>20.38</td>
<td>0</td>
</tr>
<tr>
<td>A₅</td>
<td>51.89</td>
<td>0</td>
</tr>
<tr>
<td>A₆</td>
<td>22.48</td>
<td>5</td>
</tr>
<tr>
<td>A₇</td>
<td>65.05</td>
<td>7</td>
</tr>
<tr>
<td>A₈</td>
<td>21.98</td>
<td>10</td>
</tr>
</tbody>
</table>

### 4.4.2.2 Saltwater Creek Catchment

Saltwater creek is a small micro-tidal estuary in Gold Coast region of Australia. The creek is the main tributary of Coomera River. It is approximately 17 km in length and the lower 10 km of the creek is being marine environment (Benfer et al. 2007; Webster and Lemckert 2002). Headwater of the creek runs through forested areas. The creek is located between Coombabah creek and Coomera River. It joins with Coombabah creek before entering into Coomera River. A map of the catchment and sub-area division is shown in Figure 4.5.
In this study, considered area of SWCC was approximately 1,025 ha with 33.5% impervious. The summary characteristics of the catchment are shown in Table 4.5. The area of the catchment is of varied landscape including virgin bush and housing subdivisions. The catchment was significant in social, economic and environmental terms. The detailed information of each sub-catchment is shown in Table 4.6. Similar to HTCC these information were derived from aerial photograph.
Table 4.5: Summary of the characteristics of SWCC

<table>
<thead>
<tr>
<th>Catchment</th>
<th>Total Area (ha)</th>
<th>Imperviousness (%)</th>
<th>No. of sub-catchments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Saltwater Creek Catchment</td>
<td>1024.51</td>
<td>33.5</td>
<td>16</td>
</tr>
</tbody>
</table>

Table 4.6: Geographical parameters for each sub-catchment in SWCC

<table>
<thead>
<tr>
<th>Sub-Catchments</th>
<th>Parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Area (ha)</td>
<td>% Impervious</td>
</tr>
<tr>
<td>A₁</td>
<td>81.15</td>
<td>0</td>
</tr>
<tr>
<td>A₂</td>
<td>66.77</td>
<td>0</td>
</tr>
<tr>
<td>A₃</td>
<td>76.92</td>
<td>80</td>
</tr>
<tr>
<td>A₄</td>
<td>9.75</td>
<td>0</td>
</tr>
<tr>
<td>A₅</td>
<td>96.01</td>
<td>0</td>
</tr>
<tr>
<td>A₆</td>
<td>59.26</td>
<td>0</td>
</tr>
<tr>
<td>A₇</td>
<td>67.11</td>
<td>0</td>
</tr>
<tr>
<td>A₈</td>
<td>30.62</td>
<td>35</td>
</tr>
<tr>
<td>A₉</td>
<td>81.06</td>
<td>80</td>
</tr>
<tr>
<td>A₁₀</td>
<td>75.65</td>
<td>60</td>
</tr>
<tr>
<td>A₁₁</td>
<td>28.49</td>
<td>60</td>
</tr>
<tr>
<td>A₁₂</td>
<td>47.82</td>
<td>25</td>
</tr>
<tr>
<td>A₁₃</td>
<td>67.35</td>
<td>65</td>
</tr>
<tr>
<td>A₁₄</td>
<td>57.79</td>
<td>55</td>
</tr>
<tr>
<td>A₁₅</td>
<td>26.96</td>
<td>30</td>
</tr>
<tr>
<td>A₁₆</td>
<td>64.06</td>
<td>75</td>
</tr>
</tbody>
</table>

During the study period, the catchment came under the pressure due to the new development. Residential development was planned within the middle to upper freshwater region of the catchment with a strong demand for new waterside housing accommodation. This significant development was placing pressure to the surrounding water environment due to the various anthropogenic activities (Webster and Lemckert 2002). Hence, SS and nutrients loadings increased for the catchment.
4.5 Conclusions

In water quality modelling, data is the key information enabling the accurate model responses. The application of any water quality model always depends on calibration and validation from adequate and reliable data. Significant array of hydrologic (i.e. precipitation) and water quality data are required to calibrate and validate any water quality model. These data may be experimental or measured or both. In addition, information about the characteristics of the physical environment of a watershed area which influence the quality of water is essential.

Since equations of the developed integrated model represents water quality processes and all contain certain degree of empiricism, data are essential for calibration and validation. Calibration of the model required accurate data of the catchment characteristics, rainfall-runoff, observed water quality, streamflow and stream water quality data.

Both the experimental and observed field data were collected from the Gold Coast region of Australia. The experimental data was collected from the study done by Egodawatta (2007) for road and roof surfaces using simulated rainfall. The observed precipitation data was collected from a rainfall station which is very near to the study catchments. These actual precipitation data for the selected catchments were supplied by the BoM, Australia. The observed water quality data for the selected catchments were collected from GCCC, Australia.

The data was collected from two catchments which were pervious dominated and mixed land-use. The types of issues and physical environments of the study catchments were generally similar to many other catchments of Australia. Therefore, it was suitable to adopt water quality data from these catchments for calibration and validation of the model for the widespread application.
Chapter 5
Calibration and Parameters Estimation

5.1 Introduction

In order to use any mathematical model for the prediction of water quality parameters, it is essential to estimate the model parameters which are relevant to modelling processes. The accuracy of the modelling results largely depends on the accuracy of the model parameters. Therefore, the parameters estimation is the key step in the practical application of any water quality model (Karadurmus and Berber 2004).

For all models, the precise prediction of the pollutant concentration and loads rely on an accurate estimation of the model parameters (Baffaut and Delleur 1990). However, the estimation of the model parameters is the most critical step in any water quality model (Tsihrintzis and Hamid 1998). It is often difficult to determine the model parameters accurately because the values of many model parameters are linked and interrelated to each other. For example, in the catchment water quality models, the parameters of the pollutant wash-off model are strongly depended on the pollutant build-up. Due to the absence of available pollutants on a catchment surface, the wash-off might become zero even if there is high intensity of rainfall with significant volume of surface runoff. As a result, there is a possibility in identifying a number of alternative model parameters.

Although it is difficult to estimate the model parameters, they are required for the analysis, improvement and/or update of existing BMPs (Tsihrintzis and Hamid 1998). Different researchers use different procedures for the estimation of the model parameters. For example, Deletic and Maksimovic (1998) and Kim et al. (2006) proposed indirect methods for the calculation of the model parameters. An alternative approach is the estimation of the parameters by the calibration procedure using runoff quality data collected at the watershed outlet (Alley and Smith 1981) which reflects the combined effects of an entire catchment. Tsihrintzis and Hamid (1998) noted that calibrated water quality models are essential for specific regions for the prediction of the impact of different water quality parameters into receiving water bodies.
Calibration is an iterative process in which the parameters of a particular model are constantly adjusted until the deviation or standard error between the simulated and observed values are minimised to a satisfactory level (Chen and Adams 2007). The calibration procedure includes the use of the estimated parameters in the field as well as adjustment of some parameters to match better model predictions with measured data. The calibration procedure attempts not only to identify the best set of parameters, but also helps to assess and reduce the uncertainty in parameter values (Beck 1991).

Like any other water quality model, the developed model needs to be correctly calibrated and verified for a particular catchment before its use for the reliable prediction of water quality parameters. This chapter of the thesis demonstrates the estimation of parameters for the developed integrated catchment-stream water quality model. The main focus was to obtain a set of model parameters for which the predicted pollutant concentration are close to the measured concentration. The data used to calibrate the model was collected by runoff sampling. Calibration of the catchment water quality model and the stream water quality was performed separately for the three major water quality parameters, i.e. SS, TN and TP. These parameters were selected for the estimation of the model parameters based on the simulation capability of the developed model.

5.2 Model Parameters Required Estimation

The feasibility of any model depends on its capability of producing the general results that can be used to aid the implementation of effective management options. Since the developed model is an integration of a catchment water quality model and a stream water quality model, it was needed to estimate the parameters for both of the models. However, each of the models consists of quantity and quality sub-models; and hence the quantity and quality parameters have to be determined.

For the catchment hydrological model, IL and CL are needed to be determined first. For the water quality component of the catchment model, it is required to estimate coefficients and exponents of the pollutant build-up and wash-off models. However, the phenomena of non-point pollutant build-up and wash-off are influenced by a large number of factors which make the estimation of the model parameters more difficult (Temprano et al. 2006).
Depending upon the physical characteristics of catchment, the input parameters of model can vary significantly. Obviously, it is difficult to measure the accurate build-up and wash-off rates of a particular catchment due to various reasons. Nevertheless, the application of the developed model requires the proper estimation of the build-up and wash-off model parameters. Table 5.1 summarises the required model parameters that needed to be estimated for the catchment water quality model before applying it to a particular catchment.

Table 5.1: Required parameters for the catchment water quality model simulation

<table>
<thead>
<tr>
<th>Processes</th>
<th>Parameters</th>
<th>Symbol</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runoff</td>
<td>Initial loss</td>
<td>IL</td>
<td>mm</td>
</tr>
<tr>
<td></td>
<td>Continuing loss</td>
<td>$A_{CL}$, $B_{CL}$ and $C_{CL}$</td>
<td>mm/hr</td>
</tr>
<tr>
<td>Pollutant build-up</td>
<td>Antecedent dry days</td>
<td>$t_d$</td>
<td>day</td>
</tr>
<tr>
<td></td>
<td>Maximum build-up rate</td>
<td>$C_1(p,s)$</td>
<td>kg/km$^2$</td>
</tr>
<tr>
<td></td>
<td>Build-up coefficient</td>
<td>$C_2(p,s)$</td>
<td>kg/km$^2$ · day$^{-C_1}$</td>
</tr>
<tr>
<td></td>
<td>Power of the build-up rate</td>
<td>$C_3(p,s)$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Build-up exponent</td>
<td>$k_p(p,s)$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Half saturation constant</td>
<td>$t_p(p,s)$</td>
<td>day</td>
</tr>
<tr>
<td>Pollutant Wash-off</td>
<td>Wash-off coefficient</td>
<td>$E_1(p,s)$</td>
<td>[mm/hr]$^{E_2}$</td>
</tr>
<tr>
<td></td>
<td>Wash-off exponent</td>
<td>$E_2(p,s)$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Wash-off rate coefficient</td>
<td>$E_3(p,s)$</td>
<td>kg[m/s]$^{E_4}$</td>
</tr>
<tr>
<td></td>
<td>Power of wash-off rate</td>
<td>$E_4(p,s)$</td>
<td>1/mm</td>
</tr>
<tr>
<td></td>
<td>Wash-off exponent</td>
<td>$E_5(p,s)$</td>
<td>-</td>
</tr>
</tbody>
</table>

Like the catchment water quality model, it was required to estimate the parameters for both water quantity and quality sub-models of the stream water quality model. For the quantity model, hydrologic and hydraulic parameters are to be estimated. For the quality model, the main processes of SS, TN, TP and BOD were considered and hence the parameters of relevant processes are to be determined. Table 5.2 lists the number of parameters required to be estimated for the stream water quality model.
Table 5.2: Required parameters for the stream water quality model simulation

<table>
<thead>
<tr>
<th>Processes</th>
<th>Parameters</th>
<th>Symbol</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Runoff</td>
<td>Reach Length</td>
<td>$L$</td>
<td>km</td>
</tr>
<tr>
<td></td>
<td>Sub-reach Length</td>
<td>$\Delta L$</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Weighing factor</td>
<td>$X_P$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Reach storage parameter</td>
<td>$K_{stor}$</td>
<td>sec</td>
</tr>
<tr>
<td></td>
<td>Stage discharge parameters</td>
<td>$a, b$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Compensation factors</td>
<td>$CF_c, CF_e$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Channel bed Slope</td>
<td>$S_0$</td>
<td>m/km</td>
</tr>
<tr>
<td></td>
<td>Manning’s roughness</td>
<td>$n$</td>
<td>-</td>
</tr>
<tr>
<td>SS</td>
<td>Water temperature</td>
<td>$T_{em}$</td>
<td>°C</td>
</tr>
<tr>
<td></td>
<td>Water density</td>
<td>$\rho_w$</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>Particle size of incoming SS</td>
<td>$D_{50, 90}$</td>
<td>mm</td>
</tr>
<tr>
<td></td>
<td>Density of SS</td>
<td>$\rho_s$</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td>BOD</td>
<td>$BOD$ decay rate</td>
<td>$k_{BOD}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>$BOD$ settling rate</td>
<td>$k_{sBOD}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>$BOD$ suspension rate</td>
<td>$k_{ssBOD}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Temperature correction factor for organic decay</td>
<td>$\theta_{BOD}$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Half saturation constant</td>
<td>$\delta_{BOD}$</td>
<td>1/day</td>
</tr>
<tr>
<td>TN</td>
<td>Denitrification coefficient</td>
<td>$k_{DEN}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Release rate of $TN$ from $BOD$</td>
<td>$k_{TN}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Release rate of $TN$ from SS</td>
<td>$k_{sTN}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Temperature multiplier for the mineralisation of $TN$ from $BOD$</td>
<td>$\theta_{TN}$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Temperature multiplier for sediment $TN$ release</td>
<td>$\theta_{sTN}$</td>
<td>-</td>
</tr>
<tr>
<td>TP</td>
<td>Release rate of $TP$ from $BOD$</td>
<td>$k_{TP}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Release rate of $TN$ from SS</td>
<td>$k_{sTP}$</td>
<td>1/day</td>
</tr>
<tr>
<td></td>
<td>Adjustment factor of sediment $TP$ release</td>
<td>$\delta_{TP}$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Temperature multiplier for the mineralisation of $TP$ from $BOD$</td>
<td>$\theta_{TP}$</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Temperature multiplier for sediment $TP$ release</td>
<td>$\theta_{sTP}$</td>
<td>-</td>
</tr>
</tbody>
</table>
5.3 Estimation of Catchment Water Quality Model Parameters

Selection of an appropriate set of parameter values of any catchment quality model is essential to simulate the catchment responses accurately. Since the developed catchment water quality model consists of a number of functional sub-models, such as rainfall-runoff transformation, pollutant build-up and wash-off, it was required to calibrate all these sub-models. Based on available data sets, the catchment water quality model was calibrated for the selected catchments. The major parameters of runoff, build-up and wash-off models were adjusted until the deviation or standard error between the simulated value and the observed value of pollutant loads were reduced to a satisfactory level.

The estimation of the model parameters was performed from the both experimental and observed field data. The experimental data was collected from impervious catchments and the observed data were collected from catchments with mixed (impervious and pervious) land-uses. The parameters estimation from the experimental data was performed by using trend lines of fitting data sets. From the available observed field data, the parameters estimation was done by using the calibration procedure. Where the observed data was not available, the parameters were estimated from information of available literature and by comparisons with widely used water quality models. The estimation of the hydrologic and water quality model parameters was performed separately.

5.3.1 Hydrologic Model Parameters Estimations

As the output of the runoff model is used as the input of the pollutant model, the estimation of hydrologic model parameters was performed before the estimation of the pollutant model parameters. In order to accomplish this hydrologic calibration and parameters estimation, precipitation records of the ‘Gold Coast Seaway’ station were analysed. The total five rainfall events ranging from low to high intensity were selected for the estimation of hydrologic parameters of the developed model. The duration of the simulation periods and the number of rainfall events simulated is shown in Table 5.3. Rainfall and evaporation data for the selected dates and catchments were used as the input parameters for the model.
Calibration of the hydrological model was performed for IL, CL parameters and catchments roughness. The parameters were progressively adjusted within reasonable limits to improve the model performance. These parameters were estimated for two selected catchments, i.e. HTCC and SWCC.

Table 5.3: Description of the storms used in this study

<table>
<thead>
<tr>
<th>Simulation No.</th>
<th>Duration date</th>
<th>No. of Events</th>
<th>Simulated for Catchment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10/01/2004 – 19/01/2004</td>
<td>4</td>
<td>HTCC</td>
</tr>
<tr>
<td>2</td>
<td>02/02/2007 – 15/02/2007</td>
<td>6</td>
<td>HTCC</td>
</tr>
<tr>
<td>3</td>
<td>04/07/2008 – 09/07/2008</td>
<td>2</td>
<td>HTCC</td>
</tr>
<tr>
<td>4</td>
<td>04/12/2004 – 06/12/2004</td>
<td>2</td>
<td>SWCC</td>
</tr>
<tr>
<td>5</td>
<td>21/04/2005 – 04/05/2005</td>
<td>4</td>
<td>SWCC</td>
</tr>
</tbody>
</table>

No runoff measurements for these catchments were conducted during the sampling period. For this reason, the developed catchment water quality model was calibrated with the results of WBNM and DRAINS which are widely used models in Australia. For these models, the values of IL and CL were adopted by reviewing available literature for similar conditions.

5.3.1.1 Initial Loss and Continuing Loss Model Parameters

Usually, it is considered that IL happen at the very beginning of a rainfall event and it is considered as constant. However, CL occurs after the commencement of surface runoff and mainly comprises of infiltration. In the absence of available data, these losses were determined from information of published literature.

For Australia, the values of rainfall loss can be derived from the Australian Rainfall Runoff (ARR). In Appendix A, the values of IL and CL are shown for some regions of Australia. For other regions the values of these losses can be taken from the IEAust (2001) or can be derived from the experimental studies. In this study, the values of IL and CL were determined from the suggested typical ranged values of ARR. The simulation was first performed by using the suggested parameters of IEAust (2001).
The simulation results were further improved by varying each parameter so that the values lie within the ranged values suggested by IEAust (2001). The final adopted values of rainfall loss parameters are summarised in Table 5.4. Based on the final adopted parameters, the model simulated hydrographs, the peak discharge and the duration of runoff for the selected catchments. The summary of the computed peak discharge and their comparisons with other models (WBNM, DRAINS and Rational Method) are shown in Tables 5.5 and 5.6 for HTCC and SWCC respectively.

Table 5.4: Final values of the hydrological parameters used in the runoff model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Surface</th>
<th>HTCC</th>
<th>SWCC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Loss (mm)</td>
<td>IMPV</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>PEV</td>
<td>15</td>
<td>15</td>
</tr>
<tr>
<td>Continuing loss (mm/hr)</td>
<td>IMPV</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>PEV</td>
<td>2.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Evaporation (mm/day)</td>
<td>-</td>
<td>24.1 - 27.4</td>
<td>24.6 - 25.7</td>
</tr>
</tbody>
</table>

Table 5.5: Comparison of the peak discharges for HTCC

<table>
<thead>
<tr>
<th>Events</th>
<th>Date</th>
<th>Peak discharge (m³/s) for HTCC</th>
<th>Rainfall Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Simulated</td>
<td>WBNM</td>
</tr>
<tr>
<td>1</td>
<td>15/01/2004</td>
<td>32.96</td>
<td>35.29</td>
</tr>
<tr>
<td>2</td>
<td>30/08/2005</td>
<td>0.06</td>
<td>0.057</td>
</tr>
<tr>
<td>3</td>
<td>13/02/2007</td>
<td>104.37</td>
<td>110.77</td>
</tr>
<tr>
<td>4</td>
<td>08/07/2008</td>
<td>8.77</td>
<td>9.531</td>
</tr>
</tbody>
</table>

Table 5.6: Comparison of the peak discharges for SWCC

<table>
<thead>
<tr>
<th>Simulation No.</th>
<th>Date</th>
<th>Peak Discharge (m³/s)</th>
<th>Rainfall Intensity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Simulated</td>
<td>WBNM</td>
</tr>
<tr>
<td>1</td>
<td>06/12/2004</td>
<td>64.70</td>
<td>64.706</td>
</tr>
<tr>
<td>2</td>
<td>03/05/2005</td>
<td>74.39</td>
<td>74.382</td>
</tr>
<tr>
<td>3</td>
<td>19/03/2008</td>
<td>0.95</td>
<td>0.952</td>
</tr>
</tbody>
</table>
The plotted results of the comparisons are shown in Figures 5.1 to 5.4 for HTCC and Figures 5.5 to 5.7 for SWCC. From Table 5.5 and Figure 5.1, it is clear that the rational method gave the higher peak discharge for the low intensity rainfall for HTCC. This was also true for medium intensity rainfall shown in Table 5.5 and Figure 5.2.

![Figure 5.1: Peak discharge comparison for low rainfall intensity (HTCC)](image1)

![Figure 5.2: Peak discharge comparison for medium-low rainfall intensity (HTCC)](image2)
On the other hand, for the higher intensity rainfall, the rational method under-estimated the peak flow as shown in Figures 5.3 and 5.4. This was due to the effects of partial area contribution considered in the other methods. For low intensity rainfall, the rational method considered the whole catchment area in estimating the peak discharge which leads to the estimation of the higher values. For high intensity rainfall, it is doing the same thing as with average rainfall intensity. The other models consider the partial area effect with exact rainfall intensity producing the exact peak discharge.
From Table 5.6 and Figure 5.5 it can be concluded that for low intensity rainfall, the simulated peak discharge of the developed model was close to the outcomes of other models for SWCC. For high intensity rainfall, the simulated peak discharge was also very much closer to the WBNM simulated peak discharge as shown in Figure 5.7. However, comparing it with the DRAINS model results, the simulated peak discharge was little bit higher.

Figure 5.5: Peak discharges comparison for the low intensity rainfall (SWCC)

Figure 5.6: Peak discharges comparison for medium-high intensity rainfall (SWCC)
The difference of the simulated results with the WBNM and DRAINS were due to the sub-division of catchment area. Both the developed model and the WBNM model used the sub-area divided from the aerial photography. However, the DRAINS model subdivided the catchment area automatically. Hence, the peak discharge from the DRAINS model differs from the actual value. Nevertheless, observed discharge measurements were required to decide suitability of the models for the catchment. Also the simulated time to the peak of the developed model is similar to the other models outcomes for all rainfall events. The simulated peak discharges of the developed model were very close to the peak discharges simulated by other models indicating the suitability of the model for runoff simulations.

To compare the performance of the developed hydrological model, two error statistics $PBIAS$ (percent bias) and $NSE$ (Nash Sutcliffe efficiency) were computed. Where,

$$PBIAS = \frac{\sum_{t=1}^{N}(Q^{\text{obs}}_t - Q^{\text{sim}}_t)}{\sum_{t=1}^{N}Q^{\text{obs}}_t}$$  \hspace{1cm} (5.1)$$

$$NSE = 1 - \frac{\sum_{t=1}^{N}(Q^{\text{sim}}_t - Q^{\text{obs}}_t)^2}{\sum_{t=1}^{N}(Q^{\text{obs}}_t - Q^{\text{mean}}_t)^2}$$  \hspace{1cm} (5.2)$$
As there was no observed runoff data for the catchment at that time, statistical comparisons were made with the DRAINS model. The first statistic \( PBIAS \) measure is the average tendency of simulated flows to be larger or smaller than their observed counterparts. The optimal value is zero; positive values indicate a model bias towards the underestimation, whereas negative values indicate a bias towards the overestimation (Gupta et al. 1999). For our data sets, the \( PBIAS \) value was 0.00 for low intensity rainfall and -0.09 to -0.16 for high intensity rainfall.

The second statistic, \( NSE \), provides normalise indicators of model performance in relation to benchmarks. It measures the relative magnitude of the residual variance to the variance of flows. The optimum value is 1.0, and the value should be larger than 0.0 to be a ‘minimally acceptable’ performance (Moriasi et al. 2007). A value equal to zero indicates that the mean observed flow is a better predictor than the model. For our data sets \( NSE \) value was 1.00 for low intensity rainfall and 0.60 to 0.88 for the higher intensity of rainfall. Again these variations in statistics were due to the different methods used in the sub-division of the catchment. However, the observed data is needed to compare the reliability of statistics of the developed model.

5.3.2 Water Quality Model Parameters from Experimental Data

The water quality component of the catchment water quality model requires the specification of the model parameters for the both pollutant build-up and wash-off models. The developed model was used to represent the available experimental data collected from street and roof surfaces from residential catchments at Gold Coast, Australia. As no experimental data were found for \( TN \) and \( TP \), the estimation of the model parameters was performed only for \( SS \). It was mentioned in Chapter 4 that Egodawatta (2007) assessed the pollutant build-up and wash-off at Gold Coast, for low and high population density residential areas from impervious surfaces. The experimental study was performed on road and roof surfaces using simulated rainfall. This experimental data collected by Egodawatta (2007) was used to estimate the parameters of the pollutant build-up and wash-off equations of the developed catchment water quality model.
5.3.2.1 Pollutant Build-up Model Parameters Estimation

The maximum pollutant loads collected from the experimental sites varied in the range of 3000 to 6000 kg/km$^2$. The maximum pollutant load is the representative value of the coefficient $C_1(p,s)$ for the pollutant build-up model shown in Equations 3.12 to 3.14. Hence, the values of the $C_1(p,s)$ for SS is ranged from 3000 to 6000 kg/km$^2$. This ranged value of $C_1(p,s)$, i.e. 3000 to 6000 kg/km$^2$ was used in this thesis to estimate the other build-up model parameters from the experimental data.

The data in Figures 5.8 and 5.9 show the relationships between the build-up amounts and the $t_d$ for the three different road surfaces and the two different roof surfaces respectively. To derive the general relationship, the best fit curves were drawn for each of the surfaces. The derived best fit curves had the coefficient of determination ($R^2$) more than 0.90 indicating good fits for available data sets. From the plotted best fit curves, the values of build-up model coefficients $C_2(p,s)$ and $C_3(p,s)$ were calculated for the road and roof surfaces.

![Figure 5.8: Relationship between pollutant build-up and dry days (Road surfaces)](image)

Figures 5.10 and 5.12 show the relationships and parameters estimations of the exponential function build-up model for road and roof surfaces respectively. Exponents of Figures 5.10 and 5.12 provide the pollutant build-up rate constants $k_p(p,s)$. 

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Figures 5.9 and 5.10 show the parameters estimation of the saturation function build-up model for the road and roof surfaces respectively. The slopes of the lines in Figures 5.11 and 5.13 passing through the origin represent values of the half saturation constant $t_{p}(p,s)$. The estimated values of the road and roof surfaces build-up parameters are shown in Table 5.7.
From Figures 5.8 to 5.13, it is clear that the build-up model parameters of Gumbeel Court road surface and Corrugated Steel roof surface catchments are higher than the other catchments. Therefore, it can be concluded that water quality parameters vary from catchment to catchment, and hence the generic outcomes from one model should be used cautiously.
Table 5.7: The estimated values of SS build-up parameters

<table>
<thead>
<tr>
<th>Catchment</th>
<th>Catchment Characteristics</th>
<th>$C_1(p,s)$ (kg/km²)</th>
<th>$C_2(p,s)$ $(kg/km^2 \cdot day^{-C_1})$</th>
<th>$C_3(p,s)$</th>
<th>$k_p(p,s)$ (1/day)</th>
<th>$t_p(p,s)$ (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gumbeel Court</td>
<td>Townhouse region with high population density</td>
<td>5300</td>
<td>2623.8</td>
<td>0.238</td>
<td>0.222</td>
<td>1.244</td>
</tr>
<tr>
<td>Lauder Court</td>
<td>Single detached housing</td>
<td>2750</td>
<td>1678.5</td>
<td>0.155</td>
<td>0.210</td>
<td>0.784</td>
</tr>
<tr>
<td>Piccadilly Place</td>
<td>Region with low population density</td>
<td>2600</td>
<td>1900.7</td>
<td>0.102</td>
<td>0.382</td>
<td>0.418</td>
</tr>
<tr>
<td>Concrete Tile</td>
<td>Industrial and commercial areas</td>
<td>850</td>
<td>424.22</td>
<td>0.208</td>
<td>0.188</td>
<td>1.208</td>
</tr>
<tr>
<td>Corrugated Steel</td>
<td>Industrial and commercial areas</td>
<td>1200</td>
<td>401.33</td>
<td>0.349</td>
<td>0.122</td>
<td>0.434</td>
</tr>
</tbody>
</table>
5.3.2.2 Wash-off Model Parameters

Similar to the pollutant build-up model parameters, the estimation of the wash-off model parameters was performed from the experimental study done by Egodawatta (2007). Based on available data, the ratios between pollutant wash-off $W_t(p,s)$ and remaining build-up $(B_t)$ were calculated for different rainfall intensities and durations.

For the estimation of power function wash-off model parameters, graphs between $W_t(p,s)/B_t$ vs. $q_{A,t}$ are plotted for both road and roof surfaces. Figure 5.14 shows the graph for the road surface of Piccadilly place and Figure 5.15 shows the graph for the concrete tile roof surface. Coefficients and exponents of the best fit curves provide the values of the wash-off coefficient $E_1(p,s)$ and exponent $E_2(p,s)$ respectively. Similar relationships were observed for the other catchments and roof surfaces.

Figure 5.16 shows the graph of $W_t(p,s)$ vs. $Q_t$, for the estimation of the rating curve wash-off model parameters for the Gumbeel Court road surface catchment. Similarly, the estimation of the rating curve wash-off model parameters for roof surfaces was performed. Figure 5.17 shows the graph of $W_t(p,s)$ vs. $Q_t$ for the corrugated steel roof surface. Like the power function wash-off model parameters estimation, coefficients and exponents of the best fit equation provided the wash-off model parameters $E_3(p,s)$ and $E_4(p,s)$ for the rating curve function.

Figure 5.14: Power function wash-off model parameters estimation (Road surfaces)
For the estimation of the exponential wash-off model parameters, graphs between \( B_{ij} - W_I(p,s) \) vs. \( I_t \) were plotted for Lauder Court road surface and concrete tile roof surface shown in Figures 5.18 and 5.19 respectively. Road and roof surfaces in other catchments also followed the similar pattern. Exponents of Figures 5.18 and 5.19 are the wash-off parameter \( E_5(p,s) \) of the exponential wash-off model.
The summary of the estimated values of the wash-off model parameters for both the road surface and roof surface catchments are shown in Table 5.8. Alike the build-up model parameters, the wash-off model parameters from experimental data were performed only for SS. Similar experimental studies should be performed to determine the model parameters for other pollutants.
Figure 5.19: Exponential wash-off model parameters estimation
(Roof surface catchment)

Table 5.8: Estimated values of wash-off model parameters for SS

<table>
<thead>
<tr>
<th>Catchments</th>
<th>$E_1(p,s)$ [mm/hr]$^{E_1}$</th>
<th>$E_2(p,s)$</th>
<th>$E_3(p,s)$ [kg/m³/s]$^{E_4}$</th>
<th>$E_4(p,s)$</th>
<th>$E_5(p,s)$ [1/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gumbeel Court</td>
<td>0.0029 - 0.0135</td>
<td>0.608 - 0.986</td>
<td>3414.4 - 5260.4</td>
<td>0.472 -    0.580</td>
<td>0.011</td>
</tr>
<tr>
<td>Lauders Court</td>
<td>0.0015 - 0.0059</td>
<td>0.945 - 1.270</td>
<td>1649.0 - 2927.8</td>
<td>0.344 -    0.744</td>
<td>0.028</td>
</tr>
<tr>
<td>Piccadilly Place</td>
<td>0.0062 - 0.011</td>
<td>0.753 - 0.914</td>
<td>1864.6 - 2992.1</td>
<td>0.413 -    0.564</td>
<td>0.026</td>
</tr>
<tr>
<td>Concrete Tile</td>
<td>0.051 - 0.202</td>
<td>0.363 - 0.603</td>
<td>585 - 805</td>
<td>0.556 -    0.797</td>
<td>0.388</td>
</tr>
<tr>
<td>Corrugated Steel</td>
<td>0.112 - 0.213</td>
<td>0.333 - 0.414</td>
<td>2362 - 2685</td>
<td>0.993 -    1.000</td>
<td>0.134</td>
</tr>
</tbody>
</table>
5.3.3 Water Quality Model Parameters from Observed Field Data

With the available observed water quality data, the parameters for the pollutant component of the catchment water quality model were determined by the calibration procedure. Calibration of the build-up model and the wash-off model were performed separately to estimate the required build-up and wash-off model parameters. Transferred pollutants from catchment surfaces can be from either an impervious surface or a pervious surface or both. Hence, the parameters for the catchment water quality model were estimated for impervious and pervious surfaces separately. For each calibration run, the measured and predicted water quality was matched to achieve required pollutant transportation. This was done by adjusting the input parameters of the developed model.

However, different researchers observed that the values of the build-up and wash-off models parameters changed not only from catchment to catchment but also for rainfall events. Berretta et al. (2007) noted that a single characteristic is not comprehensive in describing the complexity of the model parameters affecting the both pollutant build-up and wash-off processes. Therefore, the estimation of the model parameters should be performed for a range of rainfall events. In this study, calibration was performed for several continuous rainfall-runoff events, so that the accurate and reliable parameters estimation could be achieved. Beven and Binley (1992); Volk et al. (2009) noted that there is no possibility to expect that any one set of parameters will represent a true parameter set. That’s why a range of parameters were estimated for the developed model.

Calibration of the both sub-models (build-up model and wash-off model) was done simultaneously until the total predicted pollutant loadings matched with the measured field observation. However, water quality at the beginning of a rainfall event will be dominated by the estimated build-up model parameters. It should be noted that for the current calibration, the upper most sub-catchments were selected to get unaltered pollutants transported only from the catchments, i.e. to avoid water quality samples without pollutants degradation within the stream, and to avoid the tidal effects of the stream.
5.3.3.1 Build-up Model Parameters

The build-up model was developed by considering the $t_d$ as the predominant parameter. The accuracy of pollutants prediction depends on the proper calculation of the $t_d$. The initial value of the $t_d$ was determined after analysing rainfall data. For the continuous simulation, intermediate dry days were calculated by the model itself. Initially, the build-up model parameters were selected based on information available in literature. These parameters were progressively refined along with the pollutant wash-off model parameters until an acceptable calibration was achieved for a range of rainfall events. The build-up model parameters were estimated before the estimation of the wash-off model parameters because the wash-off loads are directly proportional to pollutant build-up loads (Baffaut and Delleur 1990). For the estimation of the parameters, calibration of the model was performed for several rainfall events in each simulation.

Table 5.9: Estimated values of build-up model parameters of SS for SWCC

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IMPV</td>
<td>PEV</td>
</tr>
<tr>
<td>$C_1(p,s)$</td>
<td>6000</td>
<td>12000</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>2500</td>
<td>4500</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.40</td>
<td>0.40</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>$t_p(p,s)$</td>
<td>1.50</td>
<td>2.00</td>
</tr>
</tbody>
</table>

Table 5.10: Estimated values of the build-up model parameters for TN

<table>
<thead>
<tr>
<th>Parameters</th>
<th>HTCC Values</th>
<th>SWCC Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>IMPV</td>
<td>PEV</td>
<td>IMPV</td>
</tr>
<tr>
<td>$C_1(p,s)$</td>
<td>300</td>
<td>400</td>
<td>500</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>150</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.45</td>
<td>0.45</td>
<td>0.45</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.23</td>
<td>0.23</td>
<td>0.50</td>
</tr>
<tr>
<td>$t_p(p,s)$</td>
<td>2.00</td>
<td>3.00</td>
<td>1.50</td>
</tr>
</tbody>
</table>
The summary of the estimated build-up model parameters of SS for SWCC is shown in Table 5.9. The summarised values of the estimated build-up model parameters for HTCC and SWCC are shown in Tables 5.10 and 5.11 for TN and TP respectively. The detailed estimations of the build-up model parameters are shown in Appendix B.

Table 5.11: Estimated values of the build-up model parameters for TP

<table>
<thead>
<tr>
<th>Parameters</th>
<th>HTCC</th>
<th>SWCC</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>IMPV</td>
<td>PEV</td>
<td>IMPV</td>
<td>PEV</td>
</tr>
<tr>
<td>$C_1(p,s)$</td>
<td>150</td>
<td>200</td>
<td>250</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>50</td>
<td>70</td>
<td>90</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.15</td>
<td>0.15</td>
<td>0.15</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.15</td>
<td>0.15</td>
<td>0.10</td>
</tr>
<tr>
<td>$t_p(p,s)$</td>
<td>4.00</td>
<td>6.00</td>
<td>4.00</td>
</tr>
</tbody>
</table>

5.3.3.2 Wash-off Model Parameters

Similar to the pollutant build-up model parameters, the wash-off model parameters were calculated for the two catchments based on the available build-up and the instantaneous hydrological characteristics of the catchments surfaces.

Table 5.12: Estimated values of the SS wash-off model parameters for SWCC

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>IMPV</td>
<td>PEV</td>
<td></td>
</tr>
<tr>
<td>$E_1(p,s)$</td>
<td>0.030</td>
<td>0.050 – 0.055</td>
</tr>
<tr>
<td>$E_2(p,s)$</td>
<td>0.21</td>
<td>-</td>
</tr>
<tr>
<td>$E_3(p,s)$</td>
<td>0.160 – 0.180</td>
<td>0.300 – 0.330</td>
</tr>
<tr>
<td>$E_4(p,s)$</td>
<td>0.30</td>
<td>-</td>
</tr>
<tr>
<td>$E_5(p,s)$</td>
<td>0.00180</td>
<td>0.00340</td>
</tr>
</tbody>
</table>
Table 5.13: Estimated values of the wash-off model parameters for \( TN \)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>HTCC</th>
<th>SWCC</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Values</td>
<td>IMPV</td>
<td>PEV</td>
<td>IMPV</td>
</tr>
<tr>
<td>( E_1(p,s) )</td>
<td>0.0013 – 0.0017</td>
<td>0.0024 – 0.0032</td>
<td>0.0040 – 0.0050</td>
</tr>
<tr>
<td>( E_2(p,s) )</td>
<td>0.75 – 0.80</td>
<td>0.75 – 0.80</td>
<td>0.65 – 0.80</td>
</tr>
<tr>
<td>( E_3(p,s) )</td>
<td>0.000040</td>
<td>0.000065 – 0.00007</td>
<td>0.0012</td>
</tr>
<tr>
<td>( E_4(p,s) )</td>
<td>0.90</td>
<td>0.90</td>
<td>0.84 – 0.90</td>
</tr>
<tr>
<td>( E_5(p,s) )</td>
<td>0.00018 – 0.00035</td>
<td>0.00040 – 0.00065</td>
<td>0.0017 – 0.0020</td>
</tr>
</tbody>
</table>

Table 5.14: Estimated values of the wash-off model parameters for \( TP \)

<table>
<thead>
<tr>
<th>Parameters</th>
<th>HTCC</th>
<th>SWCC</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Values</td>
<td>IMPV</td>
<td>PEV</td>
<td>IMPV</td>
</tr>
<tr>
<td>( E_1(p,s) )</td>
<td>0.00012 – 0.00005</td>
<td>0.0007 – 0.00021</td>
<td>0.00030 – 0.00040</td>
</tr>
<tr>
<td>( E_2(p,s) )</td>
<td>0.75 – 0.90</td>
<td>0.75 – 0.90</td>
<td>0.75 – 0.90</td>
</tr>
<tr>
<td>( E_3(p,s) )</td>
<td>0.000020 – 0.000040</td>
<td>0.000035 – 0.000070</td>
<td>0.00010 – 0.00016</td>
</tr>
<tr>
<td>( E_4(p,s) )</td>
<td>0.78 – 0.88</td>
<td>0.78 – 0.88</td>
<td>0.70 – 0.80</td>
</tr>
<tr>
<td>( E_5(p,s) )</td>
<td>0.000001 – 0.000010</td>
<td>0.000001 – 0.000020</td>
<td>0.00013 – 0.00030</td>
</tr>
</tbody>
</table>
For the estimation of the wash-off model parameters, calibration of the model was performed for several rainfall events in each simulation. The summary of the estimated wash-off model parameters of \( SS \) is shown in Table 5.12 for SWCC; while Tables 5.13 and 5.14 summarises the estimated wash-off model parameters of \( TN \) and \( TP \) respectively for both HTCC and SWCC. The detailed calculations of the wash-off model parameters are shown in Appendix C.

### 5.3.4 Calibration Results

The simulation of \( SS \) wash-off with the calibrated parameters for SWCC is shown in Figure 5.20 for both the power function and the rating curve wash-off models. For the both simulations, the power function build-up model was selected. Although shapes of pollutographs were the same for the both wash-off models, the initial transportation was less with the rating curve wash-off model. As obvious pollutant build-up was the same in the both wash-off models simulation. The results indicate that the capability of the rating curve wash-off model to simulate the FF phenomenon of \( SS \) was less for SWCC. Also Deletic and Maksimovic (1998) found that FF occurs only in a limited number of storm events.

![Figure 5.20: Simulation of SS with the power function build-up (SWCC)](image-url)
It was also observed in Figure 5.20 that during the middle period of the storm event, there was less SS transportation with the power function wash-off model compared to the rating curve wash-off model. This was due to the available SS on the catchment surfaces. As FF washed-off most of the accumulated SS for the power function wash-off model, available pollutants were depleted during the middle period of the storm. For both wash-off models, the parameters were adjusted to fit the simulated results with the observed water quality data.

With the same build-up model (power function build-up model), the simulation and calibration results of TN wash-off for SWCC is shown in Figure 5.21 for both the power function and the rating curve wash-off models. Similar to the SS simulation, the capability of the rating curve wash-off model to simulate the FF phenomenon of TN was less in compared to the power function wash-off model. However, there was also the higher TN wash-off during the remaining period of the storm for the power function wash-off model. This difference may be due to the selection of the wash-off model parameters. More observed data is needed to establish the accurate wash-off model parameters set.

![Figure 5.21: Simulation of TN with the power function build-up](image)

Figure 5.21: Simulation of TN with the power function build-up
The simulation and calibration results of the TP wash-off with the power function build-up model for SWCC is shown in Figure 5.22. Similar to the SS simulation, the shapes of pollutographs were the same for both wash-off models for TP. However, the initial TP wash-off results contradict with the both SS and TN simulations. The figure clearly shows that the capability of the power function wash-off model to simulate FF for TP was less compared to the rating curve wash-off model. There was also the higher TP wash-off during the last period of the storm event for the power function wash-off model. For this model, the amount of FF was not significant, leading to an increase available TP to be washed-off during the reaming period of the storm. On the other hand, for the rating curve wash-off model, available TP depleted with the significant FF amount at the beginning of the storm leading to a lesser wash-off during the last period.

Figure 5.22: Simulation of TP with the power function build-up

The simulation results of the TN wash-off with the calibrated parameters for HTCC is shown in Figure 5.23 for the power function and the rating curve wash-off models. For both cases, the power function build-up model was used. Like SWCC, although the shapes of the pollutographs were the same in both wash-off models, the initial wash-off were less for the rating curve wash-off model. However, the pollutant build-up was the same for both wash-off models. Therefore, the capability of the rating curve wash-off
model to simulate the FF phenomenon of $TN$ was also less for HTCC. It can also be observed from the figure that there was no wash-off at the end of the storm for the rating curve wash-off model. This was due to more pollutant being washed off during the middle period of the storm for the rating curve wash-off model leaving no available $TN$ to be washed off before the end of the storm.

![Figure 5.23: Simulation of $TN$ for the power function build-up model (HTCC)](image1)

![Figure 5.24: Simulation of $TN$ for the exponential build-up model (HTCC)](image2)
Figure 5.24 shows the simulation results of TP for the two wash-off models (power function and rating curve) with the exponential build-up model for HTCC. From the figure, it is clear that there was less TP wash-off during the initial period of the storm event for the rating curve wash-off model. The simulated results of TP wash-off with the power function build-up model for HTCC is shown in Figure 5.25 for the power function and rating curve wash-off models. Like the TN wash-off, the shapes of the pollutographs were the same for the both wash-off models for TP. However, the initial TP wash-off was the same for the both wash-off models which contradict with the TN results.

Figure 5.25: Simulation of TP for the power function build-up model (HTCC)

Figure 5.26 shows the simulation results of TP for the power function and rating curve wash-off models with the exponential build-up model for HTCC. Similar to SWCC, the power function wash-off was more suitable for simulating the FF phenomenon for TP. Also there was the higher TP wash-off for the power function wash-off model at the end of the wash-off simulation period. As there was no FF for this case, there were more available pollutants to be washed-off with a higher rate of surface water. Due to less available TP for the rating curve wash-off model at the end, the wash-off was less. This indicates the dependency of TP wash-off to the available TP on the catchment surfaces.
The overall agreement of the simulated results with the observed data for the selected water quality parameters appear to be good. The general simulation results of the model with the calibrated parameters also looked reasonable, although these simulation scenarios depend on the values of coefficients/parameters used in the model equations. However, the results are promising and may be used for other catchments with similar conditions.

![Simulation of TP for the exponential build-up model (HTCC)](image)

**Figure 5.26: Simulation of TP for the exponential build-up model (HTCC)**

Although the observed data lay very close to the simulated results, there were small variations between the observed data and the simulated results. The reason for this small variation may be the build-up/wash-off parameters being not representative for the study catchments. In addition, these differences among sub-models are due to the different level of complexity of systems being modelled and are likely due to data uncertainty. Indeed, it is reasonable to suppose that the quality the data measured behave with a higher uncertainty level that, of course, influences modelling processes. Due to the limitation of data, the comparisons with only one observation were shown. Therefore, care should be taken when using these findings. More observed data is needed to check the better capability of the model.
5.4 Stream Water Quality Model Parameters

Similar to the catchment water quality model, the developed stream water quality model needs to be correctly calibrated and verified for a particular stream reach before its use in the reliable prediction of water quality parameters. Based on the available observed field data, the parameters of the stream water quality model were determined for Saltwater Creek which is located at Gold Coast, Australia. The stream water quality model is comprised of two sub-models, i.e. streamflow and stream pollutants processes. Therefore, it was necessary to estimate the parameters for both streamflow and pollutants processes.

In this research project, the estimation of the flow parameters and the pollutant parameters were performed separately. Usually, the output of the quantity model is used as the input of the quality model. Hence, the quantity model parameters were calibrated prior to its use in the quality model. Calibration of the model was done in two steps. In the first step, the quantity parameters were calibrated and evaluated by comparing the model output with widely used models. In the second step, the calibrated values of the quantity model parameters were set to determine the quality model parameters. During the calibration procedure, the parameters were varied individually and by means of combinations until a good agreement between the modelling output and measured data were achieved.

5.4.1 Streamflow Parameters Estimation

Estimation of the streamflow parameters required calibration of the Muskingum-Cunge parameters. The stability of the Muskingum-Cunge method is accomplished, if the routing time interval $\Delta t_R$ falls between the limits $2K_{stor}X_P$ and $2K_{stor} (1-X_P)$ (CRC for Catchment Hydrology 2005). The theoretical value of $K_{stor}$ is the time required for an elemental (kinematic) wave to transverse through a particular stream reach. It is taken as approximate the time interval between the inflow and the outflow peaks, if data are available. If not, the wave velocity can be estimated for various channel shapes as a function of average velocity $\bar{u}$ for any representative flow rate. The average velocity for steady uniform flow can be estimated by either the Manning’s or Chezy equation. Table 5.15 shows velocities for various channel shapes.
Table 5.15: Kinematic wave velocities (Veissman and Lewis 2003)

<table>
<thead>
<tr>
<th>Channel Shape</th>
<th>Manning equation</th>
<th>Chézy equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wide rectangular</td>
<td>$(5/3)\sqrt{u}$</td>
<td>$(3/2)\sqrt{u}$</td>
</tr>
<tr>
<td>Triangular</td>
<td>$(4/3)\sqrt{u}$</td>
<td>$(5/4)\sqrt{u}$</td>
</tr>
<tr>
<td>Wide Parabola</td>
<td>$(11/9)\sqrt{u}$</td>
<td>$(7/6)\sqrt{u}$</td>
</tr>
</tbody>
</table>

For some stream reaches, it is possible that the selection of the reasonable values of $K_{stor}$, $X_P$ and $\Delta t_R$ will cause one or two routing coefficients to be negative. Such negative coefficients could create negative outflows which is physically unreasonable and causes the routing procedure to be mathematically incompatible. Therefore, the challenge in the application of the Muskingum-Cunge method is the determination of the reasonable values of the parameters $K_{stor}$ and $X_P$ (Seybert 2006). In order to use the method with confidence, $K_{stor}$ and $X_P$ should be determined through the analysis of stream gauge information where both upstream and downstream hydrographs are known. However, in practice, these types of data are rarely available. Therefore, the practical application of the Muskingum-Cunge method requires adoption of some guiding rules of thumb that have some logical meaning. MUSIC deems that the values of $K_{stor}$ and $X_P$ remain constant within a stream reach throughout the simulation.

5.4.1.1 Stream Reach Storage Parameter

In Equations 3.27 to 3.29, $K_{stor}$ is referred as the channel reach storage parameter of storage delay time for a particular stream reach. The application of these equations has shown that $K_{stor}$ is usually close to the wave travel time. It has the dimension of time and can be shown as the time between the centres of mass of the inflow hydrograph and the outflow hydrograph. Furthermore, it is approximately equal to the travel time of a flood wave through channel reach and can be approximated by dividing the channel reach length by the celerity of the flood wave (CRC for Catchment Hydrology 2005). In some cases, this time is almost constant regardless of the size of flood so that the storage is a linear function of weighted average discharge, and the storage is said to be linear. However, in general, travel time varies with the size of the flood wave and in such cases the storage function is non-linear.
It should be noted that in the developed streamflow model, there are two different options for the calculation of $K_{stor}$, i.e. fixed and variable with the depth of flow. The fixed value of $K_{stor}$ is given by the users as an input parameter; while the variable $K_{stor}$ is determined by the model automatically from the reach length and the depth of flow. The final calibrated value of $K_{stor}$ is 0.30 for Saltwater Creek when it was assumed to be fixed.

5.4.1.2 Prism Parameter

In the Muskingum-Cunge method, the prism parameter $X_P$ is the weighting factor that proportions the relative effects of the upstream section and the downstream section on the ability of a channel to store water. This parameter reflects the ability to affects channel storage capacity. Since the Muskingum-Cunge method assumes that the upstream section and the downstream section are similar in the geometry and hydraulic characteristics, the reasonable value of $X_P$ ranged from 0.00 to 0.50 (Seybert 2006; Linsley et al. 1992). For the natural channel, it is generally ranged between 0.10 and 0.30 (Linsley et al. 1992); while the average value of $X_P$ is 0.20 (Seybert 2006; Veissman and Lewis 2003).

However, it is often difficult to avoid the negative values of the outflow hydrograph, especially when the value of $X_P$ is outside the range of 0.10 to 0.49 (CRC for Catchment Hydrology 2005). Therefore, the developed model limits the value of $X_P$ between 0.10 and 0.49. Since data for Saltwater Creek was available, the parameters of the stream water quality model were estimated for this creek. The final estimated value of $X_P$ for Saltwater Creek was 0.30.

5.4.1.3 Other Parameters

For Saltwater Creek, stage-discharge parameters (‘$a$’ and ‘$b$’) were estimated by using the HEC RAS model which is a widely used hydraulic model throughout the world. The compensation coefficients ($CF_c$ and $CF_e$) were determined from the relationship between the hydraulic radius and the depth of flow as shown in Figure 5.27. The final adopted values of the stage-discharge parameters and compensation factors are shown in Table 5.16.
Table 5.16: Values of the streamflow model parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage-discharge parameter</td>
<td>$a$</td>
<td>0.2812</td>
<td>-</td>
</tr>
<tr>
<td>Stage-discharge parameter</td>
<td>$b$</td>
<td>0.4009</td>
<td>-</td>
</tr>
<tr>
<td>Coefficient for compensation factor</td>
<td>$CFe$</td>
<td>0.542</td>
<td>-</td>
</tr>
<tr>
<td>Exponent for compensation factor</td>
<td>$CFe$</td>
<td>0.980</td>
<td>-</td>
</tr>
</tbody>
</table>

5.4.1.4 Hydraulic Modelling Results

With the estimated parameters of the hydraulic model, the simulation results were compared with the MUSIC model. As no runoff measurement was conducted for Saltwater creek during the calibration period, the simulation results of the developed model were compared with MUSIC, which is widely used model in Australia. In addition, both the developed model and MUSIC used the Muskingum-Cunge method for stream routing; the modelling results were compared with MUSIC. Figure 5.28 shows the plotted results of the comparison. It can be seen from the figure that there was agreement between the simulated results and the MUSIC results, which indicates the suitability of the hydraulic model in streamflow prediction.
For the statistical comparison with MUSIC, two error statistics $PBIAS$ (percent bias) and $NSE$ (Nash Sutcliffe efficiency) were also computed. Since there was no observed data, these comparisons were made with the MUSIC results. The computed value of $PBIAS$ was -0.073; whereas the optimal value is zero. On the other hand, $NSE$ value for the model was 0.963; whereas the optimal value is 1.0. Hence, the model can be used for the runoff estimation of a particular stream reach.

**Figure 5.28: Comparison of streamflow**

### 5.4.2 Water Quality Parameters Estimation

The water quality component of the stream water quality model consists of SS, nutrients and $BOD$ processes. Therefore, it was necessary to estimate the parameters of the relevant processes before its application to the practical problems. For this component of the stream water quality model, it was required to estimate the parameters of SS processes, the rate coefficients of both nutrients (TN and TP) and $BOD$. Based on the estimated discharges of the hydraulic model and available observed field data of water borne pollutants, the parameters of SS, $BOD$ and nutrients models were determined. The model was fitted with the observed field data collected from Saltwater Creek, Gold Coast, Australia.
5.4.2.1 Suspended Sediment

For the simulation of SS, it was required to know field data of the sediment parameters, such as $D_{50}$, $D_{90}$ and $\rho_s$. However, these data of SS were not available for Saltwater Creek. Therefore, these parameters were determined from calibration of SS data. The final calibrated values of the SS parameters for Saltwater Creek are shown in Table 5.17.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Median particle diameter</td>
<td>$D_{50}$</td>
<td>0.10</td>
<td>m</td>
</tr>
<tr>
<td>Particle size for which 90% sediment is finer</td>
<td>$D_{90}$</td>
<td>0.15</td>
<td>m</td>
</tr>
<tr>
<td>Sediment density</td>
<td>$\rho_s$</td>
<td>2650</td>
<td>kg/m$^3$</td>
</tr>
</tbody>
</table>

The typical applications of the developed model to the SS processes include predicting the transportation and the deposition of sediment for a particular stream reach. The model was applied for the both transportation and deposition processes of SS along the reach section of Saltwater Creek. The creek was characterised by a variable hydraulic regime. The length of the studied reach was approximately 1,825 m. For this length the two catchments were contributing surface runoff and pollutant loads. The area of the catchments were 1,025 ha and 245 ha having imperviousness 33.5% and 53% respectively. The calibration results of the simulation are shown in Figure 5.29.

Figure 5.29 presents the observed and simulated transportation rate of SS for Saltwater Creek during the two consecutive rainfall events. From the figure, it is clear that during the beginning and the end of the runoff simulation, there was increased SS transportation. This was due to the higher concentration of incoming sediment from the contributed catchments of the creek. When there was an increased amount of incoming SS in the creek, there was more transportation. However, during this time the streamflow was low. Hence, there should have more SS deposition instead of transportation. As the streamflow was low, the volume of water decreased. Therefore, the concentrations of SS per unit volume of water increased with more incoming SS.
On the other hand, there should be more SS deposition with the increased incoming sediment concentration. However, when the transport capacity of the streamflow increases the deposition rate decreases. For the creek, the transport capacity of the flow was strong enough to carry all SS to the downstream at that time. Hence, the transportable SS concentration was higher during the beginning and the end of the simulation even with the higher incoming sediment. However, the observed concentration is very much closer to the simulated results indicating the good performance of the model simulations shown in Figure 5.29. Due to the limitation of data only one observation point was shown. For an accurate morphological prediction, more observed data is needed.

There was a slight deviation of the simulated results from the observed data. This small deviation can be attributed mainly due to the higher complexity of phenomena involved in the sedimentation of SS. A slight miscalculation of these phenomena may contribute to the higher disagreement between the observed and simulated values. In the simulation of the model, only the mean diameter of the suspended particles was used. The influence of the presence of various grain size fractions in the creek may be responsible for the deviation. In addition, for the simplicity of the model, re-suspension of the bed materials was not considered.
5.4.2.2 Parameters for the Reactive Pollutants

Similar to the parameters of the SS model, the parameters of the nutrients models were estimated for Saltwater Creek. The parameters of the model were estimated with only those water quality data which were collected during rainfall events. The initial values of these parameters were selected from available literature. However, most of the other studies were done by considering fragmental components of the nutrients processes. This study concentrated on estimating the rate parameters for total nutrients, instead of fragmental determination. The values of rate coefficients were estimated by the calibration procedure. Table 5.18 summarises the calibrated values of rate coefficients of the reactive pollutants, i.e. $BOD$, $TN$ and $TP$. The plotted results of the simulation are shown in Figures 5.30 and 5.31 for $TN$ and $TP$ respectively. The simulation was performed with the calibrated parameters and the predicted values were compared with the observed field data.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Values</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rate of oxidation of $BOD$</td>
<td>$k_{BOD}$</td>
<td>0.23</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Rate of $BOD$ loss due to settling</td>
<td>$k_{sBOD}$</td>
<td>-</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Re-suspension rate of $BOD$</td>
<td>$k_{susBOD}$</td>
<td>-</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Half saturation constant for $BOD$</td>
<td>$K_s$</td>
<td>-</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Denitrification coefficient of $TN$</td>
<td>$k_{DEN}$</td>
<td>0.35</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Release rate of $TN$ from $BOD$</td>
<td>$k_{TN}$</td>
<td>0.75</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Release rate of $TN$ from sediment</td>
<td>$k_{sTN}$</td>
<td>0.65</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Release rate of $TP$ from $BOD$</td>
<td>$k_{TP}$</td>
<td>0.40</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Release rate of $TP$ from sediment</td>
<td>$k_{sTP}$</td>
<td>0.35</td>
<td>(1/day)</td>
</tr>
<tr>
<td>Adjustment factor for sediment $TP$ release</td>
<td>$\delta_{TP}$</td>
<td>0.31</td>
<td>-</td>
</tr>
</tbody>
</table>

From Figures 5.30 and 5.31, it is clear that at the beginning and the end of the simulation, there were more pollutant transportation through the creek. However, during these times, the creek discharge was less. As a result, pollutants per unit discharge increased; and hence the transported pollutant concentrations were higher.
The input parameters of the stream water quality model were the output of the catchment water quality model. During the initial period and the end of the simulation, increased pollutants were coming from catchments to the creek; and hence more pollutants were transferred as shown in Figures 5.31 and 5.32.
There was also a sharp change in the simulation of pollutant at the very beginning and the end of the simulation shown in the Figures 5.30 and 5.31 for $TN$ and $TP$ respectively. This phenomenon also happened due to the incoming pollutant concentration of the creek. The contributing catchment areas of the creek were mixed land-uses, i.e. impervious and pervious. During the first time step of a rainfall event, only impervious area contributed to surface runoff as well as pollutant loads, which caused less transportation in the second time step. However, during the third step both impervious and pervious areas were contributing to pollutant loads. The combined contribution of impervious and pervious areas increased the transport rate which affected transportation within the creek as well.

Although the observed values were slightly biased from the simulated ones, the modelling results were reasonable for the water quality variables, $TN$ and $TP$. This deviation can be attributed mainly due to the higher complexity of the phenomena involved in the determination of $TN$ and $TP$. Indeed the concentration of the variables was the results of the several chemical/physical/biological processes ($BOD$ mineralisation, nitrification, denitrification, photosynthesis, atmospheric reaeration, settling, re-suspension from sediment etc.). A slight miscalculation of these processes may contribute to the higher disagreement between the observed and simulated values of stream water quality. In addition, the model does not consider the contribution of $TN$ and $TP$ from photosynthesis of benthic plants. This can be considered as reasonable because for small stream like Saltwater Creek, pollutants contribution from algal activity is negligible (Mannina and Viviani 2010). On the other hand, the model considered saturated level of $DO$ in the creek as the simulation was performed for storm events only.

Overall, the calibrated model responses were in agreement with the observed water quality data. It was seen that the model is quite able to predict the dynamics of $TN$ and $TP$. Therefore, the model is consistent and can be used to generate scenarios as a part of general strategy to conserve or improve the quality of water. Due to the limitation of data only one observation point was shown. For the accurate prediction of pollutants more observed data is needed.
5.5 Conclusions

It is widely accepted that effective and efficient pollution mitigation strategies of a particular watershed area depend on the water quality modelling results. However, the practical application of any mathematical model depends on the proper selection of the appropriate model parameters. The detailed knowledge of the controlling model parameters of any water quality model will help for the reliable prediction of water borne pollutants.

The ideal method to determine the water quality model parameters is the adoption of the calibration procedure using adequate and reliable hydrologic and water quality data. In the absence of adequate data, models are calibrated by visual comparison with other widely used models or from information of available literature. If regional equations (correlating to the model parameters to the watershed basins and other details) are available, they can be used to estimate the model parameters. However, to derive regional equations, it is also necessary to determine the model parameters accurately through the calibration procedure. Hence, for the application of the developed integrated model, it is necessary to estimate the required model parameters.

This chapter of the thesis has presented the results of the research undertaken to estimate the parameters of the developed integrated catchment-stream water quality model. The aim was to estimate the regional parameters of the model. The model parameters were estimated from the both experimental and measured field data. Since the output of the catchment water quality model is used as the input of the stream water quality model, the catchment water quality model parameters were determined before the estimation of the stream water quality model parameters by the calibration procedure. The calibrated model parameters revealed which of the different model equations best represented the field data to catchment and stream.

Calibration of the model was intended to determine a range of parameter values for the both water quantity and quality models. Calibration of the model was made by means of an iterative process of trial and error by adjusting the parameters and comparing the simulation results with the observed data. Calibration of the catchment water quality model was demonstrated for two catchments (HTCC and SWCC), Gold Coast, eastern Australia.
The runoff estimation from the hydrologic model was compared with the calculated discharges from widely used Australian runoff models, i.e. the rational method, WBNM and DRAINS. HTCC is predominantly pervious with only 2.41% imperviousness. So, the pervious surface parameters can be considered as the dominating parameters of the model for HTCC. On the other hand, SWCC is of mixed land-use with 33.5% impervious. So, both impervious and pervious surfaces dominated the pollutant accumulation and transportation for SWCC. Therefore, the values of the model parameters were estimated for impervious and pervious surfaces separately.

Calibration of the stream water quality model was performed for Saltwater Creek. The output of the catchment water quality model was used as the input of the stream water quality model. The estimated discharges of the streamflow model were evaluated and compared with the calculated streamflow of MUSIC and HEC RAS which are widely used models in Australia. The initial values of the rate parameters of the reactive pollutants were determined from available literature.

After estimating the model parameters from the observed data, it can be concluded that the dynamics of the non-point source pollutants build-up and wash-off are still not well-known. The water quality component of the model is much more complex having many parameters/coefficients to be determined. The model parameters vary a lot depending on geographical locations, land-uses and the percentage of imperviousness of catchment surfaces. The values of coefficients and exponents are different for each of the water quality parameters associated with the build-up and wash-off models. This is to be expected because each pollutant has different accumulation and transformation behaviour. The model parameters were treated as the spatially and temporally averaged constants.

Comparing the observed field data with the simulated results using the calibrated parameters, it was found that the results are satisfactory. The model provided excellent representation of the field data demonstrating the simplicity yet effectiveness of the proposed model. The estimated parameters can be used for the analysis of the long term pollutants behaviour and the impact of the land-use changes. They parameters are expected to be useful for the future use of the model and to the development of BMPs in this region.
The outcomes of the study suggest that the model is a possible tool, which will aid in the development of management strategies for complex watershed areas like HTCC and SWCC. The calibrated model responses are in agreement with the observed water quality data. Therefore, the model is consistent and can be used to generate scenarios as a part of the general strategies to conserve or improve the quality of water. The calibrated model can be used to assess the effectiveness of management decisions.

It should be noted that calibration of the model parameters was performed with only a limited number of observations. Therefore, care should be taken when using the findings. Calibration of water quality models should be performed from real measurements for a specific area for the continuous storm events. The ultimate predictive capability of the model should be checked with the observed field data.
Chapter 6
Sensitivity of the Model Parameters

6.1 Introduction
Sensitivity analysis is a formalised procedure to identify the changes in model responses due to the changes in the various model parameters (Snowling and Kramer 2001). This analysis forms an important part of the model validation process where model development and data gathering activities are focused (Newham 2002). The sensitivity of the model parameters illuminates information on the following types of questions (Beres and Hawkins 2001):

- Which of the model parameters exert a significant influence on the modelling output?
- Which parameters are inconsequential?
- Do increments of any parameter produce unexpectedly large alterations in the results?

Without answering these questions, proper understanding of any mathematical model responses remains incomplete. Thus, sensitivity analysis is a significant aspect of every modeller's job.

The general purpose of sensitivity analysis is to determine which input parameters apply the most influence on the model results (Dayaratne 2000). This analysis also helps to acquire the detailed knowledge of the controlling model parameters of any particular model. Sensitivity analysis increases the modeller’s understanding about the techniques considered in the model development. Moreover, through sensitivity analysis, we can learn how to select model complexity and how to improve the quality of information derived from models for planning processes (Laenen and Dunnette 1997). In addition, sensitivity analysis is used to give insights into interactions between the different components of a mathematical model (Newham 2002). Therefore, Beres and Hawkins (2001) noted that recommendations based upon a mathematical model without an explicit sensitivity analysis lack a basic foundation.
In a mathematical model, there may have several parameters which need to be determined. However, the parameters to which the modelling output is sensitive and which have significant uncertainty require special attention in their determination. Hence, it is important to know the most sensitive parameter which needs to be calculated with great care. It is also important to identify those parameters that have little influence on the behaviour of models so that they may be aggregated, modified or removed (Newham 2002). Therefore, sensitivity analysis is an integral part of any mathematical model simulation which is commonly used to examine the model behaviour; and hence sometimes influences the model formulations.

This chapter of the thesis was designed to depict the detailed investigation of different sensitive parameters of the developed model. Sensitivity analysis was performed to better pin down the most sensitive parameters of the model and to evaluate different roles played by the water quality modelling processes. The chapter starts with a review of sensitivity analysis approaches to provide a context for the examination of the model parameters. After that the chapter describes sensitivity analysis of the different parameters of the developed integrated model. The analysis was performed on both catchment and stream water quality models. In each case, careful consideration was given to inherent limitations of the modelling techniques.

### 6.2 Sensitivity Analysis Methods

The general procedure of sensitivity analysis is to define a model output variable that represents an important aspect of the model behaviour. The values of various input parameters are then varied and the resultant changes in the output variables are monitored. The variation of the input parameters can be done in various ways. Hence, there is a wide range of sensitivity analysis techniques described in literature.

Research on model sensitivity was undertaken to improve understanding of the behaviour of the models. However, there is a general paucity of literature which reviews the methods of sensitivity analysis to the model components and data inputs (Newham 2002). Beres and Hawkins (2001) also noted that a well-accepted procedure on the performance of sensitivity analysis is often lacking. This reflects the difficulty in generating a general approach for sensitivity analysis across a broad range of models.
Campolongo et al. (2000) identified the three main settings of sensitivity analysis of a mathematical model:

- Factor screening: to identify the most influential factor in a system with many factors
- Local sensitivity analysis: to identify the local impact of the model parameters and involves the use of partial derivatives
- Global sensitivity: to identify the output uncertainty to the uncertainty in the input parameters

The classification provides a useful means of structuring the current review of sensitivity analysis. This information can be used to simplify the structure and parameterisation of any mathematical model and its improvement for future application to specific problems (Newham 2002).

Sensitivity analysis of the developed model was performed to a variety of model input parameters, including the state variables, the environmental variables and the initial conditions. The analysis of the model parameters was accomplished to assess the importance of the various model parameters by showing how the simulation can be changed with the changed input parameters. The factor screening sensitivity measure was used for this research thesis. The screening method was chosen because it is useful in dealing with models containing a large number of parameters (Newham 2002), and the developed model contains a large number of parameters. In many respects, this method is similar to the local sensitivity analysis technique. This procedure holds ‘‘all other things are equal’’; while each parameter is altered to determine its effect in isolation from the possible effects of other values (Beres and Hawkins 2001).

**6.3 Sensitivity of the Catchment Water Quality Model**

Sensitivity analysis of the catchment water quality model was performed for the investigation of two main phenomena. Firstly, the logical sensitivity of the model was performed to investigate the capability of the continuous simulation and pollutant availability on catchment surfaces. Secondly, the influences of the input model parameters to the output were investigated to demonstrate the most influential parameter.
6.3.1 Sensitivity to Model Capability

As a first step to demonstrate the usefulness and capability of the continuous simulation of surface runoff and water quality parameters, the model was used to simulate runoff and SS loads from a hypothetical catchment using the assumed values of the different model parameters. The purpose was to assess whether the model was capable of generating logical trends that are consistent with the expected runoff and pollutant wash-off behaviour. The model was applied for the simulation of two different rainfall-runoff and pollutant loads scenarios.

In the first simulation, two rainfall events were assumed to be 29 hours apart with similar intensities, as shown in Figure 6.1. The chosen catchment had an area of 25 km$^2$ and runoff slope of 1 m/km. The rainfall loss parameters were assumed to be in accordance with IEAust (2001), with $IL = 15$ mm. The values of $CL$ were assumed to follow the exponentially decreasing loss model. Coefficients of the $CL$ model were assumed to be $A_{CL} = 1.5$ and $B_{CL} = 3.0$ so that the values of $CL$ recline within the recommended values suggested by ARR. For the calculation of the pollutant accumulation on the catchment surfaces, the power function build-up equation was used. The parameters of the power function build-up model for impervious surface were: $C_1(p,s) = 6000$ kg/km$^2$, $C_2(p,s) = 2600$ kg/km$^2$, $C_3(p,s) = 0.16$; and for pervious surface were: $C_1(p,s) = 3000$ kg/km$^2$, $C_2(p,s) = 800$ kg/km$^2$, $C_3(p,s) = 0.16$. For the calculation of the transported pollutants, the power function wash-off model was used. The values of the parameters used for the wash-off model were: $E_1(p,s) = 0.10$, $E_2(p,s) = 0.15$ for impervious surface area; and $E_1(p,s) = 0.07$, $E_2(p,s) = 0.05$ for pervious surface area.

The plotted data in Figure 6.1 shows the rainfall and SS simulation results. The simulation results of the model show that SS wash-off for the second rainfall event was a little bit lower than the SS wash-off for the first event. Although the same rainfall intensities were used for the both rainfall events, the SS wash-off for the first rainfall event was the higher than that for the second rainfall event. This was because following SS washed-out during the first event, the intermediate dry period was not long enough to build-up significant additional SS to produce a higher wash-off during the second rainfall event. In addition, there was no SS wash-off between the two rainfall events.
This phenomenon happened, because at that time there was no rainfall to produce surface runoff which could transport pollutant loads from the catchment surfaces. Similar wash-off patterns were observed for water quality parameters of \( TN \) and \( TP \).

![Simulation of rainfall and SS wash-off for isolated rainfall events](image)

Figure 6.1: Simulation of rainfall and SS wash-off for isolated rainfall events

In the second simulation, the performance of the model in simulating a significant continuous rainfall event of 32 hrs duration was tested. The values of the runoff and pollutant model parameters, and catchment characteristics were the same as the first simulation except the rainfall event. For a continuous rainfall event, rainfall loss declines with time and surface runoff lasts for an extended period of time. However, the pollutant wash-off is limited by the amount of available pollutants on a catchment surface which is mostly removed during FF. The data in Figure 6.2 shows the simulation of catchment runoff and SS simulation for the long continuous rainfall event. The model simulation revealed the obvious FF phenomenon with significant SS wash-off at the early stage of the storm. As rainfall continued, the pollutant wash-off decreased with the time to negligible values. The data in Figure 6.2 also indicates that at one stage of the decreasing trend, there was a sudden increase in wash-off generating a localised SS peak, which can be attributed to the time difference between the arrival of SS from impervious and pervious areas of the catchment.
Surface runoff from impervious area started earlier than from pervious surface. The contribution of pervious surface SS starts upon the arrival of runoff from pervious surface. Since both impervious surface and pervious surface started to wash-off together, there was an increased SS wash-off as shown in Figure 6.2.

The above simulations demonstrated that the developed model is capable of generating logical runoff and SS wash-off trends that are consistent with the expected behaviour. However, the usefulness of the model required further testing using real data.

6.3.2 Parameters Sensitivity

To investigate the changes in the model output with the changes in the input parameters, sensitivity analysis of the calibrated model parameters was performed. The calculated model parameters were varied one at a time around standard (control) values and the magnitude of residuals was compared in order to evaluate the parameters to which the model is significantly sensitive. The purpose was to identify the most important parameter from amongst a large number that may affect model responses. The analysis was performed around the optimal calibrated parameters in order to assess the impact of selecting the inaccurate values of the model parameters.
To find out the influential parameters of the model, the sensitivity of both the build-up model and the wash-off model parameters were performed. Tables 6.1 to 6.3 show sensitivity of the $PBIAS$ values due to the perturbation of the build-up and wash-off model parameters of the developed catchment water quality model for $SS$, $TN$ and $TP$ respectively. Sensitivity of the $NSE$ values due to the perturbation of the build-up model and the wash-off model parameters for $SS$, $TN$ and $TP$ are shown in Tables 6.4 to 6.6.
Table 6.3: Sensitivity of the PBIAS values for TP parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -20.00%</th>
<th>Perturbation -10.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +10.00%</th>
<th>Perturbation +20.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_1(p,s)$</td>
<td>0.249</td>
<td>0.111</td>
<td>0.000</td>
<td>-0.091</td>
<td>-0.167</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>0.250</td>
<td>0.112</td>
<td>0.000</td>
<td>-0.089</td>
<td>-0.167</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.049</td>
<td>0.025</td>
<td>0.000</td>
<td>-0.023</td>
<td>-0.047</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.144</td>
<td>0.063</td>
<td>0.000</td>
<td>-0.051</td>
<td>-0.093</td>
</tr>
<tr>
<td>$E_1(p,s)$</td>
<td>0.239</td>
<td>0.106</td>
<td>0.000</td>
<td>-0.088</td>
<td>-0.161</td>
</tr>
<tr>
<td>$E_2(p,s)$</td>
<td>0.451</td>
<td>0.205</td>
<td>0.000</td>
<td>-0.171</td>
<td>-0.313</td>
</tr>
<tr>
<td>$E_3(p,s)$</td>
<td>0.248</td>
<td>0.082</td>
<td>0.000</td>
<td>-0.110</td>
<td>-0.166</td>
</tr>
<tr>
<td>$E_4(p,s)$</td>
<td>0.425</td>
<td>0.197</td>
<td>0.000</td>
<td>-0.170</td>
<td>-0.314</td>
</tr>
</tbody>
</table>

Table 6.4: Sensitivity of the NSE values for SS parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -20.00%</th>
<th>Perturbation -10.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +10.00%</th>
<th>Perturbation +20.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_1(p,s)$</td>
<td>0.931</td>
<td>0.986</td>
<td>1.000</td>
<td>0.991</td>
<td>0.969</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>0.931</td>
<td>0.986</td>
<td>1.000</td>
<td>0.991</td>
<td>0.969</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.979</td>
<td>0.995</td>
<td>1.000</td>
<td>0.996</td>
<td>0.984</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.999</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$E_1(p,s)$</td>
<td>0.942</td>
<td>0.988</td>
<td>1.000</td>
<td>0.992</td>
<td>0.972</td>
</tr>
<tr>
<td>$E_2(p,s)$</td>
<td>0.998</td>
<td>0.999</td>
<td>1.000</td>
<td>0.999</td>
<td>0.998</td>
</tr>
<tr>
<td>$E_3(p,s)$</td>
<td>0.928</td>
<td>0.984</td>
<td>1.000</td>
<td>0.990</td>
<td>0.843</td>
</tr>
<tr>
<td>$E_4(p,s)$</td>
<td>0.986</td>
<td>0.996</td>
<td>1.000</td>
<td>0.997</td>
<td>0.991</td>
</tr>
</tbody>
</table>

From Tables 6.1 to 6.6, it is clear that the maximum build-up rate $C_1(p,s)$ and the build-up rate coefficient $C_2(p,s)$ were the sensitive parameters of the build-up model. A small change in the values of the parameters $C_1(p,s)$ and $C_2(p,s)$, caused the large changes in the statistical parameters PBIAS and NSE as can be seen in Tables 6.1 to 6.6 compared with the build-up exponent $C_3(p,s)$ and the accumulation rate coefficient $k_p(p,s)$. Therefore, these parameters $C_1(p,s)$ and $C_2(p,s)$ of the build-up model should be estimated with great care. However, in between $C_3(p,s)$ and $k_p(p,s)$, the parameter $k_p(p,s)$ was the least sensitive parameter.
Table 6.5: Sensitivity of the NSE values for TN parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -20.00%</th>
<th>Perturbation -10.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +10.00%</th>
<th>Perturbation +20.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_1(p,s)$</td>
<td>0.923</td>
<td>0.985</td>
<td>1.000</td>
<td>0.990</td>
<td>0.966</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>0.923</td>
<td>0.985</td>
<td>1.000</td>
<td>0.990</td>
<td>0.984</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.970</td>
<td>0.993</td>
<td>1.000</td>
<td>0.994</td>
<td>0.986</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$E_1(p,s)$</td>
<td>0.960</td>
<td>0.992</td>
<td>1.000</td>
<td>0.994</td>
<td>0.980</td>
</tr>
<tr>
<td>$E_2(p,s)$</td>
<td>0.944</td>
<td>0.987</td>
<td>1.000</td>
<td>0.999</td>
<td>0.962</td>
</tr>
<tr>
<td>$E_3(p,s)$</td>
<td>0.916</td>
<td>0.983</td>
<td>1.000</td>
<td>0.989</td>
<td>0.963</td>
</tr>
<tr>
<td>$E_4(p,s)$</td>
<td>0.634</td>
<td>0.917</td>
<td>1.000</td>
<td>0.938</td>
<td>0.791</td>
</tr>
</tbody>
</table>

Table 6.6: Sensitivity of NSE values for TP parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -20.00%</th>
<th>Perturbation -10.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +10.00%</th>
<th>Perturbation +20.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_1(p,s)$</td>
<td>0.924</td>
<td>0.985</td>
<td>1.000</td>
<td>0.990</td>
<td>0.966</td>
</tr>
<tr>
<td>$C_2(p,s)$</td>
<td>0.924</td>
<td>0.985</td>
<td>1.000</td>
<td>0.990</td>
<td>0.966</td>
</tr>
<tr>
<td>$C_3(p,s)$</td>
<td>0.997</td>
<td>0.999</td>
<td>1.000</td>
<td>0.999</td>
<td>0.997</td>
</tr>
<tr>
<td>$k_p(p,s)$</td>
<td>0.975</td>
<td>0.995</td>
<td>1.000</td>
<td>0.997</td>
<td>0.990</td>
</tr>
<tr>
<td>$E_1(p,s)$</td>
<td>0.932</td>
<td>0.986</td>
<td>1.000</td>
<td>0.991</td>
<td>0.969</td>
</tr>
<tr>
<td>$E_2(p,s)$</td>
<td>0.777</td>
<td>0.953</td>
<td>1.000</td>
<td>0.966</td>
<td>0.886</td>
</tr>
<tr>
<td>$E_3(p,s)$</td>
<td>0.921</td>
<td>0.990</td>
<td>1.000</td>
<td>0.985</td>
<td>0.964</td>
</tr>
<tr>
<td>$E_4(p,s)$</td>
<td>0.840</td>
<td>0.961</td>
<td>1.000</td>
<td>0.965</td>
<td>0.872</td>
</tr>
</tbody>
</table>

The changes in parameters $C_3(p,s)$ and $k_p(p,s)$ did not produce a significant change in the modelling output; and hence in the PBIAS and NSE values. The variations of the plotted results of sensitivity analysis are shown in Figures 6.3 to 6.6 for the build-up model parameters; and Figures 6.7 to 6.10 for the wash-off parameters for SS. The sensitivity of the build-up parameters followed the same pattern. The analysis was performed by changing each parameter; while keeping others constant and observing the changes in the model output.
Similar to the build-up model parameters, Tables 6.1 to 6.3 show the changes in the PBIAS values; and Tables 6.4 to 6.6 show the changes in the NSE values due to the perturbation of the wash-off model parameters. The plotted variations of the percentage output change in pollutant loads to the perturbation of the wash-off model parameters are shown in Figures 6.3 to 6.6 for SS. The variations in the sensitivity of the wash-off model parameters for the other pollutants (TN and TP) follow the same pattern.

Figure 6.3: Sensitivity of the build-up parameter $C_1(p,s)$ for SS

Figure 6.4: Sensitivity of the build-up parameter $C_2(p,s)$ for SS
From Tables 6.1 to 6.6 and Figures 6.7 to 6.10, it is clear that the pollutant wash-off coefficient $E_1(p,s)$ was the most sensitive parameter for the power function wash-off model. On the other hand, for the rating curve wash-off model, the coefficient of the rating curve wash-off function $E_2(p,s)$ was the most sensitive parameter. The changes in all other parameters did not produce a significant change in the simulation results.
For both the power function and the rating curve wash-off models, the output variations were much larger during the peak flow. The large changes in the output variables imply that the particular input variable is important in controlling the model behaviour. Similar to SS, the results for TN and TP follow the same pattern of sensitivity. It should be noted that the output responses were non-linear across the examined parameters.

Figure 6.7: Sensitivity of the wash-off model parameter $E_1(p,s)$ for SS

Figure 6.8: Sensitivity of the wash-off model parameter $E_2(p,s)$ for SS
6.4 Sensitivity of the Stream Water Quality Model

Similar to the catchment water quality model, sensitivity analysis of the developed stream water quality model was also performed to assess the most influential parameter. For this analysis, a 1,825 m section of Saltwater Creek was chosen. Since the model was comprised of two sub-models, i.e. streamflow and stream pollutants processes, the sensitivity of the parameters were performed for the both sub-models.
6.4.1 Sensitivity of Streamflow Parameters

Sensitivity analysis of the developed streamflow model parameters was performed to assess their effects on the simulation of stream water quantity. Since, the measured streamflow data were not available; this assessment was accomplished with the commercially available models, MUSIC and HEC RAS. The comparisons of the plotted results are illustrated in Figures 5.11 to 5.13.

![Figure 6.11: Sensitivity with other models with variable $K_{stor}$ values](image)

From Figure 6.11, it is clear that the simulated streamflow hydrograph was very much closer to the hydrograph of MUSIC, which was much higher than the simulated results of the HEC RAS model. This was reasonable because both the developed model and MUSIC used the Muskingum-Cunge method for the prediction of downstream discharges of the creek. The Muskingum-Cunge method was developed based on the equation of continuity. On the other hand, the HEC RAS streamflow simulation was calculated by solving the energy equation with an iterative procedure called the standard step method. For the simulation of the HEC RAS model, the value of Manning’s roughness coefficient was assumed to 0.03. Since the Muskingum-Cunge method does not require the value of Manning’s ‘$n$’, it was not considered for the developed model and for MUSIC.
The value of $K_{stor}$ was determined to 15 minutes for MUSIC. For the developed model, the value of $K_{stor}$ was variable with the depth of flow. The value of the other routing parameter $X_P$ was assumed to 0.15 both for MUSIC and for the developed model. A large difference between the model simulation and the HEC RAS simulation was also observed.

When the simulation of the developed model was performed with the fixed $K_{stor}$ value, the output response was very close to the calculated discharges of the HEC RAS model. However, the simulation results deviated from the estimated flows of MUSIC as shown in Figure 6.12. This slight deviation was due to the consideration of the fixed value of $K_{stor}$. Therefore, we can conclude that there is a relationship between the Muskingum parameter $K_{stor}$ and the standard step method of energy equation. The future research direction will be focused on the exploration of this relationship.

The sensitivity of the Manning’s ‘$n$’ values of the HEC RAS model was also performed. For this analysis, the value of $X_P$ was 0.15; and the value of $K_{stor}$ was fixed to 15 minutes for the developed model. Figure 6.13 demonstrates the sensitivity of the HEC RAS model with different Manning’s ‘$n$’ values and their comparison with the developed model.
From Figure 6.13, it is clear that the lower ‘n’ values produced the higher peak discharge and the higher ‘n’ values created the lower peak discharge in the HEC RAS model. It is reasonable because the higher ‘n’ value means more friction and the reduced peak. The figure also reveals that the model responses were very much closer to the calculated discharges of HEC RAS with $n = 0.030$. This was also reasonable because the usual value of Manning’s ‘n’ is 0.030 for a natural stream.

### 6.4.2 Sensitivity of SS Parameters

The predominant mode for the sediment transportation and deposition depends on the size, shape and density of particles in respect to the velocity and turbulence field of water bodies (Celik and Rodi 1989). To identify the influential parameters of SS, the developed model was simulated under different scenarios. The individual simulation runs for the transportation and deposition of SS were performed on the basis of streamflow, the inflow sediment concentration and the characteristics of the sediment particles. In these simulations, only mean diameter of suspended particles was used. Table 6.7 shows the differential influential parameters of the SS processes and the effects of their variations in the simulation of the model during the peak flow on sediment transportation.
From Table 6.7, it is difficult to assess the most influential parameter of the model during the peak flow. Almost all of the model parameters have significant influence on the SS processes during the peak flow. The plotted results of the SS sensitivity analysis are shown in Figures 6.14 to 6.25.

Table 6.7: SS transport sensitivity of the peak flow

<table>
<thead>
<tr>
<th>Influential Parameters</th>
<th>% Increase</th>
<th>% Decrease</th>
<th>Original SS conc. (mg/l)</th>
<th>Increased</th>
<th>Decreased</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>SS conc. (mg/l)</td>
<td>Change (%)</td>
</tr>
<tr>
<td>SS Input</td>
<td>100</td>
<td>50</td>
<td>347.66</td>
<td>347.66</td>
<td>0.00</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>100</td>
<td>50</td>
<td>347.66</td>
<td>148.81</td>
<td>-57.20</td>
</tr>
<tr>
<td>$\rho_s$</td>
<td>25</td>
<td>12.5</td>
<td>347.66</td>
<td>174.67</td>
<td>-49.76</td>
</tr>
<tr>
<td>$Te$</td>
<td>100</td>
<td>50</td>
<td>347.66</td>
<td>320.10</td>
<td>-7.93</td>
</tr>
</tbody>
</table>

6.4.2.1 Influence of Streamflow

Streamflow has a significant influence on the transportation as well as the deposition of SS to the downstream and in the stream bed (Ali et al. 2010). As a general rule, the transport rate of SS increases with the increased flow rate and decreases with the decreased flow rate. However, the rate of deposition increases with the decreased flow rate and decreases with the increased flow rate. To demonstrate the influence of streamflow rate, the model was simulated for different streamflow with the same inflow of the SS concentration. The variations of the simulated results are shown in Figures 6.14 and 6.15 for SS transportation, and Figures 6.16 and 6.17 for SS deposition.

Figure 6.14 shows the simulation results of the SS transportation for the three different streamflow. From the figure, it is clear that the transport rate was higher during the higher streamflow rate and the lower during low streamflow rate. However, for a very high streamflow rate (i.e. during the flood), all of the inflow SS was transported to the downstream. At that time, the inflow sediment concentration was equal to the outflow sediment concentration and there was no sediment deposition as shown in Figure 6.15.
Figures 6.16 and 6.17 demonstrate the variations of the SS deposition rate with varied streamflow. Similar to the transport rate, for all the simulation runs, the incoming SS concentration was the same. Figure 6.16 reveals the obvious phenomena that deposition of SS occurred under low flow conditions. However, during the flow rate, all sediments were transported to the downstream.
At this stage i.e. during the high streamflow rate, the inflow sediment concentration of a particular stream reach is equal to the outflow sediment concentration of that reach; and hence the sediment deposition was zero. The simulation results of the SS deposition during the higher flood are shown in Figure 6.17. It is to be noted that for the simplicity of the model, the scouring processes were not considered in the model.

![Figure 6.16](image1.png)

**Figure 6.16:** Influence of streamflow on SS deposition during usual flow rate

![Figure 6.17](image2.png)

**Figure 6.17:** Influence of streamflow on SS deposition during high flood
6.4.2.2 Influence of Inflow Sediment Concentration

The amount of sediment transferred from a particular catchment during storm events is another important factor for the SS transportation within stream reaches. The general trend for the SS transportation and deposition relies on the inflow sediment concentration coming from contributed catchments to stream. van Rijn (1986) found a large inaccuracy in the determination/assumption of the incoming sediment concentration used as boundary conditions at the inlet. Therefore, an accurate estimation of the inflow sediment concentration is essential. To demonstrate the influence of incoming SS, the model was simulated with the three different inflow concentrations at a particular stream reach. The plotted results of the SS transportation and deposition rates are shown in Figures 6.18 and 6.19 respectively.

Figure 6.18: The influence of incoming SS concentrations on transportation

Figure 6.18 shows the variations of the simulated results for the transportation of SS through a particular stream section with the selected varying inflow sediment concentrations. From the figure, for this particular condition, it was found that the transport rate of SS was the same even if the incoming sediment concentrations were double. This supports the fact that the transport rate of SS from a particular stream reach for a particular flow rate depends on the transport capacity of that section, i.e. the stream
has a maximum capacity to transport SS to the downstream. Where the concentration of SS is more than the transport capacity of the stream, SS is deposited onto the stream bed. Figure 6.19 illustrates the variations of SS deposition for that particular section. The influence in the SS deposition appears to be large especially during the rising and falling limb of the hydrographs. From Figure 6.19, it is clear that when the incoming sediment concentration was higher, the deposition rate was higher. For very low concentration of the incoming sediment, there was no deposition, i.e. all the sediments were transported to the downstream. It should be noted that the streamflow was the same for all the three different inflow sediment concentrations.

![Figure 6.19: The influence of the incoming SS concentration on deposition](image)

6.4.2.3 Influence of Median Particle Diameter

The median particle diameter ($D_{50}$) is one of the most important factors in the transportation and deposition of SS. van Rijn (2007) found that the transportation of SS is strongly related to the particle size. Nicholas and Walling (1998) also noted that the deposition rate is strongly controlled by the particle fall velocity, which is usually determined from $D_{50}$. In most of the practical cases, data for $D_{90}$ is not readily available, so it is determined from $D_{50}$. To show the variations of $D_{50}$ on the SS processes in a particular stream section, the model was simulated with different $D_{50}$. 

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The graphical presentation of these variations is shown in Figures 6.20 and 6.21 for SS transportation and deposition respectively. Figure 6.20 demonstrates the variations of the transport rate with the variations of $D_{50}$. The figure reveals that the transport rate increases with decreased $D_{50}$ and decreases with increased $D_{50}$. However, the rate of deposition increases with increased $D_{50}$ and decreases with decreased $D_{50}$. 
When $D_{50}$ increases, particle fall velocity will increase, which helps to deposit more particles on the stream bed. However, these variations were not even. Figure 6.21 shows the variations of the deposition rate with the variations of $D_{50}$. From the figure, it is clear that the size of sediment particles significantly impacts upon the deposition rate. Hence, the field measurement of the particle size is extremely important for the better prediction of the SS transport and deposition rate.

6.4.2.4 Influence of Particle Density

Density of the particles has a considerable influence on the particle fall velocity which is another most influential parameter of the SS processes. Hence, the determination of the effects of sediment density on the SS processes is essential. To illustrate the influence of sediment density on the SS processes, the model was simulated with varying sediment densities; while the inflow sediment concentration was the same. The simulated results of the variations are shown in Figures 6.22 and 6.23 for the transport and deposition rates respectively. Figure 6.22 illustrates the transport rate of SS with the three different sediment densities. The figure reveals the obvious phenomena that transportation of SS is sensitive to the density of the particles. From Figure 6.22 and Table 6.7, it is clear that an increase in sediment density by 25% decreased the transport rate by up to 50% during the peak flow.

![Figure 6.22: The influence of sediment density on SS transportation](image-url)
On the other hand, decrease in sediment density by 25% increased the transport peak by about only 30%. Figure 6.22 shows the plotted results of the SS deposition rate with the three different sediment densities. It is clear from the figure that similar to the transport rate, the deposition of SS was also sensitive to the density of the sediment particles. If the sediment density went much lower, there was no SS deposition, i.e. all the sediment was transported to the downstream. However, those outcomes will largely depend on the other parameters, such as streamflow, $D_{50}$, water temperature etc. In general, the SS processes are highly depended on the particle density and its field measurement is highly important.

Figure 6.23: The influence of sediment density on SS deposition

6.4.2.5 Influence of Water Temperature

Water temperature is also another important parameter in determining the transport and deposition rate of SS. With the increase in water temperature, water density reduces causing an increase in sediment fall velocity. Figures 6.24 and 6.25 show the influence of water temperature on the SS processes. Figure 6.24 demonstrates the variations of SS transportation with the temperature which represents the obvious phenomena that the transport rate decreases with increasing temperature and increases with decreasing temperature. From Figure 6.24 and Table 6.7, it was found that the peak transport rate varies more than 7% for a temperature variation of 100%.
Figure 6.24: Effects of the temperatures on SS transportation

Figure 6.25: Effects of the temperatures on SS deposition

Figure 6.25 demonstrates the sensitivity of SS deposition with the temperature variations. From Figure 6.25, it is clear that the deposition rate increases with increasing temperature and decreases with decreasing temperature. From the detailed analysis, it was also observed that the change in the peak sediment transport and deposition was linear with the temperature variations. Hence, an accurate measurement of water temperature will increase the accuracy of model predictions.
6.4.3 Sensitivity of TN and TP Parameters

Although the modelling processes of TN and TP are widely used in water quality modelling, there is the lack of systematic testing of sensitivities of the rate parameters. In this research, the sensitivity of the developed model was undertaken to improve understanding of the behaviour of the developed model. This information will be used to simplify and improve the model structure and parameterisation where possible. The factor screening sensitivity measure was used to assess the impact of the perturbations of the model parameters.

The sensitivity of the rate parameters of the reactive pollutants (TN and TP) for the stream water quality model was identified in terms of the PBIAS and NSE values. The summary results for sensitivity analysis of the rate parameters are shown in Tables 6.8 and 6.9. Table 6.8 shows the sensitivity results of PBIAS values; while Table 6.9 shows the sensitivity results of NSE values.

Table 6.8 reveals that in terms of the PBIAS values, the output of the TN processes were more sensitive to the perturbation of the coefficient for mineralisation of TN from BOD ($k_{TN}$), and less sensitive to the denitrification coefficient ($k_{DEN}$) and release rate of TN from sediment ($k_{sTN}$). There was no change in the PBIAS values due to the perturbation of $k_{DEN}$ as shown in Table 6.8. The initial value of $k_{DEN}$ was assumed based on published literature which considered only a fragment of nitrogen instead of TN.

Table 6.8: Sensitivity of PBIAS values of the rate parameters for TN and TP

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -10.00%</th>
<th>Perturbation -5.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +5.00%</th>
<th>Perturbation +10.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_{BOD}$</td>
<td>0.001</td>
<td>0.0002</td>
<td>0.000</td>
<td>0.0003</td>
<td>0.001</td>
</tr>
<tr>
<td>$k_{DEN}$</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>$k_{TN}$</td>
<td>0.092</td>
<td>0.0432</td>
<td>0.000</td>
<td>0.041</td>
<td>0.029</td>
</tr>
<tr>
<td>$k_{sTN}$</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>$k_{TP}$</td>
<td>0.233</td>
<td>0.104</td>
<td>0.000</td>
<td>-0.086</td>
<td>-0.159</td>
</tr>
<tr>
<td>$k_{sTP}$</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>$\delta_{TP}$</td>
<td>-0.845</td>
<td>-0.043</td>
<td>0.000</td>
<td>0.050</td>
<td>0.102</td>
</tr>
</tbody>
</table>
Similar to $k_{DEN}$, there was no change in the $PBIAS$ values due to the perturbation of $k_{sTN}$. This was reasonable because the considered stream was shallow in depth and there was no deposited sediment to release $TN$ during the simulation period. Table 6.8 also reveals that the predicted values of the $TP$ processes were more sensitive to the perturbation of the adjustment factor for sediment $TP$ release ($\delta_{TP}$) and to the coefficient for mineralisation of $TP$ from $BOD$ ($k_{TP}$); and less sensitive to the release rate of $TP$ from sediment ($k_{sTP}$) in terms of the $PBIAS$ values. However, $k_{TP}$ and $\delta_{TP}$ are interrelated because both of these parameters are associated with sediment $TP$ release. Hence, all of the parameters should be estimated with great care from the observed field data.

Table 6.9: Sensitivity of $NSE$ values of the rate parameters for $TN$ and $TP$

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Perturbation -10.00%</th>
<th>Perturbation -5.00%</th>
<th>Perturbation 0.00%</th>
<th>Perturbation +5.00%</th>
<th>Perturbation +10.00%</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_{BOD}$</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>0.999</td>
</tr>
<tr>
<td>$k_{DEN}$</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$k_{TN}$</td>
<td>0.991</td>
<td>0.998</td>
<td>1.000</td>
<td>0.998</td>
<td>0.957</td>
</tr>
<tr>
<td>$k_{sTN}$</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$k_{TP}$</td>
<td>0.976</td>
<td>0.994</td>
<td>1.000</td>
<td>0.994</td>
<td>1.068</td>
</tr>
<tr>
<td>$k_{sTP}$</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$\delta_{TP}$</td>
<td>0.996</td>
<td>0.999</td>
<td>1.000</td>
<td>0.999</td>
<td>0.975</td>
</tr>
</tbody>
</table>

Table 6.9 shows the sensitivity of the rate parameters of $TN$ and $TP$ in terms of $NSE$ values for the estimated parameters of Saltwater Creek. From the table, it is clear that for all the perturbations, the $NSE$ values were very close to 1.0 which contradicts with the sensitivity of the $PBIAS$ values. This contradiction reveals that the estimated parameters may not be represented for the creek. It should be noted that due to the limitations of measured data, the parameters were estimated only from a limited number of observations. Furthermore, the perturbation of the model parameters was done based on the estimated parameters of SWCC only. Hence, for the real time sensitivity, the model parameters should be estimated from the adequate and reliable observed field data.
6.4.3.1 Influence of Hydraulic Radius

The usual practice of the hydraulic radius is that it is assumed to be the depth of flow. This assumption is quite good for wide channels where width of a stream channel is much greater \((B_W > 5h)\) than the depth of flow. However, for the streams with a small width, this assumption will produce gross error. To reduce this error, the model was developed by considering the hydraulic radius as a power function of the flow depth by introducing the two new terms \(CFc\) and \(CFe\). Figure 6.26 shows the simulation results of SS for Saltwater Creek by considering it as having a wide channel and by estimating \(CFc\) and \(CFe\). The values of \(CFc\) and \(CFe\) were estimated from the calculated hydraulic radius and flow data estimated by the HEC RAS model. From Figure 6.26, it is clear that the transportation of SS was almost the same for both cases, i.e. when the channel was wide as well as non-wide. Hence, someone might argue that the estimation of \(CFc\) and \(CFe\) is useless. It is true for the streams where the SS transport capacity is higher, i.e. when all the incoming sediment is transported to the downstream. However, for too much incoming sediment concentration or for too low streamflow, there was sediment deposition on the stream bed shown in the Figure 6.26. At this stage, the consideration of the hydraulic radius equal to the flow depth produces gross error.

![Figure 6.26.: The variation of hydraulic radius on SS transport](image-url)

6.5 Conclusions

Sensitivity analysis is an important component for evaluating the water quality modelling techniques. This chapter presented the detailed sensitivity analysis of the different parameters of the developed integrated catchment-stream water quality model. The analysis was undertaken to assess sensitivity of individual parameter and to find out the most sensitive parameter of the developed model. This analysis was required as a part of an effort to increase understanding of a modeller’s knowledge about considered processes in the model development and to provide a guide for shifting the scale of model application.

Since the main purpose of sensitivity analysis of this chapter was to obtain a broader feeling of sensitive parameters of the developed model, a simple sensitivity analysis method (i.e. one at a time sensitivity analysis) was used. For this thesis the factor screening approach was used which is useful for providing a guide for the future assessment of the water quality model using a general sensitivity technique. Based on the detailed analysis, the following conclusions were drawn.

By examining the expected direction of the changes in the model output, it can be concluded that the model was coded correctly and behaved as expected. For example, the model can simulate the phenomenon that increases in rainfall intensity and duration does not always increase the pollutant wash-off rate rather it decreases with time from the beginning. Also decrease in $D_{50}$ of SS decreases the amount of deposition on the stream bed. However, the transport rate would remain unaffected by the change in the input SS due to the limited transport capacity of a particular streamflow rate.

From the sensitivity of the catchment water quality model, it was found that maximum build-up rate is the most sensitive parameter of the pollutant build-up model. Another sensitive parameter of this model is the build-up rate constant. On the other hand, coefficients of both the power function wash-off model and the rating curve wash-off model were more sensitive than exponents. Therefore, these coefficients should be determined carefully from calibration. Moreover, it was observed that the pollutant wash-off is dependent on the available pollutants of catchment surfaces. In addition, sensitivity analysis indicated that multiple parameter sets can be derived from the similar level of calibration.
The variability in the parameter values of the catchment water quality model suggests that the generic application of the sophisticated build-up wash-off model should be performed cautiously. Further research is needed to derive the appropriate values of the build-up wash-off model parameters for the practical application with a greater degree of confidence.

From sensitivity analysis of the stream water quality model, it can be concluded that the overall accuracy of the SS processes is largely determined by the accuracy of boundary conditions, such as the inflow concentration of SS coming from upstream catchments to streams. The application of the model is hindered when the accurate boundary conditions, such as field data is missing. In the absence of accurately measured boundary conditions, the simulation leads to the biased results. The other influential parameter of the model is the streamflow rate. From the sensitivity of the TN processes, it was found that the mineralisation coefficient of TN \((k_{TN})\) from BOD was the most sensitive, and the denitrification coefficient \((k_{DEN})\) was the least sensitive parameter. For the TP processes, the most sensitive parameter was the adjustment factor \((\delta_{TP})\) for sediment TP release.

However, the accuracy of the model output responses were conducted based on limited data. For the future sensitivity work of the developed model and similar variants, the investigation of sensitivities across a range of spatial scales and sites is required. The detailed field measurements are essential to specify the appropriate boundary conditions, especially when an accurate morphological prediction is required. These results have important implications for focusing the future research associated with the developed model.

From the overall sensitivity analysis, it can be cautiously concluded that the developed model is generally performing well at predicting the patterns of both discharge and pollutant loads. Reference to this research will guide to the future sensitivity assessment. Sensitivity analysis presented here could be extended potentially to gain additional insight into the behaviour of the model. Further analysis is required before more definitive conclusions can be reached. It will also help to determine the relative importance of the model parameters involved in the modelling processes.
Chapter 7
Conclusions and Recommendations

7.1 Overview of the Study

The primary objective of this research was to develop an integrated catchment-stream water quality model which continuously simulates different water quality parameters. The main aim was to assist watershed managers to establish improved management strategies to protect aquatic environments from the impact of pollution. The procedure of the developed model was based on the hypothesis that an integrated modelling approach can be used to predict water quality parameters from both catchment and stream reach together. The integrated approach was chosen for the model development because single isolated models tend to predict the inconsistent and biased results.

This thesis explored the catchment and stream water quality modelling approaches through a detailed literature review. The literature review identified that at that time there were no integrated catchment-stream water quality model for the prediction of water quality parameters. The review demonstrated the development, application and testing processes of hydrology, hydraulics and water quality parameters. The research achieved its three main aims:

- A catchment water quality model was developed for the prediction of stormwater pollutants SS, TN and TP, which are transported from upstream catchments through a catchment outlet. This was achieved by firstly developing a runoff model and then a pollutant model. The runoff model allows the determination of CL using any of the available estimation methods (i.e. constant, linearly varying or exponentially varying), rather than using a fixed rainfall loss value for the duration of rainfall. The runoff modelling results were compared with the estimated discharges from WBNM, DRAINS and MUSIC. Widely used pollutants processes were refined and incorporated to estimate the export of water quality parameters from catchment surfaces. The catchment water quality model determines surface runoff and pollutant loads, and allows the estimation
of pollutants build-up during the dry periods and pollutants wash-off as a result of rainfall events. The users have the options of choosing the best pollutant build-up and wash-up model from various build-up and wash-off equations.

- A stream water quality model was developed for the continuous simulation of water quality parameters coming from upstream catchments to a stream reach. This was performed by incorporating the stream dynamics and stream pollutants processes. For the streamflow, a simple flow routing model (the Muskingum-Cunge method) was used. The estimated flows of the developed model were compared with the calculated flows from HEC RAS and MUSIC. The techniques of SS and nutrients processes were refined, improved and incorporated with the streamflow model.

- An integration of the catchment water quality model with the stream water quality model was performed to be able to continuously simulate different water quality parameters. Incorporation of the integrated modelling technique of catchment and stream water quality parameters is very useful for the prediction of water borne pollutants.

In addition, to achieving the three main aims of this research, other important outcomes were also achieved. After developing the basic model structure, it was proven that this model could be adapted to catchments and streams, where complex models are not suitable from the practical and economic point of view. The application of the integrated model developed in this research has provided a capacity to quantify water quality parameters.

The developed integrated water quality model, like other water quality models, needs calibration to estimate the model parameters. For this estimation, the manual calibration procedure was used in this study. The method was used to provide the best set of model parameters that considers several storm events simultaneously. For calibration and validation of the integrated model, different published data and reliable source (GCCC) data were used. Rainfall data was collected from the nearby meteorological station ‘Gold Coast Seaway’. The catchment characteristics data was obtained from the land-use maps, contour maps and aerial photographs of the area.
The experimental water quality data was collected from the available literature for the two impervious surfaces, i.e. road and roof surfaces. Field measurement water quality data was collected for the two catchments, i.e. HTCC and SWCC, which are located at Gold Coast, Australia. These two catchments represent typical small and medium watershed areas in Gold Coast. The initial values of the model parameters were also obtained from the available literature. Since catchment water borne pollutants are generated from impervious and/or pervious areas, the parameters of the catchment water quality model were determined for both impervious and pervious areas separately.

Calibration of the stream water quality model was also performed for the Gold Coast area. The parameters of this model were determined from an 1825 m long reach section of Saltwater Creek. The parameters of the Muskingum-Cunge routing technique and pollutants processes were determined separately for the creek.

Finally, the calibrated model parameters, the catchment and stream characteristics were used to assess sensitivity of the model parameters on the simulated hydrographs and pollutographs. This model extends beyond the previous efforts in the area of dynamic water quality modelling approaches and proposes a method for generating a reliable estimation of water quality parameters. The factor screening sensitivity analysis technique was used for this study.

This research into the integrated water quality model enhances the current knowledge about isolated water quality models and provides a direction in further development of integrated models. Furthermore, this research contributes towards the effective design of water quality mitigation strategies for the protection and improvement of aquatic environments and ecosystems from the impact of water pollution.

7.2 Conclusions
This thesis made a number of contributions to the development of an integrated water quality modelling technique for the purpose of improving management of aquatic environments. The goal of the approach was to reduce the complexity and effort using several individual modelling approaches to compute and analyse water quality parameters. The integrated model was tested for the widespread application to catchments and streams with the same physical environments and constraints.
The catchment water quality model can account for the fact that \( CL \) not only remain constant for the whole duration of a storm event but also decreases with time. The phenomena of non-point pollutant build-up and wash-off are influenced by a large number of factors and their dynamics are still not well-known. Based on the analysis of the modelling output, the model parameters should be estimated through the calibration procedure using field measurements for a specific area for several continuous storm events, instead of a single rainfall event. The catchment water quality model provides fairly accurate field data when correctly calibrated with the observed data. It is therefore reasonable to conclude that the developed model can be applied to any catchment in Australia and throughout the world, provided the proper values of the model parameters are selected. The model allows the users to specify time step for both the hydrograph and pollutograph simulation.

The stream water quality model introduced a new technique for the calculation of the hydraulic radius of a particular stream reach instead of taking it equal to the depth of flow. This method of the hydraulic radius calculation is very useful method for water engineering practices. The proposed \( SS \) transport model yields the reliable results in the prediction of the sediment concentration, transportation and deposition. The method was applied to estimate the suspended sediment transportation and deposition, and provided fairly accurate prediction results comparing with the usual methods.

Comparing the modelling results with the commercially used models, it was found that this model produces the similar results. It can therefore be concluded that the proposed technique is promising and may be beneficial to overcome the burden of using individual models. The model will help to provide guidelines not only to resource managers in restoring aquatic ecosystems, but also to local planners devising viable and ecologically sound watershed development plans as well as for policy makers in evaluating alternate land management decisions. The outcomes of the study suggest that the model is a potential tool which will aid in the development of management strategies for complex watershed areas. The model can be used to generate scenarios as a part of a general strategy to conserve or improve the quality of water. The developed model and similar variants have much potential to predict water quality parameters of an aquatic environment.
Both the catchment water quality model and the stream water quality model have complexity commensurate with data availability and the potential for the widespread application. The disadvantage of the model is the need for the accurate field measurement data to calibrate the model parameters. There is also a loss of accuracy, because of unsophisticated representation of the physics of the relevant modelling processes. Another disadvantage is that the model does not consider the cohesive properties of SS particles, such as flocculation, consolidation and fluidisation. These processes were ignored in order to make the model user friendly.

The calibrated model responses and sensitivities are in agreement with the observed water quality data. Since some of the model parameters are interrelated with each other, it is difficult to find out the most sensitive parameter of the model. However, the major conclusion reached through the sensitivity assessment was that boundary conditions of the developed model are the most sensitive. Hence, the investigation to the boundary conditions should be the initial focus in the practical application of the model and its improvement.

The integrated model which has been developed can be used to assess the effectiveness of management decisions about catchments and streams. The output will provide useful data for decision making about existing wastewater treatment plants and/or controlling the non-point source pollutant loadings.

Finally, by replacing the earlier forms of system modelling, the integrated assessment and modelling of water quality parameters will facilitate scenario generation and decision support functions. Further development of this integrated water quality model could eliminate the inconsistent results arising from the isolated models.

### 7.3 Recommendations and Future Research

It is expected that there are spatial variations of rainfall within a particular catchment especially for large rainfall events. The model allows the users to consider data only from one rainfall station. Therefore, this model should be further developed to include the spatial variations of rainfall to make a more accurate prediction of water quality parameters.
It was hypothesised that the characteristics of the influential variables are fairly constant over the studied catchments. The future research should investigate the model parameters for the variable catchment characteristics.

The current catchment water quality model did not consider the $BOD$ process which is one of the main pollutants of streams. For an accurate estimation of $BOD$ within a particular stream, the BOD process should be incorporated with the catchment water quality model.

It should be noted that the estimation of the model parameters in this research was performed with only a limited number of observations. Therefore, further tests of the model are most desirable using more experimental and observed field data. This model was calibrated only for data from Gold Coast. It is suggested that further calibration with data from other catchments in different regions be performed so that a ranged values of the model parameters can be established.

Furthermore, sensitivity of the model parameters was performed by considering the calibrated parameters as optimal. However, the parameters were estimated only with a limited number of observed data as mentioned earlier. Therefore, future investigation of sensitivity analysis needs to be directed across a wide range of spatial scales and sites. Moreover, examination of model sensitivity should be performed by considering the correlation structure of the model parameters to increase the better understanding of the model users.

In addition, as this model does not take into the consideration of economic aspects of management changes, this should be incorporated as a part of pollutant load model to further assist in prioritising management expenditure. The model also does not identify the relationship between the Manning’s ‘$n$’ value and the Muskingum-Cunge method. The relationship needs to be established in the future.
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### Appendix A The Values of Rainfall Loss for Australia

Table A.1: Design Rainfall Loss Rates for Queensland

<table>
<thead>
<tr>
<th>Location</th>
<th>Loss Model</th>
<th>Loss Model and Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eastern Queensland</td>
<td>Initial loss – continuing loss</td>
<td>Initial loss – continuing loss&lt;br&gt;Median continuing loss = 2.5 mm/hr&lt;br&gt;Median initial loss = 15 – 35 mm&lt;br&gt;Initial loss = 0 – 140 mm. Higher values were from rainforest areas. All values obtained by fitting runoff routing model to observed floods</td>
</tr>
<tr>
<td>Western Queensland</td>
<td>As for Northern Territory</td>
<td></td>
</tr>
</tbody>
</table>

Table A.2: Design Rainfall Loss Rates for Northern Territory

<table>
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<tr>
<th>Location</th>
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<th>Loss Model and Design Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 4 if temporal patterns study (Figure 3.2 in ARR 2001)</td>
<td>Initial loss – continuing loss</td>
<td>From calibration flood models against largest observed floods, all in the middle or latter part of the wet season.&lt;br&gt;Initial loss = 0 – 80 mm, median = 0 mm&lt;br&gt;Median continuing loss = 1.4 mm/hr</td>
</tr>
<tr>
<td>Central Australia</td>
<td>As for Arid Zone in South Australia</td>
<td></td>
</tr>
</tbody>
</table>

Table A.3: Design Rainfall Loss Rates for New South Wales

<table>
<thead>
<tr>
<th>Location</th>
<th>Loss Model</th>
<th>Median Value of Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>East of western slopes</td>
<td>Initial loss – continuing loss</td>
<td>Initial loss = 10 – 35 mm, varying with catchment size and mean annual rainfall.&lt;br&gt;Continuing loss = 2.5 mm/hr</td>
</tr>
<tr>
<td>Arid Zone, mean annual rainfall &lt;= 300 mm</td>
<td>Initial loss – continuing loss</td>
<td>Initial loss = 15 mm&lt;br&gt;Continuing loss = 4.0 mm/hr</td>
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</table>
### Table A.4: Design Rainfall Loss Rates for South Australia

<table>
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<tr>
<th>Location</th>
<th>Loss Model</th>
<th>Loss Model and Design Parameters</th>
</tr>
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</table>
| Humid Zone (Mediterranean) | Initial loss – continuing loss | Winter
|                         |                     | Median initial loss = 10 mm                       |
|                         |                     | Median continuing loss = 2.5 mm/hr                |
|                         |                     | Summer                                           |
|                         |                     | Median initial loss = 25 mm                       |
|                         |                     | Median continuing loss = 4.0 mm/hr                |
|                         |                     | Median initial loss = 30 mm                       |
|                         |                     | Median continuing loss = 1.0 mm/hr                |
| Arid Zone               | Initial loss – continuing loss | Median initial loss = 15 mm                       |
|                         |                     | Median continuing loss = 4.0 mm/hr                |
|                         |                     | Initial loss = 15 - 40 mm                         |
|                         |                     | Continuing loss = 1.0 – 3.0 mm/hr                 |

### Table A.5: Design Rainfall Loss Rates for Victoria

<table>
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<tr>
<th>Location</th>
<th>Loss Model</th>
<th>Loss Model and Design Parameters</th>
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<tr>
<td>South and east of the Great</td>
<td>Initial loss –</td>
<td>Median continuing loss = 2.5 mm/hr</td>
</tr>
<tr>
<td>Dividing Range</td>
<td>continuing loss</td>
<td>Initial loss = 25 – 35 mm</td>
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<tr>
<td></td>
<td></td>
<td>Initial loss = 15 – 20 mm</td>
</tr>
<tr>
<td>North and West of the Great</td>
<td></td>
<td>Probably as for similar areas of NSW</td>
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<tr>
<td>Dividing Range</td>
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Appendix B Build-up Wash-off Models Parameters for HTCC
Table B.1: Detailed Estimation Results for Parameter TN for Hotham Creek Catchment (Observed date 15 January, 2004)

<table>
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<tr>
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<th>Wash-off parameters</th>
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<th>Simulated (mg/l)</th>
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<td>PEV</td>
<td>Unit</td>
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<tr>
<td>$C_1$</td>
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<td>400</td>
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Table B.2: Detailed Estimation Results for Parameter TN for Hotham Creek Catchment (Observed date 13 February, 2007)

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Table B.3: Detailed Estimation Results for Parameter TN for Hotham Creek Catchment (Observed date 8th July, 2008)

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<th>Unit</th>
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Table B.4: Detailed Estimation Results for Parameter TP for Hotham Creek Catchment (Observed date 15 January, 2004)

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Table B.5: Detailed Estimation Results for Parameter TP for Hotham Creek Catchment (Observed date 13 February, 2007)

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Saturation

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Table B.6: Detailed Estimation Results for Parameter TP for Hotham Creek Catchment (Observed date 8th July, 2008)

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Appendix C Build-up Wash-off Models Parameters for SWCC
Table C.1: Detailed Estimation Results for Parameter SS for Saltwater Creek Catchment (Observed date 6th December, 2004)

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Table C.4: Detailed Estimation Results for Parameter TP for Saltwater Creek Catchment (Observed date 6th December, 2004)

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<td>Unit</td>
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Table C.5: Detailed Estimation Results for Parameter TP for Saltwater Creek Catchment (Observed date 3rd May, 2005)

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<tr>
<td><strong>Saturation</strong></td>
<td>$C_1$</td>
<td>250</td>
<td>400</td>
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<td>$E_2$</td>
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Publications from this Thesis

Refereed Journal Papers


Refereed National and International Conference Papers

- **Hossain, I.** and Imteaz, M.A. (2011). Continuous simulation of suspended sediment transport along a stream section. Proceedings of the 34th IAHR World Congress, Brisbane, Australia, 26\textsuperscript{th} June to 1\textsuperscript{st} July.
