PARAMETER OPTIMISATION OF RIVER WATER QUALITY MODELS USING GENETIC ALGORITHMS

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ABSTRACT

Rivers can provide valuable supply of drinking water for humans, irrigation water to farmlands, water for hydropower and home for many aquatic ecosystems. However, due to population increase and its adverse effects on the rivers, inappropriate farming activities in the river catchments, and other similar adverse activities, the water quality in rivers has generally declined. Therefore, appropriate river water quality management strategies aimed at controlling and improving water quality should be seriously considered. At least, these strategies should not reduce further degradation of current water quality in rivers.

To manage river water quality in the most effective and efficient way, the cause and effect relationships of the river system must first be investigated. One example of this cause and effect relationship is the inappropriate setting of effluent license limits for sewage treatment plants (STPs). Setting low effluent license limits causes poor river water quality with high concentration of nutrients in rivers. Water quality simulation modelling tools are extensively used in water quality management to identify these cause and effect relationships, and to simulate and study the effects of various 'what-if' management strategies prior to their implementation. Such a simulation model is not available to model the Yarra River, its tributaries and associated STPs, and therefore, the development of a water quality model for Yarra River is the focus of this thesis.

The Yarra River is one of major rivers in Melbourne in Victoria (Australia) and is considered as one of the most valuable assets for all Victorians. It is situated in the eastern part of Victoria and stretches 245 km from headwaters to the mouth of estuary at Port Phillip Bay. This river provides a major source of water supply for some 2.5 million Victorians, and is a major contributor to primary produce. Due to increases in population, recent landuse developments in the catchment and inappropriate application of farming chemicals, the river water quality had degraded as indicated by the increases in nutrient concentrations. This degradation of water quality in the Yarra River not only poses a threat to aquatic ecosystems and aesthetically unpleasing, but also causes

undesirable algal blooms in Port Phillip Bay, which receives flow with high nutrients from the Yarra River and other adjacent rivers.

Several management strategies were considered by the Environment Protection Authority of Victoria (EPAVIC), which were used to improve the Yarra River water quality, in particular to reduce the nutrient level. Of these strategies, the effluent license limits on STPs, have attracted the most attention over the past 10 years, perhaps because the effluent discharges can easily be monitored and controlled, as they are point sources. Furthermore, EPAVIC has planned a further stringent effluent license limit on STPs in Year 2004. Setting of effluent license limits on STPs by EPAVIC has been solely based on the Best Available Technology (BAT) on wastewater treatment. Although EPAVIC has claimed that the overall water quality has been improved with the current effluent license limit upgrade, this does not guarantee further significant water quality improvement with another stringent effluent license limit upgrade. This is because there is a limit to the improvement of river water quality due to improvement in effluent license limits, because the other pollutant sources become dominate then. Therefore, it is necessary to assess the level of water quality improvement in the Yarra River with different effluent license limits. This requires the use of a well-calibrated Yarra River Water Quality Model (YRWQM). Due to the concern of high nutrient concentrations in the Yarra River, nitrogen (N) and phosphorus (P) were considered in YRWQM. In addition, dissolved oxygen (DO) was also considered, because DO is one of the major indicators of the river health.

Development of the YRWQM considered the following steps.

- Data collection
- Selection of the appropriate software and assembly of the model
- Pre-calibration uncertainty and sensitivity analysis of model parameters
- Model calibration and verification
- Post-calibration sensitivity analysis of model parameters

The successful model development relies primarily on the availability of good quality data. The type of data available also dictates the selection of the appropriate modelling tool for development of YRWQM. The standard river water quality software – QUAL2E was selected for the development of YRWQM, since the available data on STP effluent and river water quality can be considered as steady state data, at best. Furthermore, QUAL2E can be used to analyse 'what-if' management scenarios, in particular in relation to STP effluent license limits. An extensive data analysis was undertaken to extract and transform valuable information to assemble YRWQM, which included hydraulics, effluent characteristics and water quality data. The data analysis provided the required flow events for calibration and verification of YRWQM. The ranges for decay rates, which are responsible for degradation of river water quality, were also determined for Yarra River through first order reaction equations for use in the model calibration.

The model calibration is considered as one of the most important stages of the overall model development. A prior knowledge of the effects of different model parameters to output water quality can greatly enhance the calibration process. This can be achieved by conducting an uncertainty and sensitivity analysis prior to model calibration, which allows the identification of sensitive model parameters so that the modellers can concentrate more on these parameters during calibration. The pre-calibration uncertainty and sensitivity analysis was conducted using Monte Carlo simulation (MCS), and the results showed that in general, all decay rates were insensitive to total kjeldahl nitrogen (TKN), total nitrogen (TN), total phosphorus (TP) and DO concentrations of YRWQM. This suggested that a reasonable effort can be put into calibration of model parameters (i.e. decay rates) in this study.

The model calibration yields the set of model parameters that predicts the actual river water quality condition. Parameter optimisation is preferred over the traditional trial and error manual approach due to the subjectivity and time-consuming nature of the latter approach. Furthermore, the trial and error manual approach can often miss the 'optimum' parameter set. However, the effectiveness of any optimisation method depends on the type of search method used. Recently, an optimisation called Genetic Algorithm (GA), which uses the concept of natural genetics as the search method, has proven to be successful in obtaining the 'optimum' parameter set in many water resource applications and therefore, was selected for this study. However, this method has not been extensively used for parameter optimisation of river water quality models.

In general, the efficiency of GA optimisation depends on the proper selection of its operators. These operators deal with parameter representation, population initialisation, selection of subsequent populations, and crossover and mutation rates. Although the importance of GA operators on model parameter optimisation has been studied in the past, the findings were inconclusive and no guidelines were available to select appropriate GA operators for specific applications such as optimisation of river water quality model parameters. Therefore, a comprehensive study on the importance of GA operators on model parameter optimisation was conducted using a hypothetical river network model with both insensitive and sensitive parameters. The findings were then tested with YRWQM. Based on the limited numerical experiments conducted, it was found that the GA operators were not significant in the model with sensitive parameters in reaching convergence to the actual parameter set. However, it was significant for the model with insensitive parameters. Nevertheless, due to the insensitive nature of the model, the deviation from the actual parameter set did not pose significant difference to the actual output water quality prediction. Therefore, the use of GA in optimising parameters of river water quality models can be done efficiently by selecting a robust GA operator set from the literature. Although the optimised GA operator set can guarantee the 'optimum' decay parameter set, it is necessary to consider the amount of effort required in achieving such accuracy, which does not contribute a great difference in the overall water quality prediction. Therefore, a GA operator set obtained from literature was used in the YRWQM calibration.

Eleven decay rates, which were responsible for N, P and DO were considered in YRWQM calibration. Three low flow events were used for calibration. These low flow events are particularly useful in calibration of decay rates, since these flow events do not have the effect of non-point source pollution. The parameters that affect TKN and TP (since these two are independent of each other) were first optimised, and then the additional decay rates that affect TN were optimised keeping the previously 'optimised' parameters of TKN constant. Finally, the additional decay rates that affect DO were optimised by keeping the TKN, TN and TP decay rates at their 'optimised' value.

Three sets of decay rates were obtained for the three low flow events and all these parameters were able to match water quality observations of the three low flow events at 95% significant level. Due to insignificant difference of water quality prediction obtained from the three parameter sets, the set with the lowest cumulative absolute relative error (CARE) was adopted as the single 'optimum' parameter set. The CARE index was computed from the absolute difference between the observed and the predicted water quality responses of TKN, TN, TP and DO at the six water quality monitoring stations due to three calibration events. This single 'optimum' parameter set was then verified using three independent low flow events which were not used in calibration. It was found that the water quality of three events was modelled within 95% of their observed water quality.

The post-calibration sensitivity analysis is commonly used to investigate the effect of small deviations in model parameters from the 'optimised' values on output water quality. This post-calibration sensitivity analysis can enhance the confidence in the 'optimised' model parameters in water quality predictions and in general, in the river water quality model itself. The commonly used 'one-at-a-time' sensitivity analysis method was used in this study. Each of the 'optimised' decay rates was perturbed by a certain percentage from the 'optimised' value. The results showed that even a deviation of 50% away from the 'optimised' parameter values can predict the output water quality within 95% of concentration obtained from 'optimised' parameters.

The calibrated YRWQM was then used to evaluate the current point source management strategy on STPs with different effluent license limits and to investigate the feasibility of using a seasonal effluent discharge program to improve the water quality in Yarra River. Comparison of the water quality improvement in both of these cases was based on a number of effluent license limits under different river conditions. It was found that further increase in effluent license limits on STPs does not significantly improve Yarra River water quality, and in some cases, the wastewater treatment based on the current license limits has already satisfied or very close to satisfying the water quality standard. Therefore, further increases in effluent license limits in improving Yarra River water quality is not a feasible solution. On the other hand, the seasonal effluent discharge program was found as a feasible point source management strategy for Yarra River, due to distinct wet and dry period flows that occur within the year. It was also found for the same effluent treatment level, significant difference in river water quality concentration was observed for the wet and dry period flows. Furthermore, in some instances, the water quality response from high level of effluent treatment is very similar to the water quality response produced from a lower level of effluent treatment under the wet conditions. Based on these findings, it can be said that the seasonal effluent discharge program can be used as a point source management strategy for Yarra River. This will obviously reduce the operating costs of wastewater treatment.

DECLARATION

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university or institution and, to the best of the author's knowledge and belief, contains no material previously written or published by another person except where due reference is made in the text.

[redacted]

Anne Wai Man Ng

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LIST OF ABBREVIATIONS

The following list of abbreviations is used throughout this thesis. Other abbreviations which were only used in certain sections of the chapters are defined in that section.

NH_3	Ammonia
CBOD	Carbonaeous biochemical oxygen demand
$CBOD_d$	CBOD decay rate
$CBOD_s$	CBOD settling rate
CV	Coefficient of variation
CDF	Cummulative distribution frequency
DFS	Dights Falls
DO	Dissolved oxygen
Diss-P	Dissolved phosphorus rate
Diss-Pben	Dissolved phosphorus benthos rate
EPAVIC	Environment Protection Authority of Victoria
EPAWQ	Environment Protection Authority water quality
FOEA	First order error analysis
GA	Genetic Algorithm
MCS	Monte Carlo simulations
NH_3ben	NH_3 benthos rate
$NH_{3\text{-}d}$	NH_3 decay rate
NO_3	Nitrate
NO_2	Nitrite
$NO_{2\text{-}d}$	NO_2 decay rate
NOX	NO_2+NO_3
Org-N	Organic nitrogen
Org-N	Organic nitrogen
Org-P	Organic phosphorus
Org-N_d	Org-N decay rate
Org-N_s	Org-N settling rate
Org-P_d	Org-P decay rate
Org-P_s	Org-P settling rate
PDF	Probability density functions
K2	Reaeration rate
RDR	Relative deviation ratio
SOD	Sediment oxygen demand rate
SC	Sensitivity coefficient
STPs	Sewage treatment plants
SWWQ	StreamWatch water quality
TKN	Total kjeldahl nitrogen (Org-N + NH_3)
TN	Total nitrogen
TP	Total phosphorus
UYD	Upper Yarra Dam
YRWQM	Yarra River Water Quality Model

PUBLICATIONS AND AWARDS DURING CANDIDATURE

This thesis is the result of 3 years and 9 months of research work since Feb 1998 at the School of the Built Environment of Victoria University of Technology. During this period, the following research papers were published and two awards were won related to this project.

Publications

Ng. A.W.M. and Perera, B.J.C. (2001) Importance of Genetic Algorithm Operators in River Water Quality Model Parameter Optimisation. *International Congress on Modelling and Simulation (MODSIM 2001), Canberra, Australia 10-13 Dec. 2001, Vol. 4, pp. 1943-1948.*

Perera, B.J.C. and Ng. A.W.M. River Water Quality Modelling – Parameter Uncertainty, Sensitivity and Estimation (Invited Paper). *Water Pollution 2001, Sixth International Conference on Modelling, Measuring and Prediction of Water Pollution, Rhodes, Greece, 17-19 Sep 2001, pp.187-196.*

Ng. A.W.M. and Perera, B.J.C. Uncertainty and Sensitivity Analysis of River Water Quality Model Parameters. *First International Conference on Water Resources Management, Halkidiki, Greece,* 24-26 Sep. 2001, *pp.175-184.*

Ng, A.W.M. and Perera, B.J.C. (2000) Data Analysis and Preliminary Estimates of Parameters for River Water Quality Model Development, *Proceedings of the 26^th^ Hydrology and Water Resources Symposium, Perth, Australia, pp. 773-778.*

Ng, A.W.M., Takyi, A. and Perera, B.J.C. (1999) Evaluation of Water Quality Management Policies for the Yarra River Basin, *Proceedings of the 25^th^ Hydrology and Water Resources Symposium, Brisbane, Australia, pp. 864-869.*

Awards

Runner-up Award for the best paper presentation at *the 27th Hydrology and Water Resources Symposium, Perth, Australia (2000).* The paper presented was on "Data Analysis and Preliminary Estimates of Parameters for River Water Quality Model Development".

First prize winner for the best paper presentation at the *26th Hydrology and Water Resources Symposium, Brisbane, Australia (1999).* The paper presented was on "Evaluation of Water Quality Management Policies for the Yarra River Basin".

CHAPTER 1

INTRODUCTION

1.1 Background

The Yarra River is one of the major rivers in Melbourne in Victoria, Australia, and is considered as one of the valuable assets for all Victorians (EPA Victoria, 1999). The Yarra River catchment has a catchment area of over 4,000 square kilometres. The forested area of the catchment provides the highest quality drinking water for over 1.5 million Victorians, while the rural area supports a thrive of agricultural industries. The urban area is becoming the focus for tourist and recreational activities.

Increase in population have resulted in extensive landuse activities in the catchment, which have been blamed for the unhealthy water quality in the Yarra River and its tributaries. This degradation of river water quality has also caused a loss of crucial habitat for indigenous species. The Environment Protection Authority of Victoria (EPAVIC) recently identified five major environmental threats or activities in the catchment, which were responsible for the degradation of river water quality (EPA Victoria, 1999). They are effluent discharge from sewage treatment plants (STPs), urban and rural stormwater runoff, modified flow regime, waterway erosion, and losses from sewerage system and unsewered areas. These threats, not only caused problems in the river itself, but also posed problems in Port Phillip Bay, which is also a valuable asset for all Victorians. A recent study also showed that the high level of nitrogen (N) and phosphorus (P) carried from Yarra River (and other rivers, creeks and drains) into the Bay has caused excessive algal growth (Harris *et al.*, 1996).

To improve water quality in Yarra River and ultimately reduce the threat of algal blooms in Port Phillip Bay, appropriate implementation of management strategies is required. As such, the EPAVIC launched a series of management strategies to improve the water quality in Yarra River. The effluent discharge from STPs has attracted the most attention over the other pollution threats during the last 10 years, perhaps because it can be easily monitored and controlled, as it is a point source. The EPAVIC has progressively increased the effluent license limits of STP discharges into Yarra River and its tributaries over the past 10 years. In Year 2004, a further upgrade of the effluent license limits is planned so that the effluent discharge should cause no difference in the water quality upstream and downstream of the point of discharge.

The setting of the effluent license limits on STPs by EPAVIC has been done solely on the Best Available Technology (BAT) of wastewater treatment at the time, without conducting detailed studies on the effect of these effluent license limits on river water quality. As reported in EPA Victoria (1999), the upgrade of STP effluent license limits in 1997 has improved river water quality. However, there is no guarantee that further stringent effluent license limits can significantly improve water quality in Yarra River. This is because there are limits to improvement of water quality through management of STP effluent discharges. Furthermore, the other pollutant sources in the catchment generated from river itself (i.e. resuspension of enriched nutrient sediments that were trapped under low flows) may become dominant. However, these issues have not been studied. Therefore, the improvement in water quality due to different effluent license limits should be studied prior to their implementation and preferably studied in relation to the costs (both capital and operational costs) associated with extra wastewater treatment. The effect of the effluent license limits can be assessed through well-calibrated river water quality models. The development of a well-calibrated river water quality model for Yarra River and its tributaries including STPs is the subject of this thesis.

1.2 River Water Quality Model Development

River water quality models are commonly used to study the responses of rivers due to different management strategies, which are designed to improve water quality or at least to manage the water quality without further degradation. These models mimic the response of the river system to various inputs (i.e. flow in the river, STPs effluent discharges, other pollutant sources, etc.) using mathematical relationships describing various physical, biological and chemical processes.

In general, several essential stages are considered in the overall model development. They are stated in Beck (1983) and are listed below with some modifications done by the candidate.

- Data collection
- Selection of the appropriate water quality modelling software
- Assembly of the model
- Calibration and verification of the model
- Uncertainty and sensitivity analysis (pre- and post-calibration) of the model parameters

Collection of accurate and reliable data is the most important stage of overall model development. If the data used in the model contain errors, then the model is not accurate. The type of available data primarily governs the selection of the water quality modelling software. For example, if time-varying data are available, then an unsteady water quality model can be used. The assembly of the model involves data collection, analysis and then enter appropriate data into the selected computer modelling software tool. The model calibration adjusts the model parameters so that the model predictions match with the observations, and is considered also as a major step in the overall model development process. The verification process investigates the performance of the calibrated model parameters under independent events, which were not used in the calibration. The uncertainty and sensitivity analysis identifies the effect of input model parameters on the output water quality response. The model calibration and the uncertainty and sensitivity analysis of model parameters are discussed in little more detail in the next few paragraphs, since they were considered in detail in this thesis.

The model calibration (or parameter estimation) yields a set of model parameters, which matches the modelled water quality with the observations. Model calibration is required because often the model parameters cannot be physically measured. It can be performed using manual and automatic methods. Traditionally, the model calibration was conducted manually using trial and error curve fitting procedures, where the model response is compared with observations visually. This method is subjective, error-prone and inefficient, and can often miss the 'optimum' parameter set. Recently, the automatic methods have gained prominence because they can overcome some of the above shortcomings. Automatic methods can be categorised into two groups namely, deterministic and stochastic. Deterministic (or local) methods can be used when an unique solution exists for the optimum parameter set (i.e. the response surface has a single peak). On the other hand, stochastic (or global) methods are designed for situations when there are multiple peaks in the response surface from which the global optimum solution can be obtained.

There are many global automatic calibration methods available and they differ from each other in the way the searches are performed. The effectiveness of the search towards the 'optimum' parameter set depends upon the selected search algorithm. Genetic Algorithm (GA) is a global optimisation technique which has gained popularity in the recent past in many different fields, and is used in this thesis to calibrate model parameters of the Yarra River Water Quality Model (YRWQM). GA is based on the concept of natural selection and genetics (Goldberg, 1989). GA was first proposed by Holland (1975) and has since been recognised in providing a robust search in complex, noisy and discontinuous search spaces across wide disciplines (Grefenstette, 1986 and Goldberg, 1989).

In general, the efficiency of GA depends on the proper selection of GA operators, which are essentially the components that make up the overall GA process. Although there had been a number of studies conducted on the importance of the GA operators on 'optimum' model parameter set (e.g. Franchini and Galeati, 1997), the results were inconclusive. Therefore, a detailed study was conducted in this thesis on the importance of GA operators on 'optimum' model parameters of river water quality models, with particular attention given to YRWQM.

Errors are unavoidable in any mathematical models, because of simplifications, approximations and assumptions used. These errors can contribute to unexpectedly poor results. This is also the case with river water quality models. Therefore, it is necessary to quantify these errors in the model both in terms of input parameters and their effect on output responses. Many types of errors exist in a model, but only the model parameter error was considered in this thesis.

The parameter uncertainty and sensitivity analysis can be performed in two different stages in the overall model development, initially during the pre-calibration stage and later during the post-calibration stage. The pre-calibration uncertainty and sensitivity analysis can be used to identify the sensitive and insensitive model parameters, prior to model calibration. Then, it is possible to put more effort into the calibration of sensitive parameters during the calibration phase of the model development. This analysis has not been commonly considered in the past, but is a very useful analysis to be undertaken prior to calibration. On the other hand, the post-calibration sensitivity analysis is commonly used to investigate the effect of changes in the parameters deviate from the 'optimised' values, on output results. The main purpose of this sensitivity analysis is to enhance the confidence in the calibrated model so that the decision-makers can use the model confidently for assessing water quality management strategies.

1.3 Significance of the Research

Water quality of Yarra River has declined in recent years due to many factors caused by human activities. However, it has been realised that the water quality in Yarra River can be improved by good management practices. These management practices can be assessed and evaluated using water quality modelling tools. The success in the use of these modelling tools depends on how well the models are calibrated. To date, a well-calibrated water quality model has not been developed for Yarra River, which can be used to evaluate management strategies. Such a model will be developed in this study, which can be used by decision-makers to assess various water quality management scenarios. Two other significant issues were considered in this thesis, one related to model calibration and the other related to uncertainty and sensitivity analysis of model parameters.

A recent 'stochastic' search technique called genetic algorithm (GA) was used in this study to calibrate the YRWQM. Although GA has been successfully used in many different fields, there was only one application reported in the literature in optimising model parameters of river water quality model. Therefore, the research conducted in this thesis on the use of GA for calibration of river water quality models will add significant knowledge to this area. Furthermore, the detailed study conducted on the importance of GA operators in achieving the 'optimum' parameter set will provide some guidance for modellers to use appropriate GA operators for river water quality model parameter optimisation. Such guidelines were not available, although it was known that the efficiency of GA depends on the proper selection of GA operators.

Conducting an uncertainty and sensitivity analysis as part of the model development has been a common practice, especially at the post-calibration stage to enhance confidence in management solutions. However, conducting such analysis prior to calibration has not been commonly used. In this study, a pre-calibration uncertainty and sensitivity analysis was conducted to investigate the advantages from such analysis before calibration.

1.4 Aims of the Study

The main aim of this research project was to develop a well-calibrated water quality model for Yarra River (which is known as YRWQM in this thesis) using the GA optimisation technique. The following specific tasks were undertaken to achieve this main aim.

(i) Collect and analyse available data on the Yarra River catchment.
(ii) Select the most appropriate water quality software to develop YRWQM
(iii) Assemble the YRWQM.
(iv) Select suitable low flow events for use in calibration and verification of YRWQM.
(v) Link YRWQM with the GA optimisation software – GENESIS through input and output files.
(vi) Conduct a pre-calibration uncertainty and sensitivity analysis of YRWQM model parameters.
(vii) Investigate the significance of GA operators in achieving the 'optimum' model parameter set of river water quality models, in particular YRWQM.
(viii) Calibrate YRWQM using GA.
(ix) Verify YRWQM.

(x) Conduct a post-calibration sensitivity analysis of YRWQM model parameters.

(xi) Investigate the effect of different management strategies on Yarra River water quality.

The scope of the thesis was limited to consideration and modelling of

- Steady low flow conditions.
- Point source pollutants.
- Nonconservative water quality constituents of DO, carbonaceous biochemical oxygen demand (CBOD), and all forms of N and P.

Steady low flow condition was considered in this study because this type of flow regime could provide the critical water quality concentrations under various design conditions, which provide conservative output result. Furthermore, unsteady flow and water quality data were not available at the time of study. Due to data and time constraints, only modelling of point source pollutants is explicitly considered in this study, although nonpoint source was considered implicitly. Only nonconservative water quality constituents as listed above were considered in this study because high nutrient concentration was the major concern in the Yarra River. Modelling these nonconservative water quality constitutes sufficiently indicate the overall river health status of the Yarra River.

1.5 Outline of the Thesis

Chapter 2 reviewed the literature on various aspects and methods, which are required to develop river water quality models. The type of river processes and interactions that exist between the water quality constituents were discussed first. Different methods that can be used for calibration of mathematical models and uncertainty and sensitivity analysis of model parameters were reviewed then. Comparison of various river water quality software tools and the selection of the most appropriate water quality software that can be used for the Yarra River catchment were also discussed in this chapter.

Chapter 3 provided background information on the Yarra River catchment. Availability of various data for use in the model development was summarised. The selection of flow events for use in YRWQM model development was also discussed in this chapter.

Chapter 4 described the assembly of YRWQM using QUAL2E software. The estimation of the ranges for decay rates of water quality constituents for use in YRWQM calibration was discussed in this chapter, as well as the identification of appropriate reaeration rate estimation method for Yarra River.

The pre-calibration uncertainty and sensitivity analysis of YRWQM parameters was conducted in Chapter 5.

Three main components related to parameter optimisation of YRWQM were presented in Chapter 6. First, the study of the importance of GA operators in achieving the 'optimum' parameter set was discussed. Then, the GA optimisation of YRWQM was presented together with the verification of the model. Finally, the results of the post-calibration sensitivity analysis of model parameters were presented.

The results of the investigations of point source management strategies were presented in Chapter 7. Two investigations were discussed. The first investigation dealt with different effluent license limits on STPs and their effect on Yarra River water quality. The second investigation discusses the applicability of seasonal effluent discharge programs for Yarra River catchment STPs and its effect on water quality.

A summary of the conclusions drawn from the study described in this thesis was presented in Chapter 8. Recommendations for future research work, arising from this research were also outlined in this chapter.

CHAPTER 2

RIVER WATER QUALITY PROCESSES AND MODELLING

2.1 Introduction

Management of river water quality has become an important issue recently, due to decline in water quality caused by many factors such as population increase and inappropriate application of farming chemicals. To be able to design and implement appropriate management strategies for river water quality improvement, development of river water quality models is essential. These water quality models enable the decision-makers to study the effect of various management strategies and to implement policy decisions.

A vast amount of general-purpose water quality computer software is available in the public domain, to develop catchment and river water quality models for specific river settings. These water quality modelling software can be categorised into three main groups: catchment, river and integrated. Catchment water quality software is mainly used to determine overland runoff from urban and rural areas. River water quality software, on the other hand, models river response due to the pollutants carried from overland flows and discharges from sewage treatment plants (STPs). The integrated catchment and river water quality software determines pollutant loads from the catchment and STPs within the catchment, simulates the river water quality response due to these pollutants and implements management strategies to improve river water quality on a basin wide scale. These software are also known as Decision Support Systems (DSSs).

The above water quality software can be used to develop river/catchment water quality models. Model calibration (or parameter estimation) is an important component in the overall model development. Traditionally, the model calibration was conducted by manual trial and error procedures, where the predictions resulting from model parameters were compared with observations. This method is subjective, error-prone and often by-pass the optimum parameter set (Sorooshian and Gupta, 1995). However, the automatic search methods (or optimisation methods) which have been introduced recently for model calibration, have overcome some of these problems. In addition, the automatic search methods have demonstrated to be the most appropriate for use in water resource applications (Duan *et al.,* 1992). Genetic algorithm (GA) is one of those parameter optimisation techniques, which has been applied recently, on a wide range of applications. GA is an optimisation technique, which utilises the concept of human genetics. In recent times, this method has also been applied to many water resource applications, and has been proven to be an efficient search technique for model parameter optimisation (Wang, 1991).

Uncertainties in model simulation have contributed to unexpectedly poor results of some stream water pollution control plans (Melching and Chang, 1996). Although there are a number of sources that contribute to model errors, the parameter estimation is the only source that can be controlled by the model user. Many methods are available which can be used to identify and quantify the effect of uncertainty and sensitivity of the input parameters to output responses.

This literature review expands on the above issues. It begins by describing different processes and interactions of water quality constituents in the river water column. The available water quality software in the public domain are then reviewed in the light of selecting the most appropriate river water quality software for this study (i.e. for Yarra River). Different methods that can be used to calibrate river water quality models are then discussed. Finally, the current methods used to determine parameter uncertainty and sensitivity are reviewed.

2.2 River Water Quality Processes

River water quality depends on the assimilative capacity of the river, which is a measure of ability to digest pollutants entering the river. This assimilative capacity is controlled by 3 processes namely physical, biological and chemical (Schnoor, 1996).

2.2.1 Physical processes

The physical processes reduce organic and inorganic pollutants through dilution, sedimentation, resuspension, adsorption, volatilsation and photolysis (Chapman and Kimstach, 1992). However, these processes do not consume oxygen in reducing organic and inorganic pollutants in the river. The factors that control the amount of degradation of pollution through dilution, sedimentation, resuspension and adsorption are mainly river flow and velocity (Dojlido and Best, 1993). Dilution is a process where the water quality pollutants are reduced through the increase in 'clean' water from tributaries and other sources. Dilution is higher during high flows, although there is a possibility of high nutrient runoff being introduced during this period. Sedimentation is a process where pollutant particles such as suspended solids in the water column settles to the river bottom during low velocity periods. These settled organic matters are subject to resuspension when velocity increases, which act as another pollutant source in the water column. Adsorption is a process where the organic matters are attached to the soil particles, and eventually removed from the water column and settled in the river bottom.

Volatilisation mainly deals with the pollutant transfer between water and air (Thomann and Mueller, 1987 and Schnoor, 1996), which does not result in the breakdown of a substance. Photolysis is a degradation process, where radiant and light energy is used to break the pollutant molecule (Thomann and Mueller, 1987). Both of these processes are mainly applicable to conservative water quality constituents.

2.2.2 Biochemical processes

The biological and chemical processes are often combined as biochemical processes (Courchaine, 1968). The biochemical processes reduce or transform pollutant matter by plants and microorganisms through consumption of oxygen (Dojlido and Best, 1993). The degradation of organic matter through biochemical processes involves mineralisation and microbially decaying to reduce one form of water quality constituent to another. Mineralisation is the microbial conversion of one form of water quality constituent to another through decomposition. Microbial decaying involves bacterial oxidation of water quality constituents. An example of this is the nitrification, where ammonia (NH_3) oxidises to nitrite (NO_2) and in turn to nitrate (NO_3). Not all biochemical processes require the presence of oxygen, for example denitrification. Denitrification is the microbiological reduction of NO_3 to NO_2, which in turn can be reduced to nitrogen gas (N_2), without the presence of dissolved oxygen.

There are many factors which effect the rate of biochemical process, including microorganism population, dissolved oxygen (DO) content, water temperature and pH level (Bowie *et al.*, 1985 and Dojlido and Best, 1993). The biochemical process normally occurs in a cycle, for example nitrogen (N) and phosphorus (P). A detail explanation of these two cycles are discussed in Section 2.3.

Both physical and biochemical processes of nonconservative water quality constituents are considered in this thesis.

2.3 Interactions of Water Quality Constituents

The oxygen level in rivers is a crucial factor in maintaining the health of the ecosystem. Many activities in the river catchment can cause the generation of different water quality pollutants, which can reduce the level of DO concentration in the river. Water quality processes, which reduce the level of oxygen level, is termed 'sinks'. On the other hand, water quality processes, which can increase the oxygen level in rivers, is termed 'sources'. To accurately determine the overall oxygen balance in the river, it is necessary to identify these 'sources' and 'sinks' and their interactions that have effects on oxygen. However, quantifying 'sources' and 'sinks' is extremely difficult, since there are diverse activities present in a river system (Chapra, 1998), and requires extensive data and sufficient time for analysis. In this review, major nonconservative water quality pollutants, which have effects on DO, are considered first, followed by a detailed discussion on nitrogen and phosphorus cycles.

(a) DO

Streeter and Phelps (1925) were the first to study the BOD-DO relationship in a river in which sinks of oxygen were caused by biodegradable waste, and the sources of oxygen were from the atmosphere. Since then, many other researchers such as Camp (1963), Dobbins (1964), and Frankel and Hansen (1968) modified and extended the Streeter and Phelps relationship by incorporating additional processes to enhance the accuracy of oxygen balance simulations (Gromiec *et al.*, 1981).

The possible nonconservative 'sources' and 'sinks' that contribute to the DO balance are shown in Figure 2.1, which has been modified from Brown and Barnwell (1987). The 'sources' and 'sinks' shown in this figure represent primary effects on DO only, and the secondary effects are not considered in this review.

Primary effect of 'sources' is that they have direct positive (i.e. increase) influence on DO. For example, the sources of DO in Figure 2.1 include reaeration and photosynthesis of plants (e.g. phytoplankton), shown with an inward pointing arrow to DO. The secondary effects are the effects on the primary 'sources' and not directly on DO. For example, the secondary effects such as the contribution of wind, rain and hydraulic structures could effect the reaeration rate, which in turn effect the DO concentration.

Similarly, the primary effect of 'sinks' is that they have direct negative (i.e. decrease) influence on DO, which include carbonceous biochemical oxygen demand (CBOD),

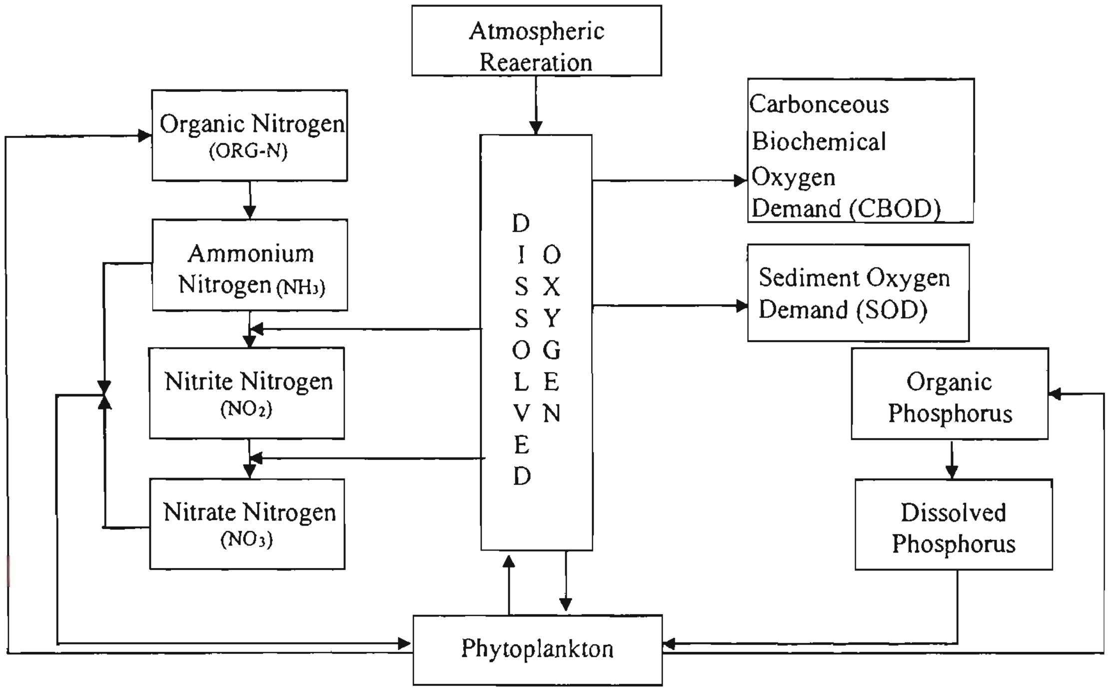


Figure 2.1 Sources and Sinks of DO (Modified from Brown and Barnwell, 1987)

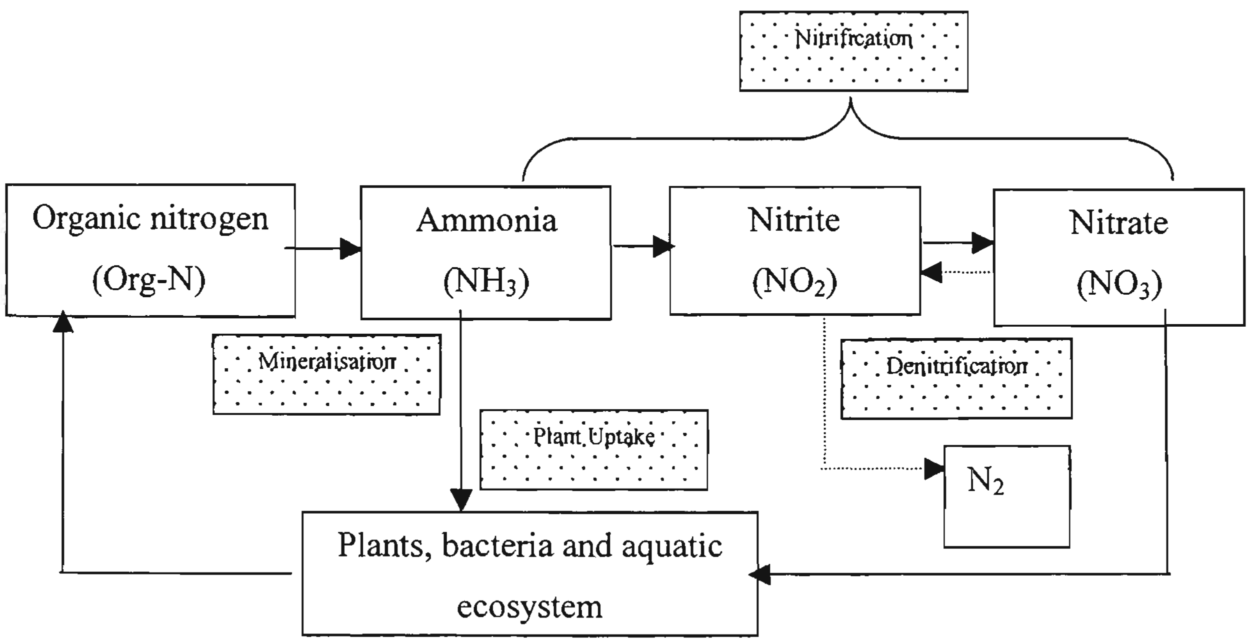


Figure 2.2 Nitrogen Cycle

sediment oxygen demand (SOD) including suspended and benthic, respiration by plants and all N and P forms, as indicated with an outward pointing arrow from DO in Figure 2.1. One example of the secondary effect of 'sinks' is the influence of velocity on SOD.

Figure 2.1 also shows the interactions within N and P forms, and their interactions with phytoplankton and DO.

(b) Nitrogen cycle

The nitrogen cycle consists of microbial transformations from one form of nitrogen to another and interactions of different forms of nitrogen within the cycle. Figure 2.2 shows the nitrogen cycle. The shaded boxes shows the processes associated with various forms of N. The nitrogen forms considered in this study are organic nitrogen (Org-N), NH_3, NO_2, NO_3. The sum of Org-N and NH_3 is called total kjeldahl nitrogen (TKN), while the sum of all four forms of N is called total nitrogen (TN).

The formation of Org-N is principally through the food chain within the water body. Death of plants and aquatic organisms produce Org-N. With time, the Org-N is mineralised to NH_3. Mineralisation is the microbial transformation from Org-N to NH_3. NH_3 also occurs naturally in water bodies from excretion by aquatic ecosystems, from effluent discharges from STPs and also from runoffs from agricultural and urban lands. When the pH in water is too high, unionised NH_3 can become toxic to fish (Bowie *et al.*, 1985 and Chapra, 1998). NH_3 may be adsorbed onto suspended particles (not as strongly as phosphorus) and bed sediments during low flows, and these particles would regenerate in the water column during high flows (Goering, 1972).

The reduction of NH_3 is via two major processes: nitrification and uptake by aquatic plants. Nitrification involves the bacterial oxidation transformation of NH_3 to NO_2, and then to NO_3. The concentration of NH_3 can fluctuate greatly between seasons (Bowie *et al.*, 1985 and Dojlido and Best, 1993). This variation is due to greater microorganism growth in summer than in winter, which causes higher uptake of aquatic plants in summer. Furthermore, the rate of nitrification is temperature dependent, therefore, causing faster transformation with higher temperature. The product of nitrification is NO_2, but this form is unstable under aerobic conditions and hence it would rapidly oxidised to NO_3 (Bowie *et al.*, 1985). If condition becomes anaerobic, NO_3 can partially undergo a process called denitrification and reduces back to NO_2, and then further reduced to N_2, which vaporises into the atmosphere. Untreated or inadequately treated STP effluent can result in high levels of NH_3. Runoff from excess application of farming chemicals, death of aquatic ecosystems and debris from plants are all sources of NO_3.

(c) Phosphorus cycle

Phosphorus is another essential nutrient for growth of aquatic plants and other microorganisms (Dojlido and Best, 1993). Phosphorus cycle is very much similar to the nitrogen cycle, but is less complex. Phosphorus can be found in the river in two main forms: organic and dissolved inorganic phosphorus. The source of organic phosphorus (Org-P) is mainly from the death of plants and aquatic ecosystems. As Org-P is generally not in a bioavailable form, it would require undergoing transformation to dissolved inorganic phosphorus (Diss-P) (Reddy *et al.*, 1999). This form is more readily available for aquatic plant uptake (Thomann and Mueller, 1987). The rate of breakdown of Org-P to Diss-P is depended upon the water temperature, the composition and the bacteria population (Dojlido and Best, 1993). The phosphorus cycle is shown in Figure 2.3. Total phosphorus (TP) is given by the sum of Org-P and Diss-P.

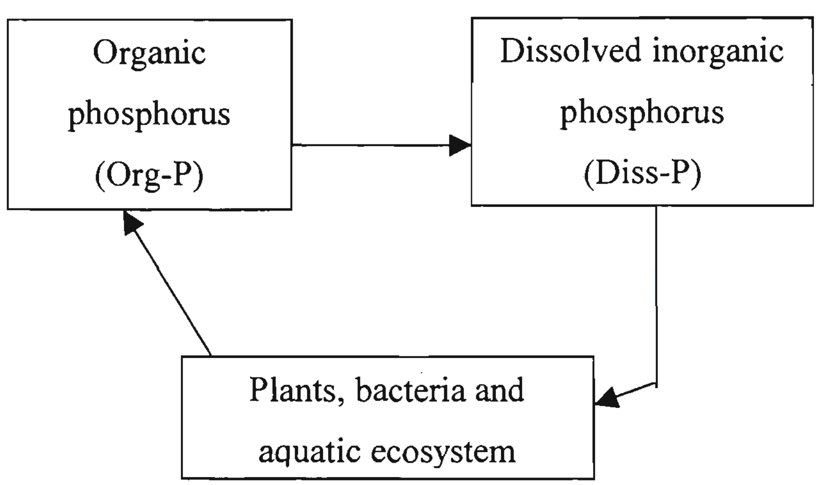


Figure 2.3 Phosphorus Cycle

Phosphorus presents in water mainly through sewage effluent discharge, soil erosion, weathering and leaching phosphorus-bearing rocks, and runoff from agricultural and urban areas (Bowie *et al.*, 1985 and Dojlido and Best, 1993). Phosphorus removal from the river is very much similar to the nitrogen removal but without the complex nitrification and denitrification processes. Another major difference is that soil particles adsorb strongly onto phosphorus. These particles then settle during low flows and would retain in the river bed which reduce phosphorus in the water column. As Reddy *et al.* (1999) stated, reducing phosphorus through particle adsorption is much higher during low summer flows than during high winter flows. Once the phosphorus settles in the river bottom, it is subject to resuspension to release phosphorus back into the water column during high flows. However, studies have indicated the effect of the release of phosphorus back to the water column is insignificant over short time scales (Reddy *et al.*, 1999). When the oxygen content is anaerobic, the return of phosphorus back to the water column via resuspension is three times greater as in aerobic condition. The greater the temperature and velocity in the river water, the greater the exchange rate between sediment and water column (Dojlido and Best, 1993).

2.4 River Water Quality Modelling Software

River water quality modelling software can be used to model the actual river system. Many water quality modelling software tools are available and the applicability of these software tools depends on the study objectives. Therefore, it is necessary to review available water quality software modelling tools, so that the most appropriate software tool can be used for the study in this thesis. This review was limited to the public domain software. In general, the water quality modelling software can be categorised into three broad groups, namely catchment, river and integrated software. Under these respective groups, the available public domain software are shown in Figure 2.4.

2.4.1 Catchment water quality modelling software

The catchment water quality modelling software tools are used to estimate the amount of pollutant loadings generated from different land surfaces in catchments, which affects

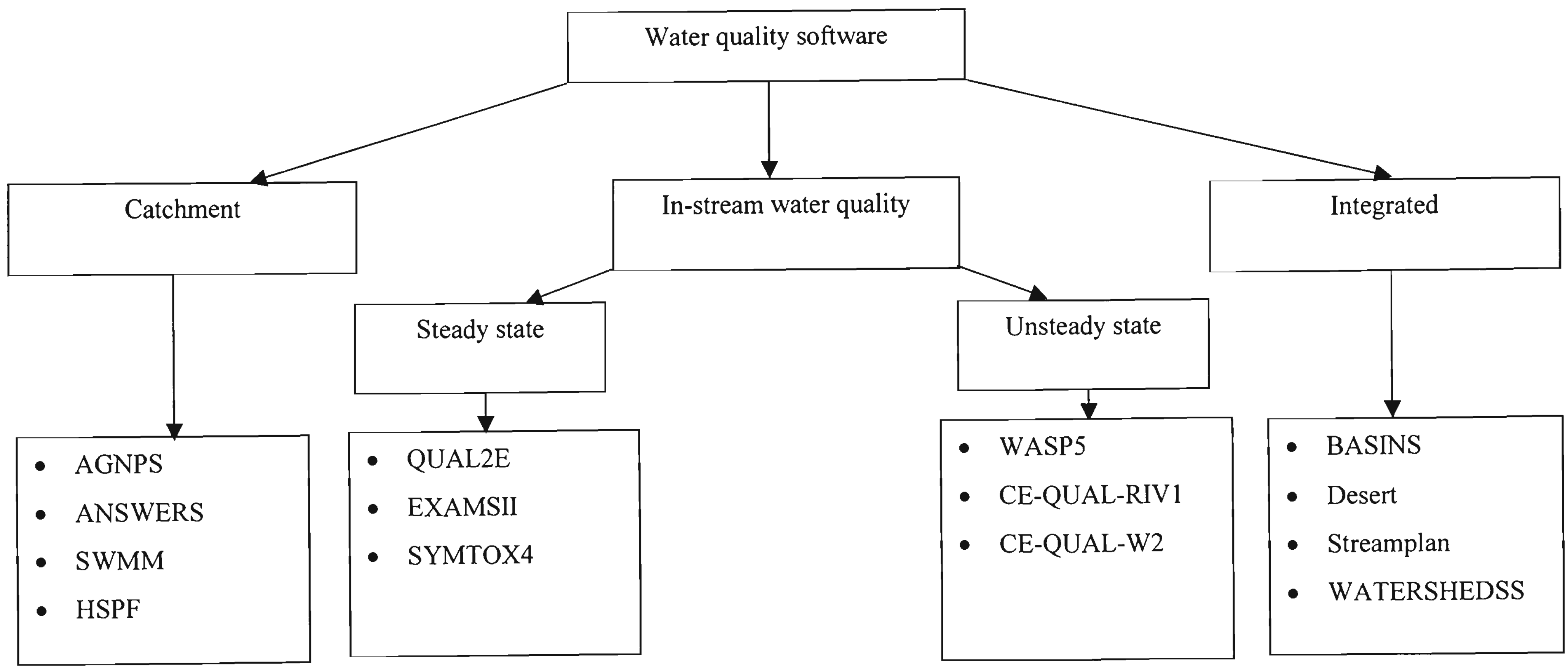


Figure 2.4 Summary of Public Domain Water Quality Software

the water quality in streams and rivers. In Figure 2.4, only a list of commonly used catchment water quality modelling software are shown.

The most commonly used catchment water quality modelling software is the Agricultural Nonpoint Source Pollution software (AGNPS) (Young *et al.*, 1989), which was developed by the United States of Agricultural Research Services. AGNPS can be used in both event and continuos simulation modes, to generate sediment and nutrient loads from agricultural areas. The catchment is divided into a number of cells to determine pollutant loadings. Tim *et al.* (1995) used AGNPS (linked with a GIS system) to examine the effect of varying widths of vegetated buffer strips on sediment yield of the Bluegrass catchment in Iowa, USA.

Areal Nonpoint Source Watershed Environment Response Simulation Model (ANSWERS) is an event based software, which is capable of predicting both urban and agricultural pollution loads (Beasley and Huggins, 1981). The catchment area is divided into square grid elements (1-4 ha), where each element simulates interception, infiltration, surface storage, surface flow, subsurface and sediment drainage, sediment detachment, transport and deposition. The output result from upstream element is the input to the downstream element.

Storm Water Management Model (SWMM) is an urban stormwater quantity and quality software tool developed by United States Environmental Protection Agency (USEPA) (Huber *et al.*, 1982), which can be used in both event and continuos simulation modes. Data required by SWMM are relatively intensive. It has the most versatile hydrological and hydraulic simulation modules, while the water quality simulation is relatively weak in representation of the true physical, biological and chemical processes (USEPA, 1997a).

The Hydrological Simulation Program - FORTRAN (HSPF) is one of the comprehensive software tools, which simulates the catchment runoff processes together with river water quality (Bicknell *et al.*, 1993). This software tool can simulate nonpoint source runoff. Major nonconservative water quality constituents can be simulated. As commented in USEPA (1997a, 1997b), HSPF is a highly complex software tool, which requires extensive resources and data. Moore *et al.* (1992) used HSPF on the North Reelfoot Creek catchment, located in the northwest corner of Tennessee, USA to examine several best management practices in reducing erosion and sedimentation.

2.4.2 River water quality modelling software

River water modelling software tools are used to simulate the effects of pollutants generated from catchment and point sources, on river and stream water quality. This category is reviewed in detail, since river water quality models are used in this study. Two types of software tools exist in the literature, namely steady and unsteady.

2.4.2.1 Steady software tools

The steady state river water quality modelling software assumes that the magnitude of flow and pollutant entering the stream do not vary with time (Thomann and Mueller, 1987). Therefore, in these software, the average inputs of flows and pollutants are considered for the flow event, giving average values for output water quality concentrations. Although these software tools cannot assess the water quality for time varying conditions, they can be useful in determining the critical water quality concentrations under design conditions. The results obtained from steady software are always conservative than the results simulated with the unsteady software (USEPA, 1987). The steady software tools are commonly used because they are less complex, easy to use and require less input data. Below is a review of some commonly used public domain steady river water quality modelling software and their applications.

The Enhanced Stream Water Quality Model (QUAL2E) is a one-dimensional steady state river water quality simulation software, which was developed and is supported by USEPA (Brown and Barnwell, 1987). In 1995, QUAL2E was upgraded with a Windows interface to enhance its user-friendliness. Although QUAL2E is a steady state software tool, it can also account for diurnal variation in meteorological inputs for the simulation of algae and temperature. QUAL2E can model all nonconservative water quality constituents, as well as 3 other user-defined water quality constituents. QUAL2E can be applied to different waterbody types, and allows modelling of multiple waste discharges and diversions. It also includes components that allow implementation of uncertainty analysis of model parameters using first order error analysis, one-at-a-time and Monte Carlo simulation.

QUAL2E has been extensively used in many applications, and majority of applications used this software to simulate nonconservative water quality constituents in rivers. Some recent applications include the work of Ghosh and McBean (1998), Cvitanic and Kompare (1999) and Ning *et al.* (2001). In comparison, limited applications were cited using QUAL2E to simulate microbial water quality (Steynberg *et al.*, 1995).

QUAL2E was used to develop a water quality model of the Kali River in India (Ghosh and McBean, 1998). However, they experienced data limitations in their application and commented that provided adequate data are available, QUAL2E is a good tool to use on rivers that have water pollution problems.

Cvitanic and Kompare (1999) applied QUAL2E to simulate and to predict the possible changes in water quality in the River Sava in Croatia with the construction of impoundments. However, they found that QUAL2E was not suitable for their application, because the model prediction could not be validated during the validation stage. They concluded that a two-dimensional model is more suitable to predict water quality in the impoundments, since large variations of river water quality exist in the river (horizontally and vertically) during summer periods. They could have simply avoided this problem by selecting the most suitable software tool for their study, since they had a good prior knowledge of water quality variations in horizontal and vertical directions.

Ning *et al.* (2001) developed and calibrated a model for the Kao-Ping River Basin in Taiwan using QUAL2E. They successfully used the model as a simulation tool to assess the water quality standard requirements downstream by hypothetically eliminating pig farming activities and construction of sewer system in the upstream areas. Although QUAL2E has been successfully used as a simulation tool in this study, they commented that an economic instrument for controlling and reducing the wasteload allocations would be needed in the long term.

Steynberg *et al.* (1995) used QUAL2E in an effort to simulate the effect of various management strategies, which can result in the desired level of faecal coliform in the Rietspruit catchment in South Africa. It was found that the model was unsuccessful in predicting the measured faecal coliform level. They clearly identified due to the time varying discharge of effluent and different levels of wastewater treatment, the use of the steady state model QUAL2E, was inadequate in their study.

Reviewing the river water quality modelling software, Shanahan *et al.* (1998) reported that QUAL2E has become the standard modelling software tool and has shown to be most applicable in situations where point source pollutants are dominant. Therefore, QUAL2E has been integrated and linked into number of other modelling software tools. For example, QUAL2E has been integrated into decision support systems (DSSs), as in BASIN (USEPA, 1997a). Mulligan and Brown (1998) linked QUAL2E with genetic algorithm optimisation software, GENESIS (Grefenstette, 1995). The use of remotely sensed water quality data with QUAL2E (Yang *et al.*, 1999) can accurately interpret the spatial variation in water quality and to keep up-to-date with the water quality conditions in the Te-Chi Reservoir in Taiwan. De Azevedo *et al.* (2000) linked QUAL2E with a water quantity network flow allocation model MODSIM of Labadie (1992), to assess and evaluate six management alternatives for strategic river basin planning.

Exposure Analysis Modelling System (EXAMSII) (Burns, 1990) can be used in modelling of streams, rivers and reservoirs in one, two and three dimensional modes. It accounts for many water quality transformation processes, such as photolysis, hydrolysis, oxidation and sorption with sediments and biota (USEPA, 1997a). This software simulates conservative water quality constituents.

SYMTOX4, the Simplified Method Program – Variable Complexity Stream Toxics Model (USEPA, 1997a), is a one-dimensional software tool that can be used to simulate the water column and benthic toxic caused by point sources discharged into rivers. No nonconservative water quality constituents can be simulated using SYMTOX4. This software is Windows based and has the capability to perform uncertainty and sensitivity analysis. No study has been found in the literature using SYMTOX4 for a river system. However, an example using data on the Flint River, Michigan in the United States was given in USEPA (1997a).

2.4.2.2 Unsteady software tools

Unsteady (or dynamic) water quality software tools can be used to simulate water quality response in rivers whose flow and water quality characteristics change with time. All natural rivers and streams have unsteady flow characteristics, especially during high flow period, and therefore unsteady modelling software are more realistic. However, they require more data inputs compared to steady software tools. Below is a summary of the available public domain unsteady water quality software, which are also listed in Figure 2.4.

Water Quality Analysis Simulation Program (WASP5) is a well-known unsteady water quality simulation software tool supported by USEPA (Ambrose *et al.*, 1993). This software has flexible compartments such as hydrodynamics, eutrophication (DO/CBOD/ nutrients/algal/carbon) and toxins. The user can use these compartments selectively or all compartments simultaneously. This software can be used to model rivers and streams in one, two and three dimensional modes.

Lung and Larson (1995) successfully used WASP5 to predict the impact of eutrophication under steady state in the Upper Mississippi River and Lake Pepin in USA. They recognised that the unsteady mode should be used to study algal growth, however, relevant data inputs for algal growth dynamics were not available to them for unsteady flow modelling. They justified the use of steady state mode after studying that the phytoplankton population did not vary greatly from hour to hour.

Suárez *et al.* (1995) developed an unsteady state water quality model of Nalón River in Spain using WASP5. This model was used to assess the impact of combined sewer flow (with daily fluctuations) on river quality and its effect on aquatic system. Majority of the water quality related inputs required in WASP5 (e.g. reaeration rate methods, decay rates) were obtained from preliminary water quality modelling using QUAL2E. The WASP5 model was successfully calibrated, which adequately simulated the water quality and activities occurring in the river accounting for time variation. However, the decay rates were required to be the same for all reaches of the river in WASP5, which they found to be one of the main deficiencies.

An eutrophication model was developed for the Tolo Harbour in Hong Kong using WASP5 by Lee and Arega (1999). This model accounts for sediment water interaction together with time and spatial variation in water quality. They successfully predicted the DO and Chlorophyll-a concentration, and matched with observations. The model was developed to study the long term trends of eutrophication in the harbour.

As reported in World Bank (1997), WASP5 is not appropriate for basins with large catchment areas, since it is complex and time consuming to calibrate and simulate water quality conditions of rivers and streams associated with these large basins.

The Hydrodynamic and Water Quality Model for Streams (CE-QUAL-RIV1) is a one dimensional software tool developed by The U.S. Army Waterways Experiment Station (USACE, 1990). It has two separate compartments: hydrodynamics and water quality. The results obtained from the hydrodynamics compartment are used as input to the water quality compartment. Many nonconservative water quality constituents can be modelled, including the effects of algae and macrophytes. One advantage of using this model is that it allows modelling of river structures such as dams. This model is less widely used compared to QUAL2E and WASP5 (Gore and Petts, 1989). CE-QUAL-RIV1 has been applied to the Cumberland River, the Chattahoochee River, and the lower Ohio River in USA (USEPA, 1997a).

CE-QUAL-W2, also developed by The U.S. Army Waterways Experiment Station (USACE, 1990) is a two-dimensional, laterally averaged, hydrodynamic and water quality model. It contains one module, which models both hydrodynamics and water quality. It can model DO, nutrients and algae interactions. Since this software accounts for variations in longitudinal and vertical directions (not in lateral direction), it is best to use this software in situations where large variations in lateral velocities and water quality concentrations do not occur (USEPA, 1997a). Martin (1988) applied CE-QUAL-W2 to DeGray Lake in Arkansas, USA, and demonstrated the usefulness of using this 2-dimensional unsteady software.

2.4.3 Integrated water quality software

The integrated software tools consists of several stand-alone tools in one package. For example, catchment and river software tools can be integrated into one package to analyse both flow and water quality in rivers and associated catchments. When decision support is available in integrated software, they are called decision support system (DSSs). There is an increase use of DSSs in river water quality management in recent times. The purpose of a DSS is to effectively allow decision-makers to simulate the whole process of decision making, related to the particular application (i.e. improving river water quality), to investigate and simulate alternative decision management scenarios, and to improve the effectiveness of decision making. Below are descriptions of four public domain integrated water quality software (or DSSs) found in the literature.

Better Assessment Science Integrating Point and Nonpoint Sources (BASINS), developed by the USEPA Office of Water (USEPA, 1997a), consists of a catchment water quality modelling software and a river water quality modelling software (QUAL2E). The Nonpoint Source Model – NPSM is a Windows interface that works with the catchment model - HSPF (Bicknell *et al.*, 1993). The graphical system in BASINS uses Arc-View GIS software. One disadvantage of this system is that the data management module is less useful to countries other than USA, since all relevant information and data are only applicable for basins in USA, which are updated annually.

Decision Support System for Evaluating River Basin Strategies (DESERT) is a flexible, Microsoft Windows-based tool for decision support for water quality management at the river catchment scale. Desert was developed by two organisations jointly: Internatioanl Institute for Applied Systems Analysis in Austria (IIASA) and the Institute for Water and Environmental Problems in Russia (Ivanov *et al.*, 1996). This software provides a powerful instrument for developing least-cost river catchment policies, and for assessing these policies under conditions that are deviating from the design scenario (Perera and Somlyódy, 1998). Fan (1996) used DESERT to identify the most efficient water quality management strategy in terms of wastewater treatment alternatives for the Veszprémi–Séd River in Budapest, Hungary.

Streamplan (Spreadsheet Tool for River Environmental Assessment Management and Planning) was developed at IIASA in 1996 (De Marchi *et al.*, 1996). It is a DSS that allows decision makers to evaluate river and catchment water quality policies on the basis of local and regional water quality goals, effluent standards, costs, financing, economic instruments, municipal water management issues and generation of wastewater treatment plant alternatives. Streamplan is developed for use on a Microsoft Excel platform, which is familiar to most model users. Perera and Somlyódy (1998) discussed the use of Streamplan in three degraded river catchments in Central and Eastern Europe: Narew (Poland), Morava (The Czech Republic) and Nitra (Slovak Republic).

Water, Soil and Hydro-Environmental Decision Support System (WATERSHEDSS) is similar to BASINS, except WATERSHEDSS is more focused on nonpoint source pollution (Osmond *et al.*, 1997). The USEPA Office of Research developed this system in 1994, with the cooperation of North Carolina State University water quality group and the Department of Biological Engineering of Pennsylvania State University.

2.4.4 Evaluation and selection of water quality software for Yarra River

Of the reviewed software tools in Sections 2.4.1 to 2.4.3, it was necessary to identify and select the most suitable software for use in modelling of water quality in Yarra

River. The catchment water quality software tools (Section 2.4.1) mainly concern with the generation and transport of overland pollution and not directly consider river water quality. Therefore, they are not suitable for modelling Yarra River water quality, although quantifying overland pollutant runoff into streams is also an important component in modelling of river water quality. However, modelling of overland pollutant runoff is outside the scope of this study. The river water quality software tools deal with river water quality and therefore absolutely relevant to this study. Although the integrated water quality modelling software (or DSS) tools are very efficient simulation and management systems as complete decision making tools, they require extensive data, and therefore, were not considered in this study. Therefore, the river water quality software tools were further investigated for modelling Yarra River, its tributaries and associated STPs.

Three criteria, as listed below, were used in selecting the river water quality software for use in Yarra River, from the set given in Figure 2.4.

- Be able to simulate nonconservative water quality constituents such as DO, BOD and nutrients,
- Be able to produce longitudinal profiles of water quality concentrations
- Wider usage of the software

These broader criteria will certainly reduce the number of river water quality software tools that can be used for Yarra River. QUAL2E and WASP5 were the only two software tools that fitted the above criteria and therefore, can be used for development of the Yarra River water quality model. Both software can simulate nonconservative water quality constituents, can produce longitudinal water quality profiles, and has been used successfully on many applications.

As stated in USEPA (1997a) and Shanahan *et al.* (1998), both QUAL2E and WASP5 software tools are well known and credible, with extensive capabilities and wide usage. These software tools were further evaluated for modelling Yarra River. A summary of

Table 2.1 Evaluation Summary of QUAL2E and WASP5 Attributes

Fundamental		**QUAL2E**	**WASP5**
Operational Requirements	Documentation	Y	Y
	Support	Y	Y
	Credibility	Y	Y
Water body type	River, Stream	Y	Y
	Estuary	N	N
	Lake	N	Y
	Reservoir	N	Y
Dimension	1-D	Y	Y
	2-D	N	Y
	3-D	N	Y
Transport	Advection	Y	Y
	Dispersion	Y	Y
Hydrodynamics	Input	Y	Y
	Simulated	N	Y
Steady/unsteady	Steady state	Y	Y
	Unsteady	N	Y
Discretisation		Y	Y
Hydraulic Structures		Y	N
Water quality			
DO	Reaeration/ (built-in equations)	Y/Y	Y/Y
	CBOD	Y	Y
	NH_3	Y	Y
	NO_2	Y	Y
	SOD	Y	Y
	Algae	Y	Y
Nitrogen forms	Org-N	Y	Y
	NH_3	Y	Y
	NO_2	Y	N
	NO_3	Y	Y
	Algae	N	Y
Phosphorus forms	Org-P	Y	Y
	Diss–P	Y	Y
	Algae	N	Y
Temperature		Y	N
Settling / Benthos		Y/Y	Y/Y
Toxicity		N	Y
Others			
Uncertainty and sensitivity analysis		Y	N

the evaluation results is given in Table 2.1. Three main categories, namely fundamental, water quality and others, were considered in classifying the attributes of these software tools. Some of these attributes were considered in a comparative study by Ambrose *et al.* (1993). The first category consists of attributes, which forms the basic structural framework of the software. Hydrodynamics is an extremely important attribute in water quality modelling, because the movement of water can affect the fate of the water quality constituents. WASP5 has an independent compartment for simulating hydrology of the water body system, whereas QUAL2E requires hydrology (or flows) as input. Both software can be operated in a steady state environment, which is the most common water quality modelling application, although WASP5 can also be used in an unsteady state environment. One-dimensional longitudinal process can be modelled with both software tools and is considered as the dominant transport process in most river systems, since the circulation process in river is considered well-mixed in both laterally and vertically (Thomann and Mueller, 1987). WASP5 can simulate river water quality in two and three dimensions. The possible increase in DO concentration by water quality structures such as dams and weirs can only be considered in QUAL2E.

The second category is related to water quality. Both software tools account for most sinks on DO processes. Built-in reaeration formulae are available in both QUAL2E and WASP5. QUAL2E accounts for the four forms of nitrogen in the nitrogen cycle (i.e. Org-N, NH_3, NO_2 and NO_3), as shown in Figure 2.2. WASP5 combines NO_2 and NO_3, in the overall nitrification process from NH_3 to NO_3. However, as stated in USEPA (1997b), lumping of NO_2 and NO_3 does not cause any significant effect in the overall result, since the transformation of NO_2 to NO_3 is rapid. All phosphorus processes including the algae cycle can be accounted in both software tools. Both software tools have the ability to simulate both settling and benthic activity, which are important for streams with low velocity. QUAL2E is the only software, which can simulate temperature using atmospheric heat balance equation.

The third category deals with the additional attributes. QUAL2E provides a built-in uncertainty and sensitivity analysis module, which is useful in determining the sensitivity of input parameters to output water quality. The uncertainty and sensitivity analysis of model parameters is a major component in the overall model development.

Based on the above evaluation, WASP5 is considered to be 'over-qualified' for the development of the Yarra River water quality model. Steady state simulation is considered sufficient for this study, because it can be used to determine the critical water quality concentrations under design conditions. This is necessary when the model is used to simulate and study the effect of different effluent license limits on river water quality. Furthermore, the required data are not available for Yarra River to develop an unsteady model, and also to model water quality in two and three dimensions. QUAL2E is also less complex and provides all the essential elements that are required for modelling Yarra River water quality. These elements include modelling of interaction of nonconservative water quality constituents and the built-in uncertainty and sensitivity analysis. The use of QUAL2E is also supported by a large number of applications in river water quality modelling (e.g. Mulligan and Brown, 1998 and Ning *et al.*, 2001).

2.5 Calibration of River Water Quality Models

Once the river water quality model is developed using the appropriate river water quality software and data, it is necessary to calibrate the model before it can be confidently used as a decision making tool. Model calibration is necessary when parameters cannot be measured physically. One such parameter (which cannot be physically measured) is the decay rate of BOD and obviously there are other such parameters in river water quality models.

Model calibration is frequently referred to as parameter estimation (Beck *et al.*, 1997), because the calibration yields the model parameters. Model calibration techniques can be broadly divided into two categories: manual and automatic, as shown in Figure 2.5. The former is a trial and error method, where the model output due to different parameter sets is compared with observations visually (Tsihrintzis *et al.*, 1995 and Janssen and Heuberger, 1999). This method is subjective and time consuming. It can also miss the 'optimum' parameter set. It may even lead to unrealistic parameter sets (Sorooshian and Gupta, 1995 and Mohan, 1997). On the other hand, the automatic calibration method provides some measure of objectivity to parameter estimation and generally conducted through optimisation. These optimisation methods are discussed in Section 2.5.1.

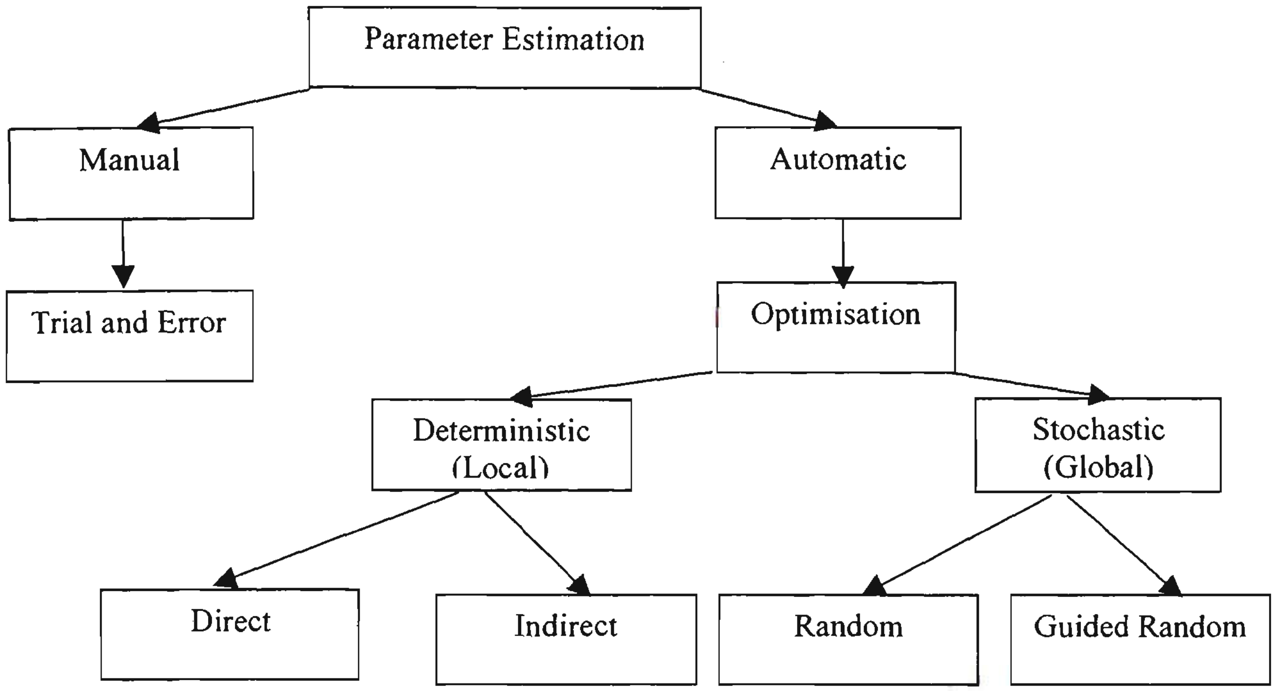


Figure 2.5 Broad Methods in Model Parameter Estimation

2.5.1 Model parameter optimisation methods

The objective of the model parameter optimisation is to find the 'best' parameter set through an objective function, which minimises or maximises an user-defined function. The minimisation of the residual sum of squares between the actual and modelled values has been commonly used as the objective function in many hydrological studies (Johnston and Pilgrim, 1976; Wang, 1991; Little and Williams, 1992; Mohan, 1997 and Mulligan and Brown, 1998). The use of this objective function is known as the least square method. As stated by Sorooshian and Gupta (1995), the selection of the objective function can be subjective and can produce different results for different objective functions. For example, the results obtained from two different objective functions considering peak flows and runoff volumes can be different.

In parameter optimisation, an optimisation technique is used to determine the 'optimum' parameter set within a prescribed parameter space. As Figure 2.5 shows, the optimisation techniques can be divided into two broad categories: deterministic and stochastic. The deterministic techniques (also defined as local search methods) determine the 'optimum' parameter set through a systematic search. They are designed to locate the 'optimum' parameter set when the response surface defined by the user-defined function is uni-modal (i.e. a function with a single peak/trough). However, if the response surface is multi-modal, the parameter set obtained from deterministic methods may not produce the global optimum, since the solution can be trapped at a local optimum point. Starting the optimisation with different 'seeds' (i.e. starting parameter set for the optimisation) may alleviate this problem to a certain extent. Sorooshian and Gupta (1995) stated that in calibration of hydrologic models, very rarely the 'optimum' parameter set is found through deterministic methods, since the hydrological problems contain multi-modal response surfaces. Duan *et al.* (1992) showed that there were more than hundreds of local optimum solutions in their rainfall and runoff model.

Direct and indirect search methods are two deterministic optimisation methods. The direct method seeks the optimum by "hopping" around the search space of a pre-defined grid and assessing the objective function at each of these grid points. The objective function values of new and old points are compared to determine the next search point. Sorooshian and Gupta (1995) listed the most common direct search methods: Rosenbrock (Rosenbrock, 1960), Pattern Search (Hooke and Jeeves, 1961), and Nelder-Mead downhill simplex methods (Nelder and Mead, 1965). The indirect method (also known as the gradient search method) seeks the 'optimum' solution by defining the next search point, considering both the objective function value and its gradient. Steepest decent, Hessian matrix and Newton method are examples of indirect methods (Sorooshian and Gupta, 1995). These three methods start from an user-defined starting point but differ from each other from the direction of moves and the length of move.

As compared to deterministic methods, the stochastic (or global) methods are more efficient in locating the 'optimum' parameter set, when the response surface is multi-modal. They can also be used when the response surface is uni-modal. Stochastic methods are also known as global search methods, since they are designed to produce the global 'optimum' parameter set. The stochastic methods can be sub-divided into two main categories, namely random and guided random. The principle of the random search method is to select the parameter set randomly from the parameter range and optimise the parameter set. Generally, the random search method selects a parameter set from a uniform distribution of parameters. It does not consider the history of previous solutions in terms of optimality to determine the next parameter set, and hence the method can be inefficient. On the other hand, the guided random search method provides guided information for the next search based on the history of previously considered points (Filho *et al.*, 1994).

Several guided random search methods exist, such as simulated annealing, adaptive random search, shuffled complex algorithms and evolutionary algorithm (EA) (Duan *et al.*, 1992 and Filho *et al.*, 1994). Of these four algorithms, EA has been recently used by many researchers and have attracted wide attention from diverse fields, such as applications in different areas of engineering, computer science, operations research, mathematics and political science. The growing number of these applications is due to their ease of interfacing, simplicity and extensibility (Dasgupta and Michalewicz, 1997).

2.5.2 Evolutionary algorithm

Evolutionary algorithm (EA) is a stochastic optimisation method that utilises the natural process of evolution (De Jong *et al.*, 1997). It has been demonstrated that EA is a robust search technique that outperforms the traditional optimisation methods in many applications, in particular when the response surface is discontinuous, noisy, non-differentiable and multi-modal (Goldberg, 1989; Schwefel, 1997 and Mulligan and Brown, 1998). Bäck and Schwefel (1993) stated that EA has become a common and successful method in model parameter optimisation. EA has been successfully used in model parameter optimisation by Mulligan and Brown (1998) and Seibert *et al.* (2000).

There are three main forms of EA: evolutionary programming (Fogel, 1993), evolutionary strategies (Schwefel, 1981) and genetic algorithm (GA) (Goldberg, 1989). Apart from these three forms of EA, there are two other forms which were originally derived from GA: classifier system (Goldberg, 1989) and genetic programming (Kinnear *et al.*, 1997). These five forms share a common conceptual principle of EA. That is, they repeatedly apply sequential evolutionary operators that simulate the evolution of parameter sets from the search space. These evolutionary operators are parameter representation, parameter initialisation, selection, crossover and mutation to yield offsprings (or new parameter sets) for the next generation. Depending on the type of EA, these evolutionary operators are applied simultaneously or selectively. Of these five forms, GA has proven to provide robust search in complex parameter search space, since it is the only form of EA that utilises all five evolutionary operators, namely parameter representation, population initialisation, selection, crossover and mutation (Eshelman, 1997). Details of the GA and its applications in the area of water resources are reviewed in Section 2.6.

2.6 Genetic Algorithm

Genetic algorithm (GA) is the most prominent and powerful optimisation technique that has been applied successfully recently in many disciplines (Paz, 1998). It is a robust search technique that is based on concepts of natural selection and genetics. For this reason, the terminology used in GA is borrowed from natural genetics. The overall GA process as applicable to parameter optimisation of river water quality models is described below.

Mathematical models have their own model parameters. According to the genetics terminology, each model parameter is a gene, while a complete set of model parameters is a chromosome. The process of GA begins with an initial population of number of model parameter sets, which are chosen at random within a specified parameter range. This is the first generation of a number of generations (generally with a constant population size for all generations) in a GA run. Each model parameter set is then evaluated via an objective function to yield its fitness value (Sorooshian and Gupta, 1995).

The second and subsequent populations (or generations) are generated by combining model parameter sets with high fitness values from the previous population (i.e. parent) through selection, crossover and mutation operations to produce successively fitter model parameter sets (i.e. offsprings). The selection GA operator favours those parent parameter sets with high fitness value to those of lower fitness value in producing offsprings. The crossover operator exchanges model parameter values from two selected model parameter sets. The mutation operator adds variability to randomly selected model parameter sets by altering some of the values randomly. Several generations are considered in one GA run, until no further improvement (within a certain tolerance) is achieved in the objective function.

2.6.1 Genetic algorithm operators

In general, the efficiency of GA depends on the proper selection of GA operators, which are essentially the components that make up the overall GA process. Five essential GA operators are required within the GA process. They are parameter representation, population initialisation, selection of subsequent populations, and crossover and mutation to yield offsprings for the next generation. Each of these operators is defined below.

2.6.1.1 Parameter representation

Traditionally, GA operates on the concept of coding parameters into string-like structure to transform from a continuous search space problem into a discrete problem. This can simplify one large parameter set into subsets and optimise these subsets first and then build towards an optimum parameter set (Goldberg, 1989 and Fogel, 1997). Binary and gray coding systems have been the two most commonly used schemes for parameter representation. However, some studies have used direct real valued vector as parameter representation which view the parameter set in one entirety without decomposing into subsets (Wardlaw and Sharif, 1999). This representation is more accurate because it remains as a continuous search space problem. Both binary and gray coding systems work on similar concept and therefore, they are described together below, followed by the real value coding system.

(a) Binary and gray coding systems

Both binary and gray coding require each parameter value encoded into bits of 0's and 1's (only two states). These methods are widely used in many applications (Wang, 1991; Liong *et al.*, 1995; Savic and Walters, 1997; Mulligan and Brown, 1998 and Vasquez *et al.,* 2000), since they are easy and straightforward to use (Goldberg, 1989). This form of coding scheme requires discretisation of the parameter space and can be done using Equation 2.1.

Consider a parameter value (X), which requires to be binary-coded within the parameter search range of (0, 2). The first step is to decide on the required precision for coding X, which effectively determines the length of the binary string (Michalewicz, 1996). This is determined using the following inequality:

$$2^{s} - 1 \geq (U - L)10^{d} \qquad 2.1$$

where	s	is the length of the binary string (which is an integer number)
	U	is the upper bound of the parameter range
	L	is the lower bound of the parameter range
	d	is the number of decimal places required for accuracy of decoded values

If two decimal places are required, then the length of the binary strings, is determined by substituting 2 in place of d, in Equation 2.1, which is shown below.

$$2^{s} - 1 \geq (2 - 0)10^{2}$$

$$2^{s} - 1 \geq 200$$

$$2^S \geq 201$$

$$s \geq 7.65$$

Therefore, to code parameters with this precision (i.e. 2 decimal places) the string lengths required is at least 8 bits (which have been rounded-off from 7.65). This means the search space (0-2) is discretised into 256 (i.e. 2^8) parts having a width of 0.0078 (i.e. ratio of search space to the total number of parts). For example, if X has a value of 1.67, it is located at the 214^{th} part of 256. To encode the value 1.67 into binary string, its corresponding part (i.e. 214) is divided by 2 (because binary string only has 2 states, 0's and 1's) repeatedly until the result is 1, as shown in Figure 2.6. Then, the result (which is always 1) and the remainder values are read from the bottom to the top as shown by the direction of the arrows in Figure 2.6. These values to be read are in bold form in Figure 2.6 and the binary string for this case is 11010110. In this case, all 8 bits are filled with 0's and 1's. There may be cases, where all bits will not be filled. In such case, the empty bits are considered to be at the beginning of the string and they are filled with 0's.

2	214	
2	107	**R0**
2	53	**R1**
2	26	**R1**
2	13	**R0**
2	6	**R1**
2	3	**R0**
	1	**R1**

R represents Remainder

Figure 2.6 Conversion from Numeric Value to Binary String

To decode '11010110' from binary back to numeric value 1.67, the following formula is used (Michalewicz, 1996)

$$x = L + x'\left(\frac{U - L}{2^S - 1}\right) \qquad 2.2$$

where x is the decoded value within the range

U	is the upper bound of the parameter range
L	is the lower bound of the parameter range
x'	is the binary string part (obtained from Figure 2.6)
s	is the length of the binary string

In the example above, the string '11010110' is converted to its corresponding part as 214 (which is x') through the reverse process of Figure 2.6, and then decoded as,

$$\begin{aligned} x &= 0 + 214 \left(\frac{2 - 0}{2^8 - 1} \right) \\ &= 1.67 \end{aligned}$$

Gray coding is an extension of the binary representation, which is designed to overcome a problem experienced in binary coding called 'Hamming Cliffs'. The 'Hamming Cliffs' occurs when two numeric values differ only by one, but in binary representation it differs by many bits. This can be inefficient during parameter optimisation, which requires changes of many bits to just increase by one numeric value. An example is given below to explain the effect of 'Hamming Cliffs'. Consider the two adjacent integer numeric parameter values, 15 and 16, which only differ by one. However, in a binary representation, as shown in Table 2.2, they are (01111) and (10000) respectively, with all binary bits different for the two numeric values. However, using the standard truth table conversion known as XOR (Exclusive OR) as shown in Table 2.3, the binary coding can be converted to gray coding where the gray code differs only by one bit. The procedure for conversion is described as follows. The first bit of the gray code remains as in binary code. The second bit of the gray code depends on first (A) and second (B) bit of the binary code. If A and B are different, then the second bit of the gray code is 1, otherwise it is 0. The third bit of the gray code is then converted as in the second bit by considering the second and third bits of the binary code. This procedure is repeated for the whole binary string. An example of the conversion from binary code to gray code for the value 15 is demonstrated in Figure 2.7 using the XOR concept. Table 2.2 shows both binary and gray codes for numeric values of 15 and 16.

Hollstein (1971) and Haupt and Haupt (1998) showed that gray codes outperformed binary coding in terms of convergence.

Table 2.2 Comparison Between Binary and Gray Codes

Numeric value	Binary code	Gray code
15	01111	01000
16	10000	11000

Table 2.3 XOR Conversion

Binary Code		Gray Code
First Bit (A)	Second Bit (B)	Second Bit (P)
0	0	0
0	1	1
1	0	1
1	1	0

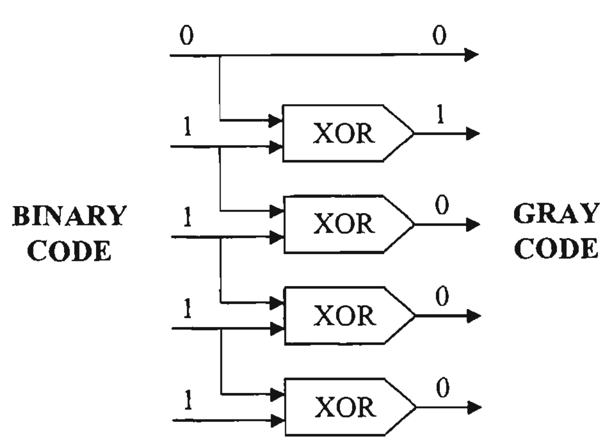


Figure 2.7 Example of Conversion from Binary Coding to Gray Coding for Numeric Value 15

(b) Real coding

The use of real value coding representation has been done with many successful applications (Oliveira and Loucks, 1997; Wardlaw and Sharif, 1999 and Yoon and Shoemaker, 1999). This method uses the actual value itself within the given parameter range which does not require encoding or decoding, and hence the computer effort is less. Furthermore, no discretisation of the parameter space is required as in binary and gray coding systems. Yoon and Shoemaker (1999) stated that the real value method outperforms the binary coding method in their comparison in terms of speed and accuracy. Michalewicz (1996) stated that the real coding provides more consistent and precise result.

2.6.1.2 Population initialisation

The population initialisation is the process of generating initial parameter sets for the GA run. Two methods, namely random and heuristic are used to initiate the population. Each method has its advantages and disadvantages. The commonly used random method generates parameters randomly without any prior knowledge of the likely 'optimum' parameter set. The heuristic method, on the other hand, requires some prior knowledge of the likely 'optimum' parameter set and therefore, this method provides the 'optimum' solution faster. However, with this method, there is a possibility of producing a local optimum due to less parameter sets variation within the population. Such small variation in parameter set is termed loss of diversity (Grefenstette, 1995).

In generating parameters randomly, a 'seed' is used to start the random number generator and the parameters. Different seeds produce different random numbers and hence different initial population. Therefore, the seed can be a factor in affecting the search of the 'optimum' parameter set. Wang (1991), Franchini (1996) and Mohan (1997) studied the effect of seed in parameter optimisation applications.

Wang (1991) used GA in his conceptual rainfall and runoff model in optimising seven model parameters. He compared the best parameter sets from 10 GA runs, initiated with different seeds. Of the 10, he found that 8 runs produced the same 'optimum' parameter set, while the other two produced results slightly different from the 'optimum' set.

Franchini (1996) considered the effect of seed in obtaining the 'optimum' parameter set in his conceptual rainfall and runoff model. Similar to Wang (1991), Franchini also analysed the 'optimum' parameter set obtained from 10 runs with different seeds. He found that the objective function value from the best parameter set of the 10 runs were comparable, however, the values of the parameters showed a difference. He claimed that this difference was due to the errors in data and the imperfect structure of the model which caused some of the parameters to be insensitive and hence different results.

Mohan (1997) used GA to estimate parameters in a nonlinear Muskingum flood routing model, considering 20 different seeds. The parameter set which produced lowest objective function from each of the 20 runs was considered as the 'optimum' parameter set. Although he did not comment on the reason in considering 20 different seeds in his study, he implicitly suggested that seed does play a role.

2.6.1.3 Selection and sampling methods

The selection process determines the number of parameter sets in the current generation that participates in generating new parameter sets for the next generation. This process is governed by the fitness values of the parameters of the current generation obtained via the user-defined objective function. The parameter sets with the highest fitness value can be expected to have the greatest number of times participating in generating new parameter sets. This number is called 'parent copies' in this thesis.

Whitley (1989) stated that it is necessary to maintain a good balance between selection pressure and maintaining the selection diversity in the selection process. The selection pressure puts more emphasis on the fitter parameter sets, and hence more of the 'parent copies' being generated with parameters with high fitness values than those with less fitness values. This loses the selection diversity (i.e. parameter choices), and could converge to a local optimum prematurely. Therefore, the methods used in the selection process should have the ability to account for a balance between selection pressure and selection diversity.

Three selection methods, namely proportionate selection (Grefenstette, 1997a), linear ranking (Grefenstette, 1997b) and tournament selection (Blickle, 1997), are commonly used to determine the number of 'parent copies'. Goldberg and Deb (1991) stated that no one selection method is superior to the others.

(a) Proportionate selection method

The proportionate selection method is the traditional selection method, which calculates the number of 'parent copies' (E(x)) relative to each parameter set fitness value in the population, as shown in Equation 2.3. The number determined from Equation 2.3 is called the real number 'parent copies'.

$$E(x) = \frac{\text{Fitness}}{\text{AvgFitness}} \qquad 2.3$$

where Fitness is the fitness value of the parameter set determined from the user-defined objective function

AvgFitness is the average fitness value of the population range

This method is popular because of its simplicity. However, in situations when fitness values between each parent (or the fitness range in the population) are comparable, then parents cannot be distinguished from one another, which causes the 'stagnate' search. This problem can be overcome by scaling or standardising each parent's fitness value with the lowest fitness in the population (Grefenstette, 1997a). This can reduce the overall average fitness within the population, hence the best and the worst parameter set can be distinguished. Although scaling can overcome the problem of stagnate search, it is unable to handle situations when selection pressure is high. This is because when a parameter set is exceptionally better than the others, the probability in obtaining the number of 'parent copies' can be high especially when the scaling is to the 'worst' parent, which lead to premature convergence (Deb, 1997). Simpson *et al.* (1995) and Reis *et al.* (1997) used the proportionate selection method without scaling, while Meier and Barkdoll (2000) used the proportionate selection method with scaling in their applications of pipe network design.

(b) Linear ranking selection method

To avoid the problem associated with high selection pressure, the linear ranking selection method can be used (Whitley, 1989 and Eshelman, 1997). The linear ranking selection method involves the determination of 'parent copies' based on the rank order that is proportional to the fitness value of the parents within the population, and not on the raw fitness values (Grefenstette, 1997a). This is done via Equation 2.4.

$$E(x) = N_{min} + rank\left(\frac{N_{max} - N_{min}}{pop - 1}\right) \tag{2.4}$$

where N_{min} is the minimum number of parent copies specified by the user for high fitness parameter set

N_{max} is the maximum number of parent copies specified by the user for low fitness parameter set

rank is the rank of the parent under consideration of the current generation

pop is the population size

This method effectively assigns the lower and upper bound of the number of 'parent copies' for each parent to limit the high selection pressure (Davis, 1991). One drawback of this method is the lack of information on the relative fitness of each parent, since the number of 'parent copies' is based on the rank rather than the fitness. Savic and Walters (1997) and Mulligan and Brown (1998) successfully used the linear ranking selection method in their applications.

(c) Sampling methods for use in proportionate and linear ranking selection methods

In the above selection methods (i.e. proportionate and linear ranking selection methods), the number of 'parent copies' calculated for each parameter set can be a real number. However, it is not realistic to have a real number of 'parent copies' of a particular parameter set going into the next generation for creating new parameter sets. Therefore, the sampling process is undertaken to translate the number of 'parent copies' from real number to integer. When translating the number of 'parent copies' from real to integer, Baker (1987) stated that the following attributes should be considered.

- The number of parameter sets in each of the population is the same
- The absolute difference between the integer number of 'parent copies' and the real number of 'parent copies' determined from Equations 2.3 and 2.4 should be small, which can account for accuracy in the conversion.
- The difference between minimum and maximum integer number of parent copies should be small to account for precision.
- The computer time used for the sampling process should not increase the overall GA time.

The sampling process is based on the concept of spinning the roulette wheel (Goldberg, 1989). The roulette wheel (or spinning) wheel consists of a slot for each parameter set of the current generation. The size of the slot is apportioned based on the real number of 'parent copies' determined from either Equation 2.3 or 2.4 (Baker, 1987). The higher the value the larger the area of the slot for that particular parameter set and vice versa. To determine the integer 'parent copies', the roulette wheel requires to spin N times, where N is the number of parameter sets in the population. The integer number of 'parent copies' allocate to individual parent can be obtained by summing the number of times the spin has landed on respective slot (i.e. parameter set). This method has low precision, because the integer number of 'parent copies' can be quite different from the real (Mitchell, 1996).

Due to the above drawback of the traditional roulette wheel sampling method, several roulette wheel modifications have emerged, as summarised in Baker (1987) and Michalewicz (1996). These methods are remainder stochastic sampling with replacement, remainder stochastic sampling without replacement, deterministic sampling, remainder stochastic independent sampling and stochastic universal sampling (SUS). These methods are discussed in Baker (1987). Of these five methods, SUS is considered as the best sampling method in terms of accuracy, precision and computer time (Baker, 1987), and has been used successfully in the applications by Liong *et al.* (1995) and Mulligan and Brown (1998).

The roulette wheel used for the SUS method is similar to the traditional roulette wheel, except that it has N number of pointers, which represents the total number of parents in the population. The pointers are equally spaced 1.0 apart. The size of slots is as in the traditional roulette wheel. SUS requires only one single spin on the roulette wheel in which the integer number of 'parent copies' of each parameter set can be obtained in that spin. This method guarantees each parent's real number of copies but not more than the integer number of 'parent copies'. For example, if a parent has 1.4 real 'parent copies', it can be guaranteed that at least 1 but no more than 2 'parent copies' will be obtained.

(d) Tournament selection method

As Goldberg and Deb (1991) commented, the major disadvantage in both proportionate selection and linear ranking selection methods is that many steps are involved for selection which lead to more computer time and effort. However, with advancement in computer technology, this problem has been reduced. However, to further enhance the efficiency in computer time in the selection phase, the tournament selection was recommended, where the real number of 'parent copies' is selected through competition (Mitchell, 1996). This is performed by randomly selecting 2 parents within the population regardless of their fitness value. These 2 parents then undergo a competition in striving to proceed to the next generation by comparing their fitness value. The parameter set with the highest fitness value is considered the winner of the competition.

After competition, both parameter sets return back to the current population and the whole process is repeated until the number of parameter sets has been obtained for the next generation. With this process, one parameter set can win many times and the winners go to the next generation to create the new parameter set. The tournament selection method eliminates the process of conversion from real number of 'parent copies' to the integer number of both proportionate selection and linear ranking selection method. Cieniawski *et al.* (1995), Wardlaw and Sharif (1999) and Vasquez *et al.* (2000) used the tournament selection scheme successfully.

2.6.1.4 Mutation

The mutation operator is used to add variability to the parent parameter sets, which are selected to produce offsprings. This operation is conducted through small alterations to the values in the parent parameter sets. It introduces some innovative parameters to the population and is governed by the mutation rate. The rate at which the gene value alters within one generation is governed by the mutation rate. The typical literature range for mutation rate is 0.01 or less (Goldberg, 1989); this will introduce only a small number of innovative values to the population of new parameters without replacing too many good values of the parameter sets.

For binary and gray coding systems, mutation is done by changing the parameter values (or genes) (by flipping 0's to 1's and vice versa) at randomly chosen locations. Two mutation methods are used in real value representation, namely uniform and non-uniform mutations (Michalewicz, 1996). These methods differ from each other by the way the mutation rate is used in GA. In uniform mutation, the number of parameter sets and genes to be mutated are determined by the mutation rate, which is fixed for all generations within one GA run. However, in non-uniform mutation, the mutation rate reduces as the run progresses from one generation to another (Wardlaw and Sharif, 1999). In both methods, individual genes (i.e. a digit of a parameter value) in the parameter set are mutated randomly within their feasible parameter range.

2.6.1.5 Crossover

The crossover operator is used to create new parameter sets (i.e. offsprings) by modifying the parent parameter sets that were selected to participate in the next generation through selection. This is done by partially exchanging parameter values between two selected parents.

Three different crossover approaches have been cited in the literature namely, one-point crossover, two-point crossover and uniform crossover (Booker, 1997). The one-point crossover, as shown in Figure 2.8, exchanges one block of genes from two parent parameter sets at a randomly selected gene location to yield 2 offsprings. The two-point crossover is similar to the one-point crossover, except two blocks (2 blocks are commonly used) of genes from two parent parameter sets are swapped at two randomly selected gene locations. A two-point crossover is shown in Figure 2.9. The uniform crossover operates on randomly selected individual genes of two parent parameter sets, as shown in Figure 2.10. In Figures 2.8-2.10, the genes (or the parameter values) are defined by letters.

A B C D	E F G H	Parent (1)
I J K L	M N O P	Parent (2)
A B C D	M N O P	Offspring (1)
I J K L	E F G H	Offspring (2)

Figure 2.8 One-Point Crossover

A B	C D	E F G H	Parent (1)
I J	K L	M N O P	Parent (2)
A B	K L	E F G H	Offspring (1)
I J	C D	M N O P	Offspring (2)

Figure 2.9 Two-Point Crossover

A B C D E F G H	Parent (1)
I J K L M N O P	Parent (2)
A J C D M F O H	Offspring (1)
I B K L E N G P	Offspring (2)

Figure 2.10 Uniform Crossover

When undertaking crossover with parameters encoded in binary, the choice of the location performing the crossover operation is important in preventing the splitting of genes (i.e. parameter values), because each parameter set contains a set of genes. Splitting of genes may lead to a diverging solution (Haupt and Haupt, 1998). Therefore, in binary and gray coding, the crossover should occur only at the gene boundaries. In real value representation, the splitting of gene is not an issue, since the gene itself is the parameter value.

The number of parameter sets that participate in the crossover process is governed by the crossover rate. A higher crossover rate allows more parameter sets to participate in the crossover process and vice versa. Goldberg (1989) suggested a crossover rate between 0.6-0.9, while Wardlaw and Sharif (1999) suggested a value between 0.5-1.0.

2.6.1.6 Importance of GA operators

It is generally understood that the GA operators (i.e. parameter representation, population initialisation, selection of subsequent populations, crossover and mutation) play a role in the GA optimisation of model parameters from the viewpoint of computer efficiency (Franchini, 1996 and Franchini and Galeati, 1997). There are general guidelines available for selecting GA operators for applications in any field (Goldberg, 1989). However, the importance of GA operators has only been studied for very few applications in specific fields (e.g. rainfall and runoff applications).

Several studies have been reported on the importance of GA operators on water resource applications. The most comprehensive study was on a rainfall and runoff application by Franchini and Geleati (1997). They found that the GA operators did not have any significant impact on the 'optimum' parameter set, and therefore stated that a robust GA operator range was adequate. On the other hand, Davis (1991) commented that the optimum GA operator set varies from problem to problem, but a reasonable robust GA operator range can provide an efficient solution. The reasonable robust ranges for various GA operators (for crossover and mutation rates) are given in Goldberg (1989). Mulligan and Brown (1998), in their water quality modelling application, explored the effect of constant mutation rate throughout the GA run against varying rates, and found that the constant rate was efficient. Michalewicz (1996) suggested that a detail investigation of GA operators could guarantee better performance in GA optimisation. As seen from these studies, the importance of the GA operators in achieving the 'optimum' parameter set was inconclusive. Therefore, this issue is investigated in detail in Chapter 6.

2.6.2 Application of genetic algorithm in hydrological model parameter optimisation

The literature describing GA applications in water resources is not extensive. Few applications were cited in catchment and water quality modelling, which used GA to optimise model parameters (Wang, 1991; Liong *et al.*, 1995; Mohan and Loucks, 1995; Mohan, 1997 and Mulligan and Brown, 1998). The other applications in water resources include solving groundwater management problems (McKinney and Lin, 1994; Ritzel and Eheart, 1994 and Yoon and Shoemaker, 1999), analysis of water distribution networks (Simpson *et al.*, 1995; Reis *et al.*, 1997; Savic and Walters, 1997 and Meier and Barkdoll, 2000), optimisation of wastewater treatment cost and reliability in meeting water quality standards (Vasquez *et al.*, 2000) and detecting leakage in pipe networks (Vitkovsky *et al.*, 2000).

Wang (1991) used GA to calibrate the rainfall-runoff model - *The Xinanjiang model* (Zhao and Liu, 1995). He used the objective function of minimising the residual sum of squares to determine the optimum set of seven model parameters. Wang (1991) performed 10 different GA runs starting with different initial conditions, and found that in eight GA runs, the same parameter set was obtained, while the remaining two runs produced parameters slightly different to the above parameter set.

Liong *et al.* (1995) coupled the public domain GA software - GENESIS (Grefenstette, 1995) with the urban catchment model SWMM (Huber *et al.*, 1982). They adopted the default values of crossover and mutation rates given in GENESIS, and did not study the effects of these rates on the optimum model parameter set and the convergence rate. Mohan and Loucks (1995) and Mohan (1997) used GA to calibrate parameters in two non-linear Muskingum flood routing models, and compared the results with those from previous studies which used different parameter estimation procedures (Gill, 1978 and Yoon and Padmanabhan, 1993). The objective function used was the minimisation of the residual sum of squares between observed and computed peak outflows. A sensitivity analysis was then carried out, which proved that an unique parameter set existed for the routing problem. The use of GA in this application has shown to be efficient and accurate with respect to computation time and accuracy, compared to the other parameter estimation studies of Gill (1978) and Yoon and Padmanabhan (1993).

Mulligan and Brown (1998) coupled GENESIS with the river water quality model QUAL2E (Brown and Barnwell, 1987), to optimise the model parameters. The results from GA were compared with a calculus-based calibration method known as the Marquardt algorithm (Marquardt, 1963). Both methods produced comparable results for the optimum parameter set, however, slight differences were discovered due to numerical rounding-off errors in the GA method. They only studied the effect of mutation rates on the results.

2.7 Parameter Uncertainty and Sensitivity in River Water Quality Models

As discussed in Section 2.4, many river water quality modelling software tools are available, which can be used to simulate water quality in rivers. However, these tools were developed with certain assumptions, simplifications and approximations of the natural processes, which lead to unavoidable errors. The extent of this error is termed uncertainty, and its effect on model output is defined as sensitivity.

In any model development, it is first necessary to identify different types of uncertainties that may exist in the model, and to study the sensitivity of the output associated with these different errors. Depending on the availability of data and types of uncertainty focus required in a particular study, different uncertainty and sensitivity methods can be used. These are discussed in Section 2.7.2.

2.7.1 Types of uncertainty

As defined by Burges and Lettenmaier (1975), two broad types of uncertainties exist in mathematical models. The first type is *Type I* error, which is the result of selecting an incorrect model, while *Type II* error is defined as the error associated with the selection of inaccurate model parameters of a selected model. Beck (1987) provided an excellent review on the topic of uncertainty and discussed three main types of uncertainties in water quality modelling that affect the model output. They are listed below.

- Model structure uncertainty – uncertainty that deals with the error associated with the interpretation and transformation of natural river processes in model algorithms. This is the *Type I* error of Burges and Lettenmaier (1975).
- Data uncertainty – uncertainty caused by the inaccuracies of data, data handling and sampling errors.

- Model parameter uncertainty - uncertainty that deals with the error associated with model parameters. This is the *Type II* error of Burges and Lettenmaier (1975).

In addition to the above uncertainties, Yeh and Tung (1993) and Lei and Schilling (1994) introduced an additional uncertainty, as follows.

- Operational uncertainty – uncertainty caused by human errors that are not accounted for in modelling or design procedures, for example data entry.

In most cases, these four types of uncertainties are inter-related and can be combined to produce the overall prediction uncertainty. A fair amount of research effort has been and continuously being spent on refining model algorithms on standard river water quality models to reduce the model structure uncertainty, and therefore this uncertainty can be overcome by selecting a standard model (Shanahan *et al.*, 1998). Preliminary investigations on data accuracy and consistency can reduce data uncertainty. The operational uncertainty can be considered negligible, if models are used by the users carefully. Model parameter uncertainty is considered an important issue especially when no prior knowledge of the effect of input parameters to output water quality response is available. This situation often occurs at different phases of the model calibration in the overall model development.

Two purposes can be served in studying the uncertainty/sensitivity relationship at pre and post-calibration phases of the model development. Pre-calibration uncertainty/sensitivity analysis can identify the insensitive parameters, where less effort and time can be expended on these parameters during model calibration, compared to the sensitive parameters. Little and Williams (1992) commented on the advantage of conducting an uncertainty/sensitivity analysis on model parameters prior to model calibration. Dawdy and O'Donnell (1965) stated that more the sensitive the parameter to output response, the sooner it would be optimised during the model calibration. Furthermore, they also stated that the insensitive parameters never approach the 'optimum' value. This implies that early identification of insensitive parameters reduces much time and effort during the parameter calibration process. Other researchers, such as Haan and Zhang (1996) and Schladow and Hamilton (1997) stated similar conclusions.

Post-calibration sensitivity analysis, on the other hand, is conducted to assess the effect of changes (usually deviation from the actual or 'optimised' values) in the input parameters on the output results. This analysis can identify the consequences in the management planning results due to the possible error or deviation away from the 'optimised' values.

2.7.2 Uncertainty and sensitivity analysis methods

Uncertainty and sensitivity analysis methods can be broadly grouped into local and global methods, as suggested by Chang and Yang (1993), Yeh and Tung (1993) and Hamby (1994b). The local methods investigate the sensitivity of output response relative to the deviations of input parameters around the 'optimum' parameter set. These types of methods are most suited for use in the post-calibration phase. The global methods on the other hand, provide a wider knowledge of the sensitivity of the output responses due to many parameter sets covering their entire parameter range, which is considered more appropriate for use in the pre-calibration phase. Under local and global groups, several specific methods are available, as shown in Figure 2.11.

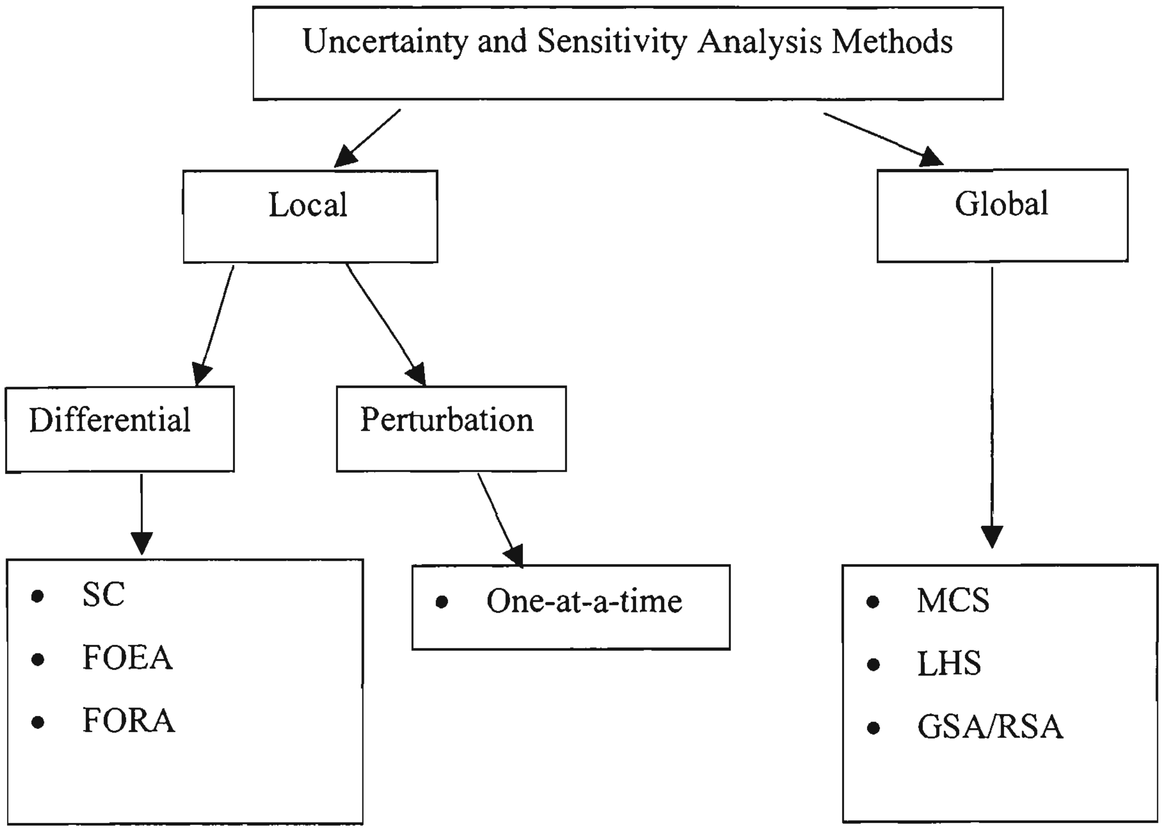


Figure 2.11 Summary of Uncertainty and Sensitivity Analysis Methods

2.7.2.1 Local methods

(a) Differential methods

Differential uncertainty and sensitivity analysis methods perform computations using analytical expressions. This method is limited to mathematical models which are not complex and can be differentiable. Three differential methods are discussed in this section namely, sensitivity coefficient, first order error analysis and first order reliability analysis.

(i) Sensitivity coefficient method

The sensitivity coefficient method analyses sensitivity of model parameters through the sensitivity coefficient (SC). The SC represents the change of the output response caused by a unit change of input parameter from a base value, while holding other input parameters constant at their base values. The base value should be a representative value for the model parameter and can be the 'optimised' value obtained from model calibration. In mathematical terms, SC is the partial derivative of the output response with respect to each of the input parameter, *i* (Sinokrot and Stefan, 1994; Haan and Zhang, 1996; Zerihun *et al.*, 1996 and Gu and Chung, 1998), and given by Equation 2.5.

$$SC_i = \frac{\partial P}{\partial U_i} \qquad 2.5$$

where ∂P is the change in the output response caused by the change in the input parameter (under consideration) from its base value

∂U_i is the change in the individual input parameter *i* from its base value

The SC computed from Equation 2.5 is a dimensional quantity. This form of SC is not commonly used for sensitivity of model parameters, because it cannot be used to rank the parameters because of their different units. To overcome this problem, SC has been normalised using Equation 2.6, and this form has been used by Sinokrot and Stefan (1994) and Zerihun *et al* (1996), as explained later in this section.

$$SCi = \frac{\partial P / P}{\partial Ui / U} \qquad 2.6$$

where ∂P is the change in the output response caused by the change in the input parameter (under consideration) from its base value

∂U_i is the change in the individual input parameter i from its base value

P is the reference value used for output response

U is the reference value for input parameter

The reference value is selected by the user to normalise the input and output values. This chosen value is subjective but it should be representative for comparison. For example, for input parameters it can be the optimised value and for output response, the representation value can be the output due to optimised input parameters. The SC computed from Equation 2.6 can either be positive or negative, depending on the relationship between input parameter and output response. When SC is positive, there is a direct proportional relationship between input parameter and output response, whereas a negative SC represents an inversely proportional relationship between the two variables. Higher the SC, the greater the sensitivity of input parameter to model response.

Sinokrot and Stefan (1994) determined the sensitivity of stream temperature in their MNSTREM (Stefan *et al.*, 1980) dynamic channel water quality model with respect to several input parameters. They found that the most sensitive input parameters to water temperature were air temperature and short-wave radiation, based on high SC values. The reference values chosen for input parameters in their study were the standard deviations of each of the solar and climatological input parameters.

Zerihun *et al.* (1996) successfully used SC to quantitatively ranked sensitivity of 13 input parameters to 7 output responses in their zero inertia furrow irrigation model. Due to limited knowledge on input parameters, they selected the sensitivity coefficient method to obtain a preliminary sensitivity result in their irrigation model for future reference.

(ii) First order error analysis (FOEA) and first order reliability analysis (FORA)

The first order error analysis (FOEA) is based on the approximation of variation in model output with variation in input parameters. FOEA determines the mean and the variance of model output due to variation of input parameters included in a model functional relationship. The truncated first order Taylor series approximation expanded about the mean value of each input parameter to the output response is used. The method is called first order because the higher order terms in the Taylor series expansion have been discarded. The method is described in detail with its mathematical formulations in Burges and Lettenmaier (1975).

FOEA performs satisfactorily with small variations away from the mean and when the model equation is approximately linear. This method lacks accuracy and representativness, especially in most water quality problems dealing with nonlinear behaviour in water quality processes. Another problem of FOEA is in the use of design situation, where most often, events failure is at some extreme points (or values) away from the mean to assess the output sensitivity (Melching and Yoon, 1996). To overcome the above shortcomings of FOEA, Hasofer (1974) proposed the first order reliability analysis (FORA) method. The major difference in FORA to FOEA is that the error expansion point for FORA is not the mean value, instead at a point of failure. This failure point is difficult to locate and may be determined by iteration or by constrained nonlinear optimisation depending on the nature of the problem (Melching and Anmangandla, 1993).

To assess the output sensitivity caused by the input parameter uncertainty, some measures are needed. Hamby (1994b) compiled an excellent review on various methods to interpret output sensitivity results. Three types of measures can be used to assess the sensitivity using FOEA and FORA. They are relative deviation ratio (RDR), sensitivity coefficient (SC) and Cumulative Distribution Frequency (CDF).

The Relative Deviation Ratio (RDR) is defined as the ratio of output coefficient of variation (CV) to input CV. The contribution of input variability can also be accounted for to assess both uncertainty and sensitivity (Hamby, 1994b). Similar to sensitivity coefficient method, RDR is computed for each output response considering each input parameters. CV is defined as the ratio of the standard deviation to the mean. The higher the RDR, the greater the sensitivity of input parameter to output response. As suggested by Hamby (1994b), a threshold value of 1 for RDR can be used to define sensitivity. When RDR is less than 1, the input parameter is insensitive to output response and vice versa. This threshold value is subjective and makes it difficult in judging parameter sensitivity when RDR is in the vicinity of the threshold.

The normalised SC (Equation 2.6) was also used for assessing the output sensitivity. Yeh and Tung (1993) applied FOEA in their Pit-Migration model. They determined SC and ranked the most sensitive input parameter (of the 28 parameters) in affecting the maximum pit depth.

CDF assesses the level of exceedance limit of an output response. The plot of CDF can give a complete view of the prediction reliability with different 'what-if' situations in planning water quality management. This sensitivity measure is not used for parameter ranking. Most often, this method is used with the simulation method such as Monte Carlo simulation (MCS) to obtain an accurate CDF. Nevertheless, based on the mean and the variance, a reasonable CDF can also obtained (Melching and Anmangandla, 1993). Despite the extensive simulations required to obtain an accurate CDF, Melching (1992) and Melching and Anmangandla (1993) have used CDF in their study comparing the output result of FOEA, FORA and MCS.

Several water resource applications have compared the results of FOEA and FORA against another well-known uncertainty and sensitivity analysis method called Monte Carlo simulation – (MCS). These applications are reviewed in Section 2.7.2.3.

(b) Perturbation method

A less complex method to the differential uncertainty and sensitivity method is the 'one-at-a-time' perturbation method (Hamby, 1994a). This method considers one of the input parameters being perturbed by a certain percentage from its chosen point (i.e. base value), while the other parameters are held constant at their base values. The base value can be the 'optimised' value from the model calibration. The advantage in using this method is that it is not restricted by the differentiability of the mathematical model. The actual sensitivity of a particular input parameter can be clearly addressed relative to the output response.

Three forms of measures can be used to assess the output sensitivity using 'one-at-a-time' method. These measures are RDR (discussed earlier) sensitivity index (SI) and a visual plot of output response versus input parameter of interest. The visual plot does not consider parameter ranking.

Hamby (1994b) suggested a simple measure, sensitivity index (SI) which is defined as

$$SI = \frac{D_{max} - D_{min}}{D_{max}} \tag{2.7}$$

where D_{min} is minimum output value obtained from a range of input perturbations

D_{max} is maximum output value obtained from a range of input perturbations

Similar to SC and RDR defined earlier, SI is defined with respect to each output response and corresponding input parameters. SI can be used to assess the output sensitivity due to the entire input parameter distribution by considering many different percentages from its base value. This method is simple yet it can give an overview of the behaviour of the output response caused by a large variation in the input parameter. The higher the SI value, the higher the sensitivity and vice versa.

A plot of the output response versus the percentage perturbation from the base value for each input parameter can visually identify the most sensitive parameter in effecting the overall water quality prediction. This is done by observing which parameter gives the greatest deviation away from the output response produced from the base value. This method gives a quick visual outcome of sensitivity but does not give the ranking of parameter sensitivity.

Drolc and Koncan (1996) performed 'one-at-a-time' sensitivity analysis with QUAL2E to identify the most sensitive parameter to DO concentration. They decreased / increased hydraulic power function coefficients and decay rates by 50% from their base values. The base values used for the hydraulic power functions were the values used for the model, while the reaction rate base values were obtained from literature. They found that temperature and hydraulic power functions were the most sensitive to DO concentration by plotting DO concentration against input parameters deviations from the base value.

2.7.2.2 Global methods

The global methods determine the output sensitivity relative to the entire range of input parameters. The MCS, Latin hypercube sampling (LHS) and Generalised sensitivity analysis (GSA) are three types of global uncertainty and sensitivity analysis methods. These methods are described below.

(a) Monte Carlo simulation

The Monte Carlo simulation (MCS) involves repeated model runs by randomly selecting input parameters from their respective probability density functions (PDFs) simultaneously, and analysing the output response to produce PDFs of output responses. This method includes the effect of model non-linearity. However, the method requires many simulations to obtain accurate distributions of output response PDFs. Despite this, it has gained popularity with the development of modern computers (Song and Brown, 1990). MCS is often used as a reference method for uncertainty/sensitivity analysis of model parameters, to compare the performance of the other methods (Chang *et al.*, 1997). At the end of a model run with many simulations, the means, variances and PDFs (or CDFs) of model output response are determined.

As input, MCS requires mean, variance and PDF of each input parameter. Generally, the information on PDF of input parameters is not available, and even the estimation of PDF is difficult and inaccurate. However, Haan *et al.* (1998) used an AGNPS (Young *et al.*, 1989) model of a catchment to investigate the effect of input parameters on the runoff using MCS. They found that PDF in MCS is of lesser importance than the mean and variance of input parameters. Melching and Anmangandla (1993) and Lei and Schilling (1994) also arrived at similar conclusions in their water quality modelling and rainfall-runoff modelling studies respectively.

Many methods are available to interpret the output sensitivity in global methods than in local methods. Common methods include the correlation coefficient (CC) and the regression coefficient (RC). In addition, RDR and CDF (previously discussed) can also be used for MCS.

The correlation between input parameters to output responses is a common measure when undertaking uncertainty/sensitivity analysis. CC is used to identify the strength of the relationship between input to output response. Many input parameter values sampled from its prescribed probability distribution which yield corresponding output responses can be used to describe the relationship between input parameter and output response. Identification of this relationship can be determined in two ways: visually through scatter plots or computing CC through equations such as Pearson's r and Spearman's ρ. The details of these equations can be found in Hamby (1994b). The higher the coefficient determined from these equations, the greater the sensitivity of the input parameter to output response.

The regression coefficient (RC) can be used to assess the sensitivity strength between input parameters to output responses (McKay, 1988). RC is determined through establishing and solving linear regression equations established between model output and inputs. This linear regression was performed by minimisation of least square estimates from the model output simulated from input parameters (Chang and Yang, 1993; Yeh and Tung, 1993 and Aalderink *et al.*, 1996). A model with many sensitive parameters may result in a complex regression equation, which requires the use of matrices for calculation. When RC is high, the strength between input to output parameters is strong, hence any uncertainty in the input parameter would definitely cause sensitivity to the output response. Due to different units and ranges in the input and output parameters, it is necessary to standardise inputs and to centralise the output to have a more meaningful RC, so that a better comparison among input parameters can be obtained. Some applications of MCS and comparisons with FOEA and FORA are discussed in Section 2.7.2.3.

(b) Latin hypercube sampling

The Latin hypercube sampling (LHS) concept is similar to MCS but differs in the way of selecting parameter sets from the given range. This method reduces the large number of simulation runs required in MCS, yet producing reliable output statistics. The LHS considers reduced number of parameters covering the whole distributions of parameters in a systematic manner. In this method, the range for each parameter is first partitioned into *n* nonoverlapping intervals. The required *n* intervals can be determined as a function of the number of parameters (k) considered in the uncertainty analysis. McKay (1998) suggested that *n* can be equal to 2k, which was sufficient for uncertainty and sensitivity analysis. First, each parameter randomly draws a value from each interval, resulting in *n* random sample values. These sample values are then randomly permuted. The above process was done for k parameters which result in *n* input parameter sets (or runs). These *n* runs are then used to obtain *n* outputs sets. Both CC and RC have been commonly cited in the literature as the sensitivity measures in LHS (Chang and Yang, 1993 and Yeh and Tung, 1993).

Aalderink *et al.* (1996) developed a one-dimensional water quality model - DUFLOW for the Vecht River in Netherlands. They conducted an uncertainty analysis using LHS in assessing the type of model parameter which cause the greatest sensitivity in the overall output water quality response for copper concentration. They commented that the use of LHS was very efficient because of the partition sampling process which reduces the number of simulation runs.

(c) Generalised sensitivity analysis

Generalised sensitivity analysis (GSA) was initially developed by Spear and Hornberger (1980a, 1980b) for use in their eutrophication model. This method was later renamed as regional sensitivity analysis (RSA) (Spear *et al.*, 1994). This method involves three steps, namely a simulation, a classification and a statistical analysis. The simulation is conducted to obtain output response given uncertainty in input parameters. This is generally performed using MCS or LHS method. Each input parameter correspond to its output response were then classified under two groups, namely behaviour (B) or nonbehaviour (NB). This classification is used to determine whether the simulation output mimics the system behaviour. For example, the system behaviour may be the actual water quality concentration. Simulation output response which can resemble the known system behaviour (i.e. simulated and observed water quality alike) is called 'B' and vice versa. Based on these two classified groups, a statistical analysis test using the Kolmogorov-Smirnov test (Hamby, 1994a) is undertaken to assess the separation between the cumulative distribution functions (CDFs) of 'B' and 'NB' classes for each uncertain parameter. The separation between the CDFs of the two classes is used as a sensitivity index to rank the importance of that uncertain input parameter in the model. When the separation of the CDFs between the two classes are not different, the output response simulated from the two classes can be considered random and therefore the uncertainty in the input parameter does not cause sensitivity in the model output and vice versa.

Spear and Hornberger (1980b) used GSA to identify critical uncertainties in their phosphorus model for cultural eutrophication in the Peel Inlet of Western Australia. They utilise MCS as the method used for simulation of output response. The system behaviour used in their study was the simulated output response of *Cladophora* biomass and phosphorus concentrations and values reported in the literature. By using the Kolmogorov-Smirnov statistical analysis test for the assessment of the difference between the two behaviour classes, the sensitivity of input parameters to output response can be determined. They found that 7 of 19 input parameters were insensitive to the *Cladophora* biomass and phosphorus concentration at 99% significant level.

2.7.2.3 Comparison of FOEA, FORA and MCS

In many water resource applications, FOEA and/or FORA results have been compared with those obtained from the widely accepted MCS method. Studies that compared FOEA results against MCS results were reviewed first. Then, the studies which had undertaken a comparison of FOEA and/or FORA with MCS, were reviewed.

Burges and Lettenmaier (1975) applied FOEA and MCS to the well-known Streeter and Phelps (1925) water quality model to determine the sources of uncertainty in the prediction of DO concentration. They found reasonably good agreements of mean and variance of DO and BOD from both methods. Similar to Burges and Lettenmaier (1975), Song and Brown (1990) compared the results of FOEA and MCS in studying the effect of input parameters on the output response of the Streeter-Phelps equation. They found that due to the nonlinearity in the model, the standard deviation of MCS output was larger than that of FOEA for all simulated stream conditions. This was because FOEA could not handle nonlinearity.

Scavia *et al.* (1981) compared the output statistics of algae and nutrient output produced from FOEA and MCS in their Saginaw Bay lake (in the United States) eutrophication model. They found that the variance produced from MCS with FOEA has some significant difference. They concluded that the interpretation of errors in variances from FOEA and MCS should be considered as fundamentally different and stated that FOEA considered predictions about the future behaviour of a typical representative of population, while MCS considered total population as a whole.

Melching (1992) applied FORA in his rainfall-runoff model. He compared the peak discharges with different exceedance probabilities relative to results produced from MCS against FOEA and FORA accounting for nonlinearites by considering a wide range of output outcomes. The results revealed that FORA was able to have a close agreement with MCS over a wide range of exceedance probabilities of peak discharge. On the other hand, FOEA only matched the central portion of exceedance probabilities with MCS and deviated significantly at either end. He also commented that the use of FORA could achieve very similar results to MCS and yet the simulation runs were not intensive.

Melching and Anmangandla (1993) used FORA in their water quality model and compared DO concentrations at different exceedance probabilities with those obtained from FOEA and MCS. They also found that the results produced from FORA were in close agreement with MCS over the entire output probability distribution, and yet less computer time was required. Recently, Maier *et al.* (2001) also showed that FORA can be used successfully to produce comparable results to MCS in their water quality management model developed for the Willamette River in Oregon, USA.

2.8 Summary

Management of river water quality has become increasingly important due to decline in water quality caused by human activities. Successful implementation of efficient management strategies requires the development of river water quality models. These water quality models can be used to simulate and assess the cause and effect relationships of river water quality and then to study various management strategies to improve river water quality, before their implementation.

Many water quality software tools are available in the public domain, which can be used to develop river water quality models. A comprehensive review of these software were conducted in this chapter, especially on river water quality modelling software tools. After a detailed evaluation of the river water quality modelling tools, QUAL2E was considered as the most suitable tool for use in the Yarra River model development. The major evaluation criteria were the appropriateness of available data for use in the modelling software and the purpose of using the model. Only grab samples of water quality data of the river and at sewage treatment plants (STPs) were available for Yarra River and at best they can be considered as steady state data. Nevertheless, these data are sufficient for use in QUAL2E. One of the major purpose of the development of the Yarra River Water Quality Model (YRWQM) is to assess various STP effluent license limits on river water quality and this can be adequately done with QUAL2E software.

In order to use river water quality models successfully, they need to be calibrated. The model calibration is an important stage of the overall model development. Various model calibration techniques were reviewed in this chapter from trial and error methods to optimisation methods, showing their advantages and disadvantages. Of the optimisation methods, stochastic (or global) methods are preferred to deterministic (or local) methods, because theoretically, the stochastic methods produce the global optimum parameter set.

Genetic algorithm (GA) is one such stochastic optimisation technique, which has proven to be successful and efficient in identifying the 'optimum' parameter set in many water resource applications. It is a search technique that is based on the concepts of natural selection and genetics. Because of the success in many different applications, GA was selected to calibrate the model parameters of the YRWQM.

In general, the efficiency of GA in achieving the 'optimum' parameter set depends on the proper selection of GA operators, which are essentially the components that make up the whole GA process. These operators include parameter representation, population initialisation, selection of subsequent populations, and crossover and mutation. Although the significance of these operators has been studied in a number of water resource applications, the results were inconclusive. Therefore, a detailed study needs to be undertaken to provide guidelines on the selection of appropriate GA operator values for use in GA optimisation of model parameters of river water quality models.

Errors are unavoidable in the development of mathematical models dealing with natural processes. They cannot be eliminated completely, but can be reduced. The error due to parameter uncertainty is one of these errors, which is considered in this thesis. This type of uncertainty is considered important, especially when the effect of input parameters to output responses is not known. Such knowledge can greatly enhance the efficiency in the model calibration phase of the overall model development. Conducting a pre-calibration uncertainty/sensitivity analysis address this issue and can therefore reduce the time and effort required in the model calibration. On the other hand, post-calibration sensitivity analysis can assess the output response cause by small deviations away from the 'optimised' value, and provides an indication of risk associated with decision-making on various management strategies related to model predictions.

Different methods that can be used to perform uncertainty/sensitivity analysis of model parameters were discussed in this chapter and can be grouped into local and global methods. Local methods are most suited for use in post-calibration sensitivity analysis because it can address sensitivity at the vicinity of the 'optimised' value. Global methods on the other hand, consider a wide range of values within the prescribed parameter probability density function, which is necessary for pre-calibration uncertainty/sensitivity analysis. Of the uncertainty analysis methods reviewed in Section 2.7.2, the Monte Carlo simulation (MCS) and 'one-at-a-time' perturbation method were selected for use in the pre and post calibration uncertainty analysis respectively in this study, since both methods are built-into QUAL2E.

CHAPTER 3

DESCRIPTION OF YARRA RIVER CATCHMENT AND DATA

3.1 Introduction

As stated in Chapter 1.1, the Yarra River catchment located in Victoria (Australia) was used as the case study in this thesis. The Yarra River is one of the rivers in Melbourne and is a major source of water supply for over 1.5 million Melbourne residents (EPA Victoria, 1999). Over the years, the water quality of the Yarra River catchment has deteriorated due to population increase, emissions from Sewage Treatment Plants (STPs) and inappropriate application of farming chemicals.

Nutrients (i.e. nitrogen and phosphorus) are the main concerns in Yarra River, especially in the Middle and Lower Yarra River segments (EPA Victoria, 1999). Recent studies have shown that these nutrients not only have impact on Yarra River, but also cause eutrophication in Port Phillip Bay (which receives water from Yarra River), thereby minimising the beneficial uses of the Bay (Harris *et al.*, 1996). Yarra Valley Water (1997) stated that nitrogen was the controlling nutrient, and that efforts should be made to reduce nitrogen load entering the Bay.

Over the years, the Environment Protection Authority of Victoria (EPAVIC) has implemented various management strategies focusing on different types of pollution sources in the Yarra River catchment in reducing nutrient levels in the river. The effluent discharge from STPs is one of those pollutant sources, which has received most attention in the recent past, because it can be easily controlled and monitored, as it is a point source. As such, the effluent license limit on STPs has progressively increased to improve river quality. However, implementing such management strategies on STPs have not been investigated for their efficiencies using a properly developed water quality modelling tool of Yarra River. Therefore, it is necessary to develop a Yarra

River Water Quality Model (YRWQM) so that various management scenarios can be studied for their effects before implementation. The proper process of the development of the model involves the assembly of the model, calibration and verification. Data are required in all stages of this process.

Data collection for a river system can be very expensive and can be a difficult task. Cost and data collection time increase if more details and better knowledge about the river system (e.g. three dimensional behaviour) are required. The type and amount of data required to develop the model depend primarily on the level and complexity required for modelling the river system. The essential data required for the development of a one-dimensional, steady state YRWQM are available for the Yarra River. They include mean daily streamflow data at various gauging stations of the Yarra River and its tributaries, cross sectional data and stage-discharge relationships at these gauging stations, diversions flows, water quality measurements at various locations of the river and effluent data from STPs. These data were collected from various organisations. From these data, several flow events which covered a period from 1992 – 1997 were selected to model the Yarra River water quality, and used throughout this thesis.

This chapter first describes the Yarra River catchment with respect to land use conditions, water storage and diversions of the catchment, and the locations of streamflow gauging stations, water quality measurement points and STPs. Analysis on streamflow, water quality and effluent data are discussed then. The methodology in the selection of representative events is presented, together with a detailed water quality data analysis. Finally, an overview of the previous water quality modelling work on the catchment is then presented.

3.2 Description of Yarra River Catchment

The Yarra River catchment is located in the eastern part of Victoria (Australia) and is shown in Figure 3.1. The Yarra River flows from east to west, and has a total catchment area of 4,044 square kilometres, and a stream course of 245 kilometres from

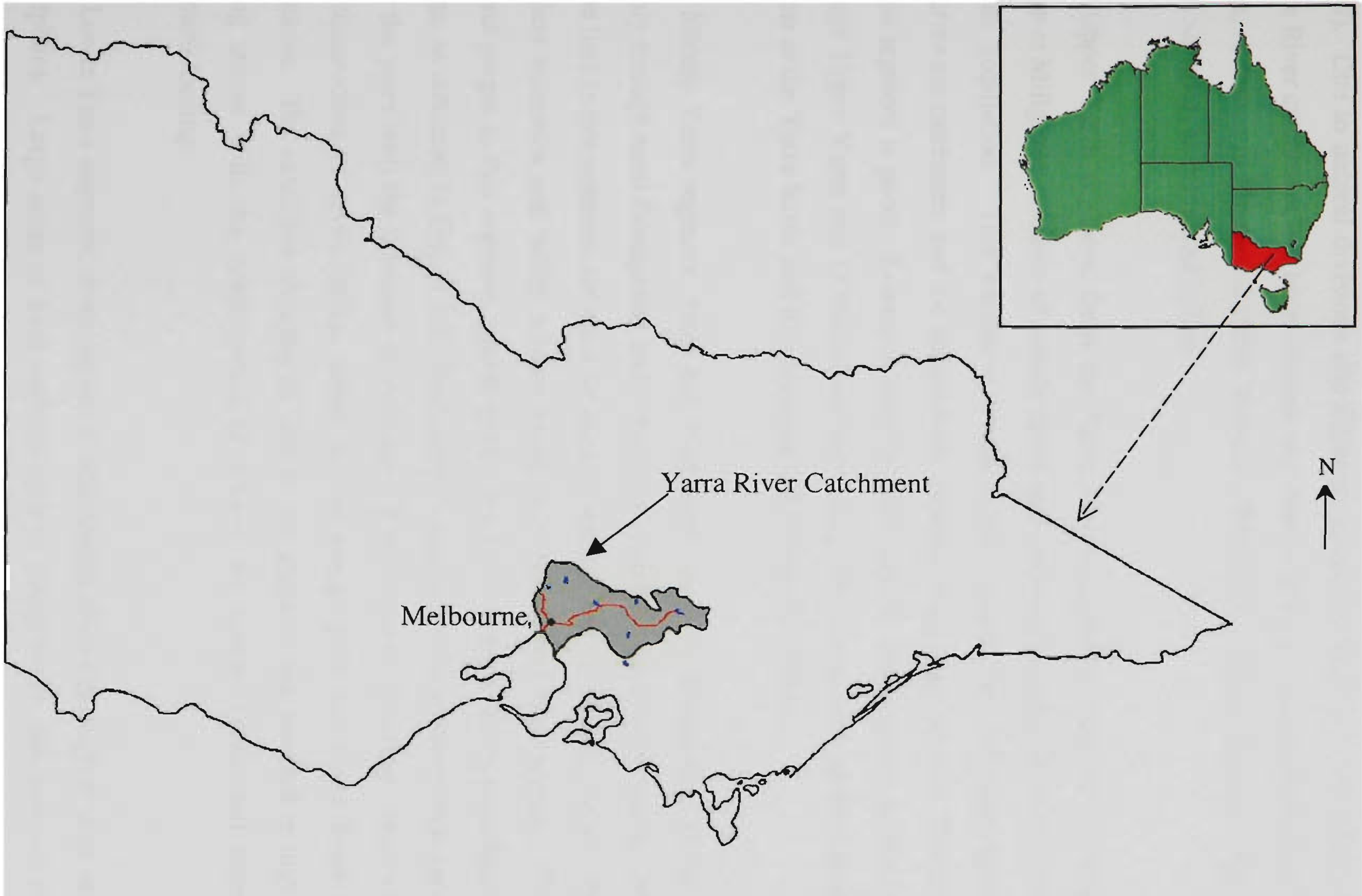


Figure 3.1 Yarra River Catchment

the Yarra Ranges National Park to the estuary at Port Phillip Bay, as shown in Figure 3.2. The Yarra River catchment consists a series of sub-catchments based on major tributaries, whose total length amounts to 1,800 kilometres, as reported in EPA Victoria (1999). Due to natural divisions and different landuse activities in the catchment, the Yarra River catchment has been divided into three segments: Upper, Middle and Lower Yarra, as shown in Figure 3.2 (EPA Victoria, 1999). Each segment has its own distinct characteristics, as discussed below.

The Upper Yarra segment, from the Yarra Ranges National Park to the Warburton Gorge at Millgrove, consists of mainly dense and extensive forested area with minimum human population. This extensive forested cover minimises overland runoff, and therefore the catchment and the stream bank experience the least erosion. Water quality in this segment is good. Extensive harvesting of potable water occurs in this segment through Upper Yarra and O'Shannassy reservoirs, which have altered the natural flow regime of the Yarra River and its tributaries (EPA Victoria, 1999).

The Middle Yarra segment, from the Warburton Gorge to Warrandyte Gorge, flows mainly through rural floodplains and valleys with limited urban development. Majority of the land in this segment are used for agricultural purposes (Gardner, 1994). The river gradient decreases and valley widens as the river approaches downstream. There are several gorges in this segment, which restrict the flow of the river, in particular Yering Gorge, as indicated in Figure 3.2. Both public and private works have been carried out over the years with the intention of reducing the incidence of flooding. Works include the construction of levee banks, drains and de-snagging of the Yarra River and its tributaries. The extensive clearing of land in this segment has resulted in high runoff during storms with the consequence of erosion on stream banks and increase in sediment loading.

The Lower Yarra segment, downstream of Warrandyte, flows through mainly urbanised floodplains. Large areas of hard surfaces such as paved roads and concrete channels have increased the runoff into streams and resulted in both high velocity and water level. These rapid velocities damage the streambed and banks. Much original

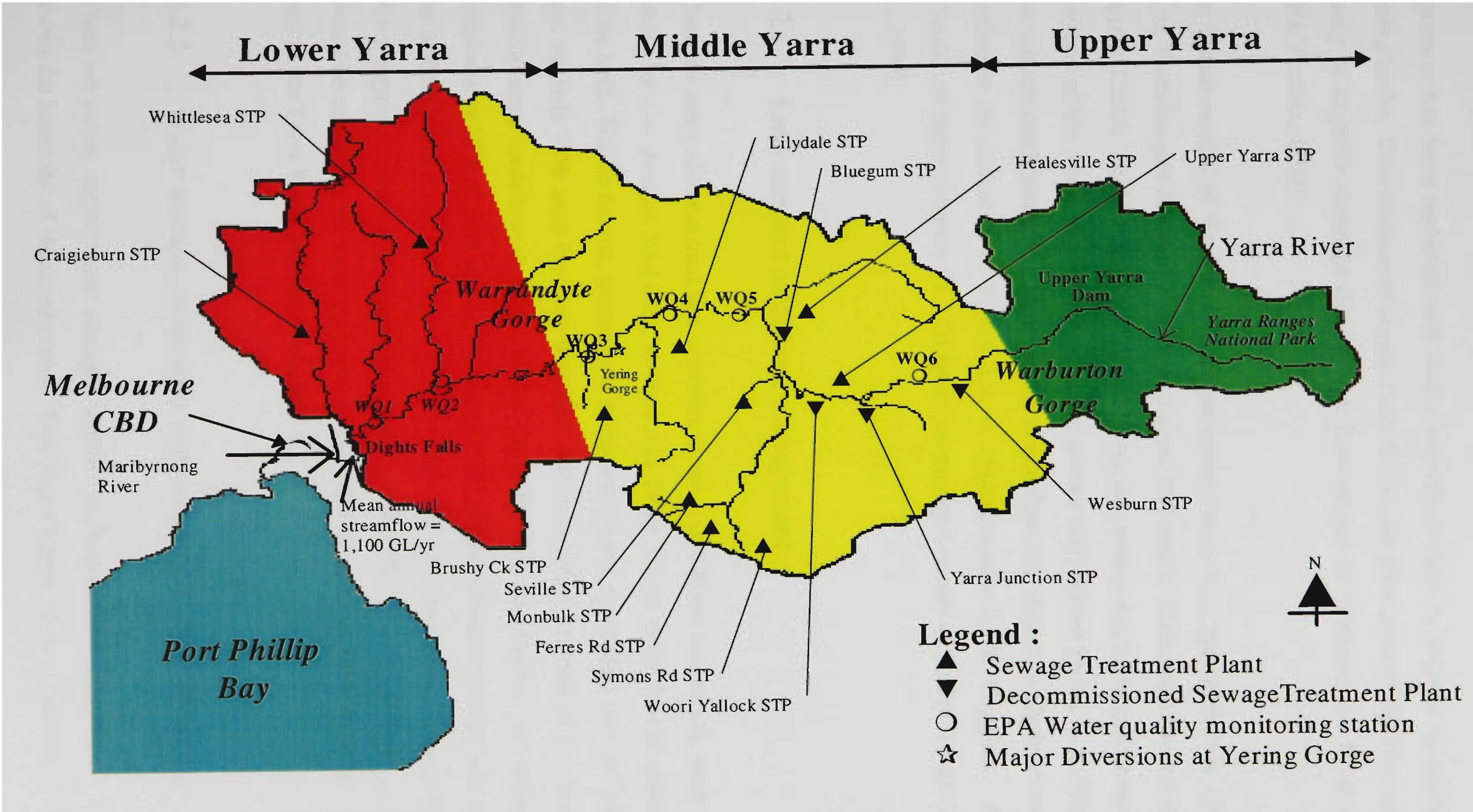


Figure 3.2 Details of Yarra River Catchment

vegetation has been replaced with exotic vegetation such as willows to stabilise the eroding banks. Downstream of Dights Falls (DFS), the river channel has been modified mainly to improve drainage during high flow period to minimise the flood damage (EPA Victoria, 1999).

The annual rainfall of the Yarra River catchment varies from approximately 1,600 mm in the Yarra Ranges National Park area to about 600 mm in Melbourne Central Business District (CBD). High streamflows occur during late winter months (e.g. October) in the upstream of the Yarra River catchment, and during early winter period (e.g. July) in the downstream catchment. The mean annual streamflow just above the Maribyrnong River confluence as indicated in Figure 3.2, is approximately 1,100 GL/year. A major diversion of approximately 51.3 GL/year (on average) occurs at Yering Gorge (Davis *et al.*, 1998).

3.2.1 Land use and catchment management

There are many diverse land use activities in the Yarra River catchment, each segment having its own distinct land uses, as shown in Table 3.1, to allow for the optimum use of the land. Native forests, pasture and horticulture have dominated the total catchment land use with 83% used for these activities, while the urban, recreation and commercial developments contribute to 17%. In the Upper Yarra River segment, the native forests contribute to 20% of land use, while the agricultural and horticulture activities dominate the Middle Yarra River catchment with a significant 22% land use. In the Lower Yarra River segment, urban and pasture land uses cover 15% of the total land use each. It should be noted that all these percentages in Table 3.1 are based on the total catchment area of the Yarra River and its tributaries.

3.2.2 Water storage, diversions and streamflow measurements

There are seven major water storages within the Yarra River catchment. Figure 3.3 shows the locations of these reservoirs. The Upper Yarra, Maroondah and O'Shannassy receive natural streamflow, while Sugarloaf, Silvan, Greenvale and Yan Yean are offstream storages, filled by diverting water from the other storages and river (Melbourne and Metropolitan Board of Works, 1985). These reservoirs provide sufficient carry-over storage and ensure that the water is available during dry periods. Diversions occur mainly in Upper Yarra and Middle Yarra segments for urban water supply. A major diversion point is at Yering Gorge (Figure 3.2) to Sugarloaf offstream storage (Personal communication with Ian Watson of Melbourne Water Corporation, 1999) and has been considered in the overall streamflow balance of this study. It should be noted that Figure 3.3 shows other storages operated by Melbourne Water Corporation, which are the Thomson and Toorourrong reservoirs. However, it should be noted that these two reservoirs are not in the Yarra River catchment.

Table 3.1 Land Uses in Yarra River Catchment

	AREA OF LAND USE ACTIVITY						
	Urban	**Recreation**	**Commercial**	**Tree Cover**	**Horticulture**	**Pasture**	**Total**
UPPER YARRA							
Land use area (km^2)	0	0	0	795.59	0.02	14.69	810.3
Land use area as a % of total catchment area	***0***	***0***	***0***	***20***	***0***	***0.36***	***20.0***
MIDDLE YARRA							
Land use area (km^2)	35.54	3.28	2.29	671.75	77.14	817.9	1607.9
Land use area as a % of total catchment area	***0.85***	***0.08***	***0.057***	***16.6***	***1.91***	***20.23***	***40***
LOWER YARRA							
Land use area (km^2)	587.84	70.22	18.28	339.81	7.37	602.97	1626.5
Land use area as a % of total catchment area	***14.54***	***1.737***	***0.452***	***8.403***	***0.182***	***14.9***	***40***
TOTAL CATCHMENT							
Land use area (km^2)	622.39	73.5	20.58	1807.15	84.53	1435.56	4043.7
Land use area as a % of total catchment area	***15.39***	***1.82***	***0.51***	***44.7***	***2.09***	***35.5***	***100***

Extracted and modified from (Thomas and Cummings, 1994)

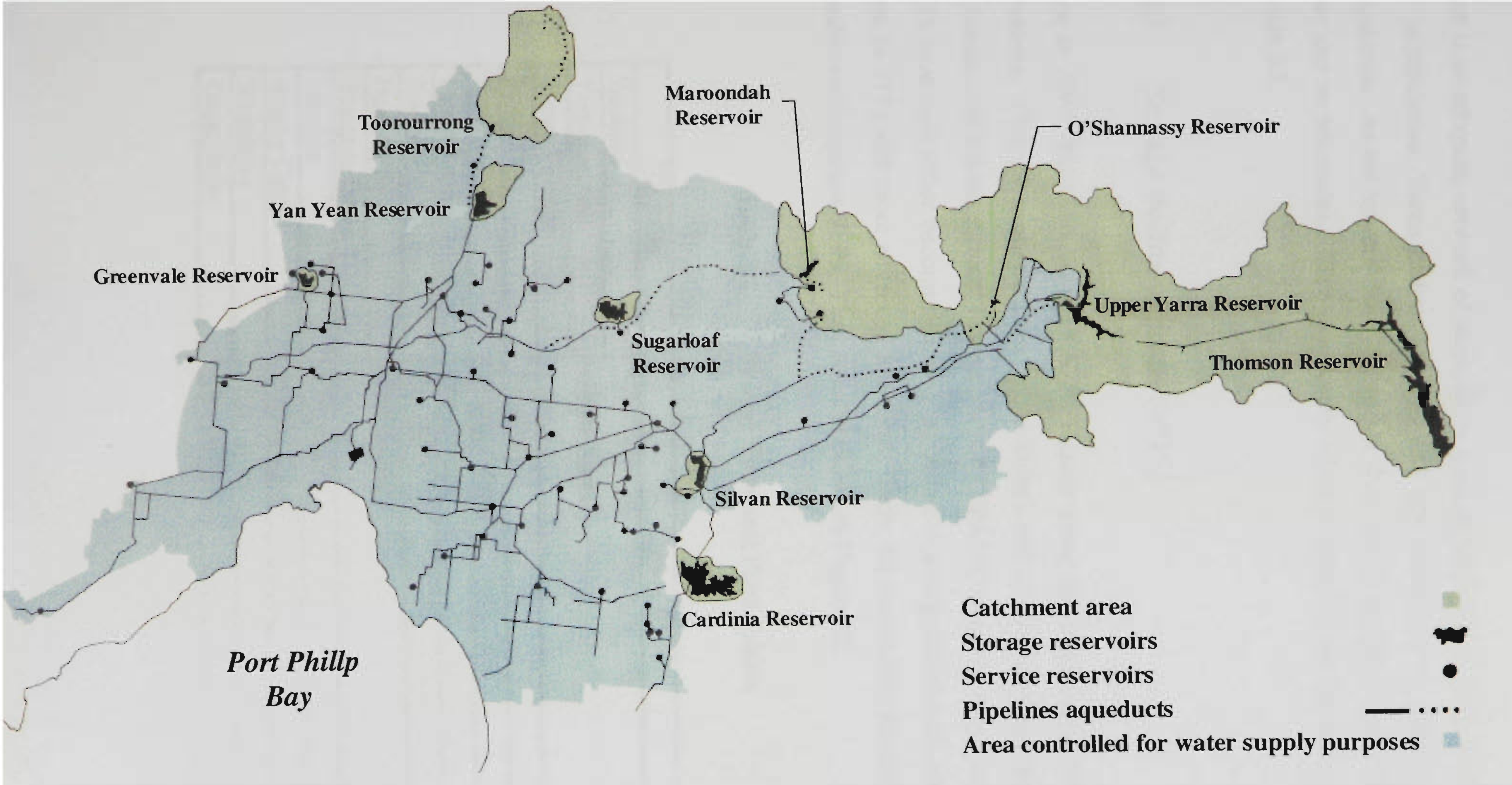


Figure 3.3 Water Storages in Yarra River Catchment

(Extracted from Melbourne and Metropolitan Board of Works, 1985)

There is an adequate network of streamflow gauging stations covering the Yarra River and its tributaries. These stations are operated and monitored by Melbourne Water Corporation. As can be seen from Figure 3.4, there are 19 gauging stations in the Yarra River and its tributaries. Data availability of these gauging stations are presented in Section 3.3.

3.2.3 Sewage treatment plants (STPs)

Prior to 1997, there had been 13 STPs located along the main Yarra River and its tributaries. They are mainly situated in the Middle and Lower Yarra segments. Due to introduction of stricter effluent license limits by EPAVIC over the years, some of these STPs have been either decommissioned, upgraded or amalgamated with other STPs to form 10 STPs with more advanced treatment. Table 3.2 shows how the old STPs were transformed to current STPs. They are also shown in Figure 3.5.

Table 3.2 Sewage Treatment Plant Profile

Old STPs	Current STPs
Wesburn	Upper Yarra
Yarra Junction	
Woori Yallock	
Monbulk	Monbulk
Symons Road	Symons Road
Ferres Road	Ferres Road
Seville	Seville
Healesville	Healesville
Bluegum Drive	
Lilydale	Lilydale
Brushy Creek	Brushy Creek
Whittlesea	Whittlesea
Craigieburn	Craigieburn

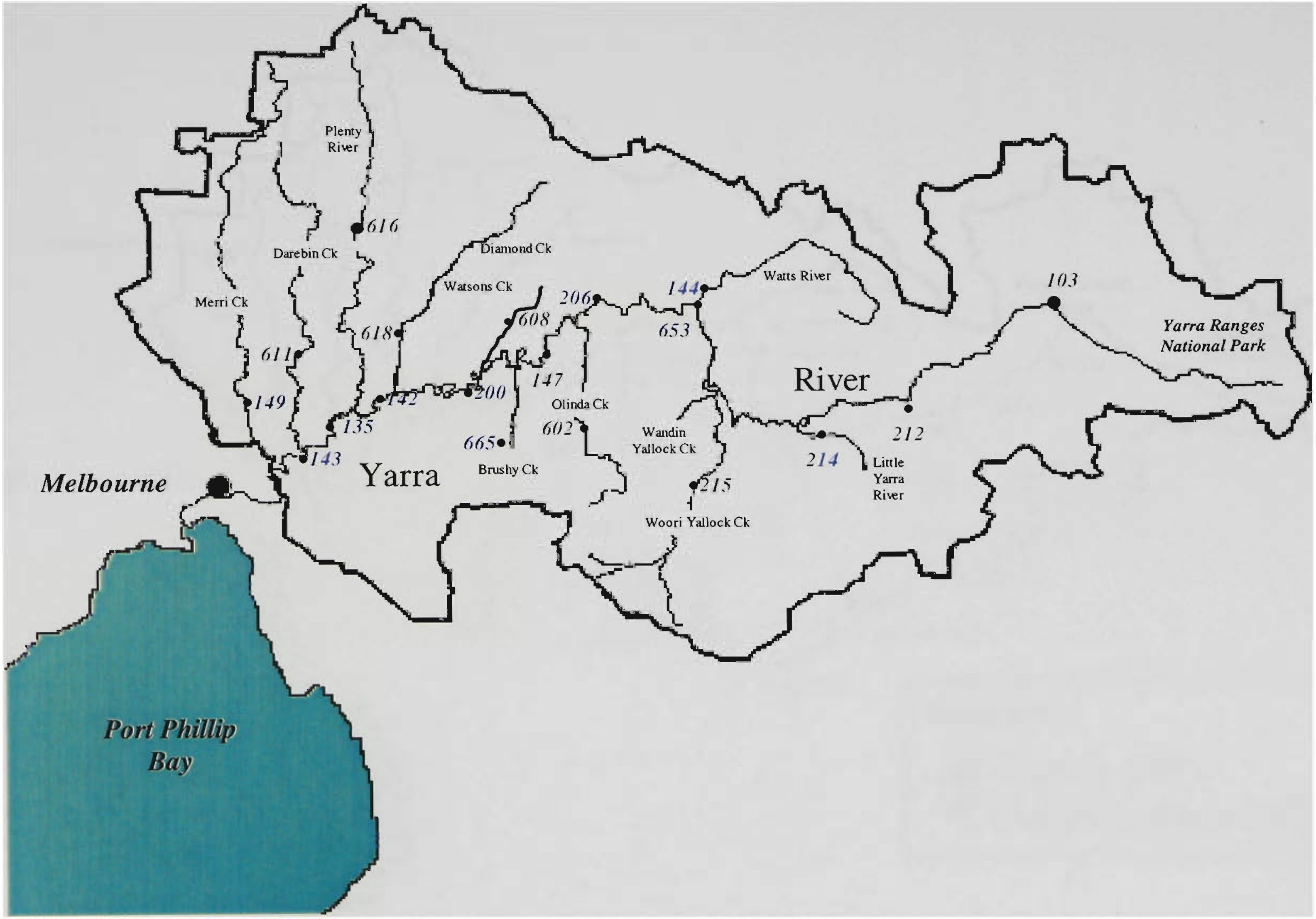


Figure 3.4 Gauging Stations Along Yarra River and Tributaries

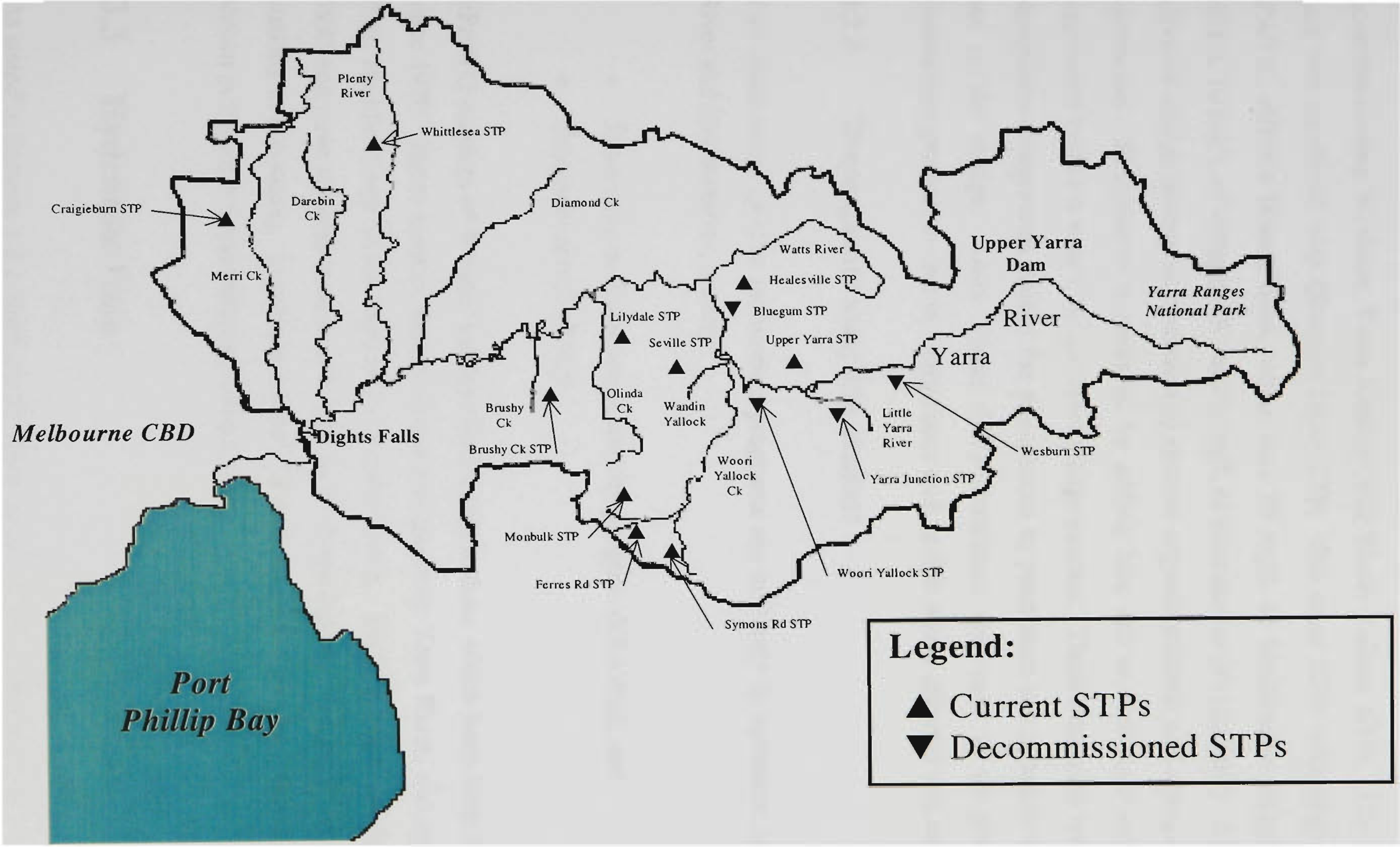


Figure 3.5 Sewage Treatment Plants on Yarra River and Tributaries

As can be seen from Table 3.2, the new Upper Yarra STP was built after decommissioning Wesburn, Yarra Junction and Woori Yallock STPs. The Healesville plant was combined with Bluegum Drive STP. The other STPs were upgraded to the EPAVIC effluent license limit, which was 10 mg/L of biochemical oxygen demand (BOD), 10 mg/L of nitrogen (N) and 1 mg/L of phosphorus (P) (10/10/1). All STPs use activated sludge biological treatment to remove organic material and nitrogen from the wastewater. Phosphorus is removed by adding iron salt or aluminum sulfate to the wastewater before it enters the activated sludge process. These chemicals react with the phosphorus compounds caused the phosphorus to precipitate out as solids which form part of the sludge. Ultra Violet (UV) irradiation was used in all plants as the disinfection process, except in Craigieburn and Seville where chlorine was used.

3.2.4 Water quality sampling stations

Two main water quality monitoring programs are currently in operation in the Yarra River and its tributaries, namely

- Environment Protection Authority program (EPAWQ), and
- StreamWatch (SWWQ)

EPAWQ consists of 6 water quality monitoring stations which have been in operation since 1970. These stations mainly situated in the main Yarra River, and spread evenly from Middle Yarra River segment to Lower Yarra. SWWQ began its operations in 1993 and have stations stretching from Upper Yarra segment to Lower Yarra, and with stations in tributaries. Locations of both EPAWQ and SWWQ monitoring stations are shown in Figure 3.6 and referenced Table 3.3.

3.3 Hydraulic Data

As stated in Section 3.2.2, there are 19 gauging stations on the main Yarra River and its tributaries. The streamflow data and the rating curves of these gauging stations were used to select flow events for this study and modelling of hydraulics respectively.

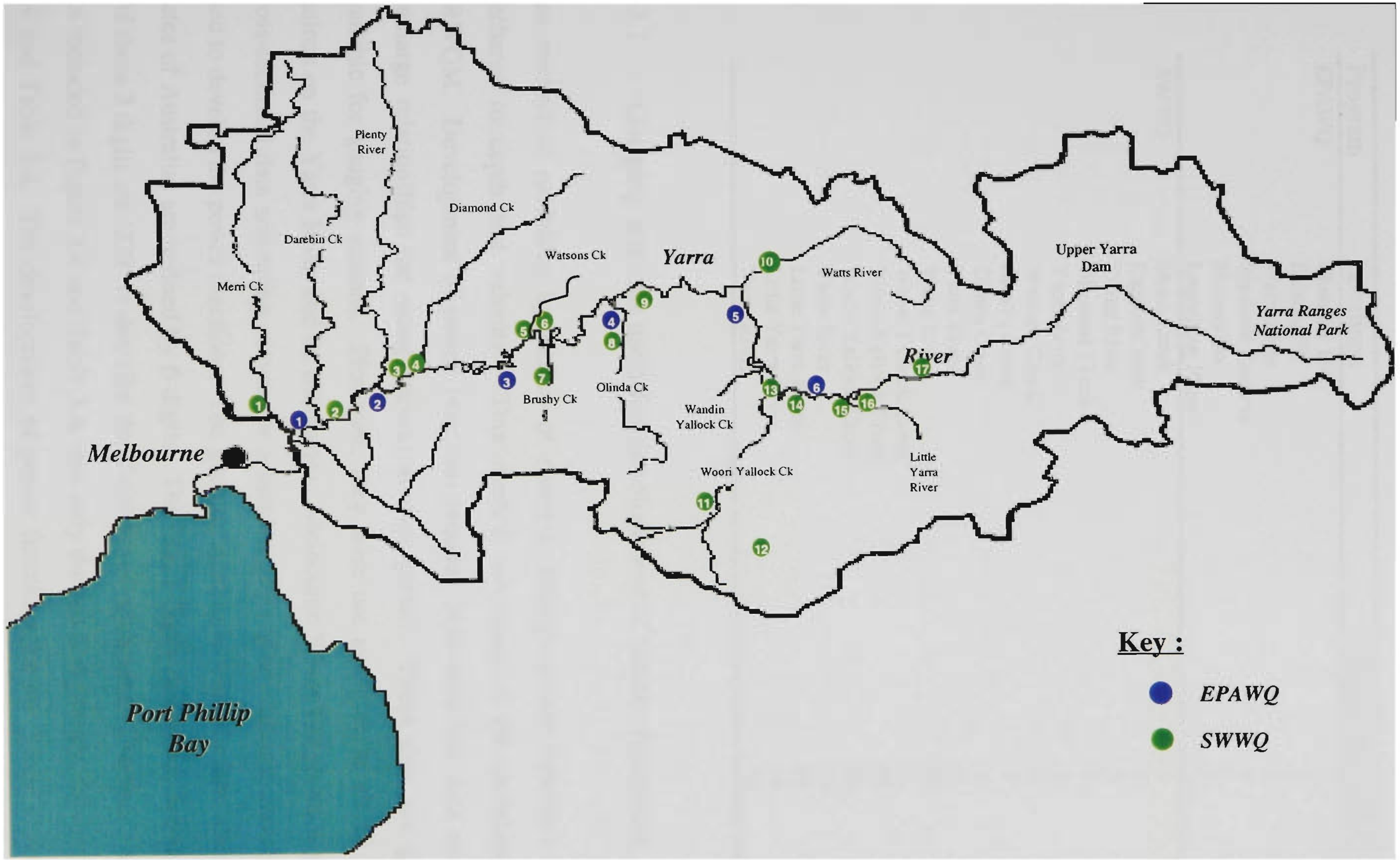


Figure 3.6 EPAWQ and SWWQ Water Quality Monitoring Stations

Table 3.3 Locations of Water Quality Sampling Stations

Program	Locations	Station No. in Fig. 3.6
EPAWQ	Chandler Hwy	1
	Banksia St.	2
	Warrandyte	3
	Spadonis Reserve	4
	Healesville	5
	Launching Place	6
SWWQ	Merri Creek	1
	DarebinCreek	2
	Plenty River	3
	Diamond Creek	4
	Yarra River	5
	Watsons Creek	6
	Brushy Creek	7
	Olinda Creek	8
	Yarra River	9
	Watts River	10
	Woori Yallock Creek	11
	Woori Yallock Creek	12
	Woori Yallock Creek	13
	Yarra River	14
	Little Yarra River	15
	Little Yarra River	16
	Yarra River	17

3.3.1 Gauging stations used for development of power functions

One method of modelling hydraulics of rivers is through power functions relating discharge to depth and velocity. This method was used in the development of YRWQM. Development of power functions requires cross-sectional data and stage-discharge relationships (or rating curves) at river points. These data are generally available for gauging stations. However, they were not available at all 19 gauging stations on the Yarra River and its tributaries. Melbourne Water Corporation provided cross-sectional data and rating curves for a number of gauging stations, and they were used to develop the power functions. The gauging stations in Victoria (and also in other states of Australia) are defined by 6 digits. The first 3 digits define the drainage basin and these 3 digits are '229' to describe the Yarra River catchment. The first 3 digits are not included in Figure 3.4 and Table 3.4, and only the last 3 digits are shown in Figure 3.4 and Table 3.4. The development of power functions for the Yarra River and its tributaries are discussed in Section 4.4.2.1.

Table 3.4 Gauging Stations Used for Development of Power Functions

Gauging Station Name	Station No. in Figure 3.4
Yarra River	
Yarra River at Doctors Creek	103
Yarra River at Millgrove	212
Yarra River at Yarra Grange	653
Yarra River at Yering Gorge	147
Yarra River at Warrandyte	200
Yarra River at Fitzsimons Lane	142
Yarra River at Banksia Street, Heidelberg	135
Yarra River at Chandler Hwy, Kew	143
Tributaries	
Olinda Creek at Lilydale	602
Merri Creek at Fitzroy	149

3.3.2 Streamflow

Streamflow gauging stations that were used to select flow events for water quality modelling of Yarra River and its tributaries are summarised in Table 3.5, and also shown in Figure 3.4. Daily mean streamflow data are available at each gauging station, and for some gauging stations from as early as 1934 (Rural Water Commission of Victoria, 1984). However, in this study, streamflow data from 1992-1997 (both inclusive) were used, since the effluent data from STPs, which were required to develop YRWQM, were available only from 1992-1997 and the data for these STPs were not available since 1997 for these STPs, at the time of this study. The data beyond 1997 could not be used, since some STPs have been modified after 1997. Table 3.5 also shows minimum, maximum and mean daily streamflows at the gauging stations for the period 1992-1997. Similar to Table 3.4, only the last 3 digits of gauging station numbers shown Table 3.5.

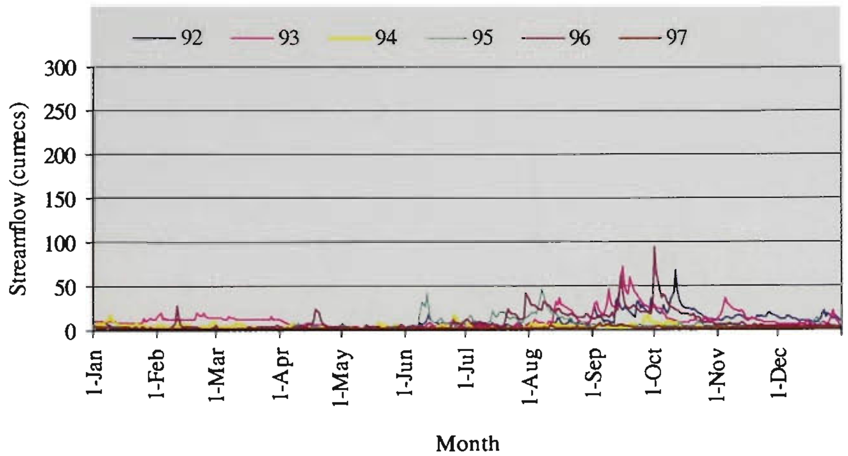
The mean daily streamflows of the Yarra River from 1992 to 1997 were analysed to identify the seasonal flow patterns and is shown in Figure 3.7. The gauging stations from upstream and downstream are shown in Figure 3.7 in that order. Generally, flow is observed to increase in the downstream direction. These plots in general, showed 2 distinct flow periods:

- November to June - 'low flow period'
- July to October - 'high flow period'

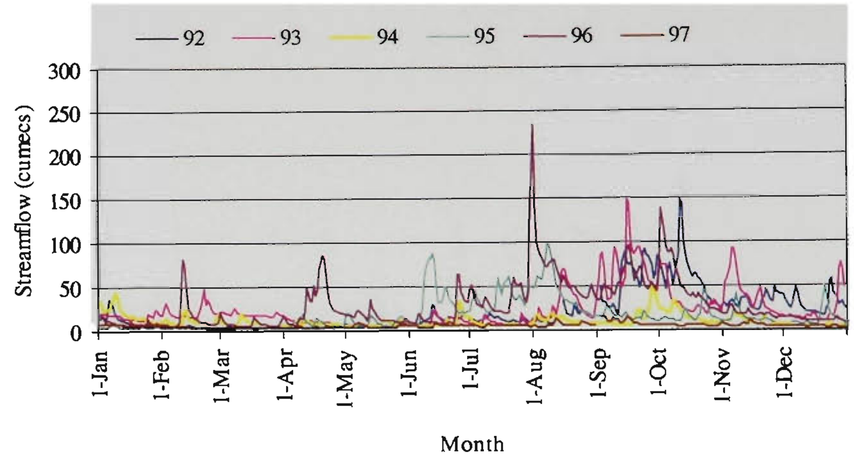
Table 3.5 Summary of Streamflow Range (m^3/s) for Period 1992 –1997

Location	Station No. in Fig. 3.4	Minimum	Maximum	Mean
Yarra River				
Yarra River at Millgrove	212	1.19	93.60	6.91
Yarra River at Yarra Grange	653	2.42	234.13	18.14
Yarra River at Yarra Glen	206	2.55	140.87	20.57
Yarra River at Yering Gorge	147	2.27	200.81	19.10
Yarra River at Warrandyte	200	0.02	209.10	0.81
Yarra River at Fitzsimons Lane	142	3.13	195.46	21.89
Yarra River at Banksia Street, Heidelberg	135	2.90	270.29	25.65
Yarra River at Chandler Hwy, Kew	143	0.10	203.32	1.25
Tributaries				
Little Yarra River at Yarra Junction	214	0.53	21.61	1.86
Woori Yallock Creek upstream of Warburton Highway	215	0.31	62.93	3.66
Watts River downstream of Maroondah dam	144	0.11	48.53	2.18
Olinda Creek at Lilydale Lake	602	0.34	8.91	0.24
Brushy Creek at Mooroolbark	665	0.44	4.39	0.11
Watsons Creek at Kangaroo Ground South	608	0.36	11.77	0.26
Diamonds Creek at Eltham	618	2.41	41.21	20.85
Plenty River at Mernda	616	0.44	51.46	0.59
Darebin Creek at Ivanhoe	611	1.59	16.51	22.63
Merri Creek at Fitzroy	149	0.01	45.42	1.28

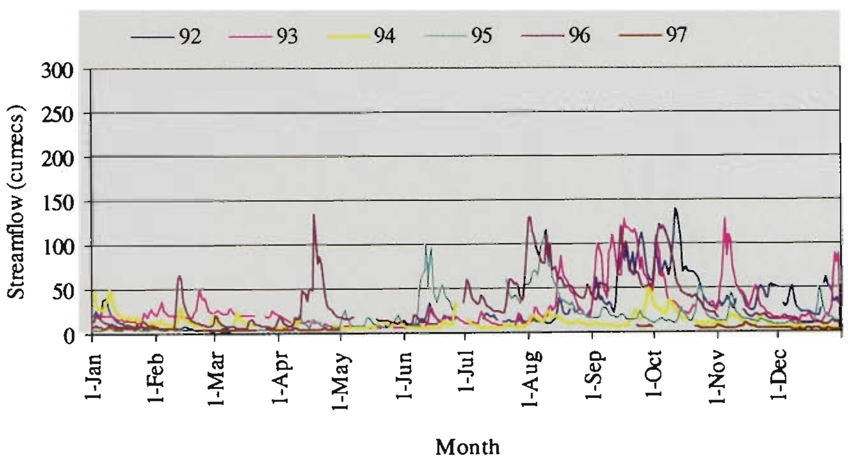
This distinctive seasonal flow behaviour may be suitable to apply alternative efficient management strategies such as seasonal effluent discharge programs. Such an investigation was conducted and described in Chapter 7. A summary of the range of flow for low and high flow is shown in Table 3.6 and 3.7 respectively, considering the flow records of 1992 -1997. Similar to Tables 3.4 and 3.5, only the last 3 digits of gauging station numbers are shown in Tables 3.6 and 3.7.



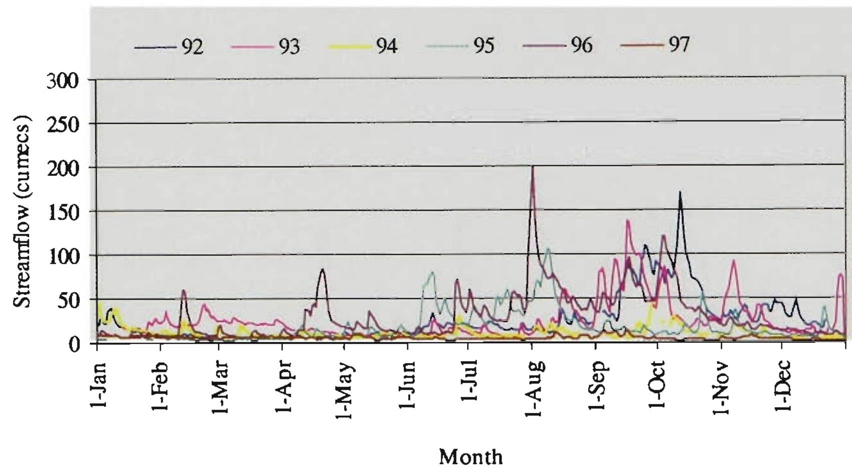
(a) Station 212



(b) Station 653

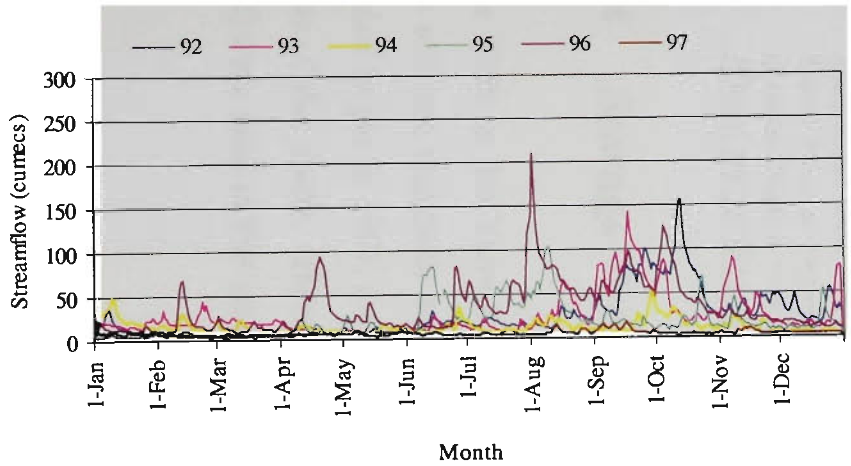


(c) Station 206

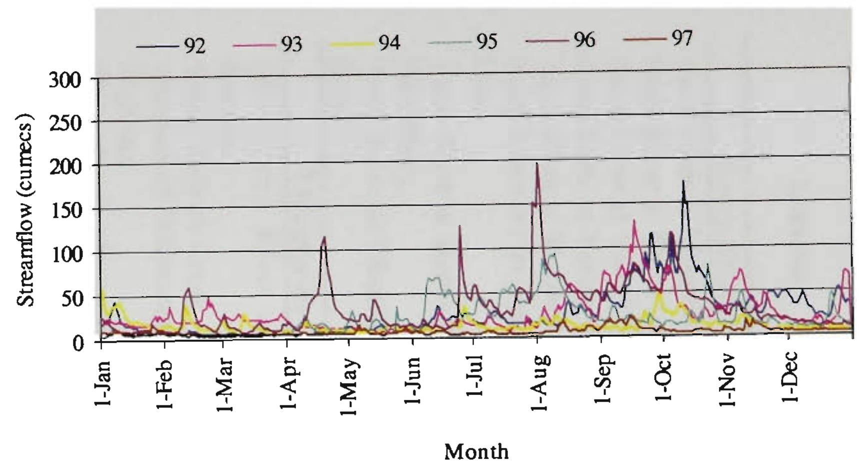


(d) Station 147

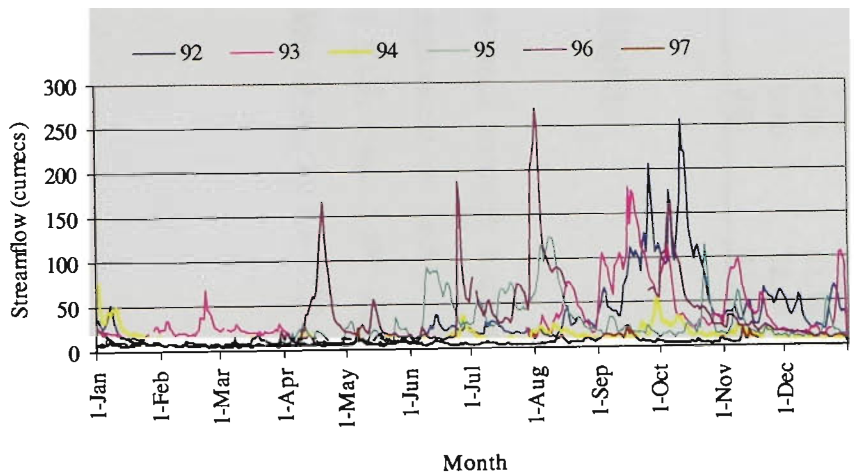
Figure 3.7 Mean Daily Streamflow



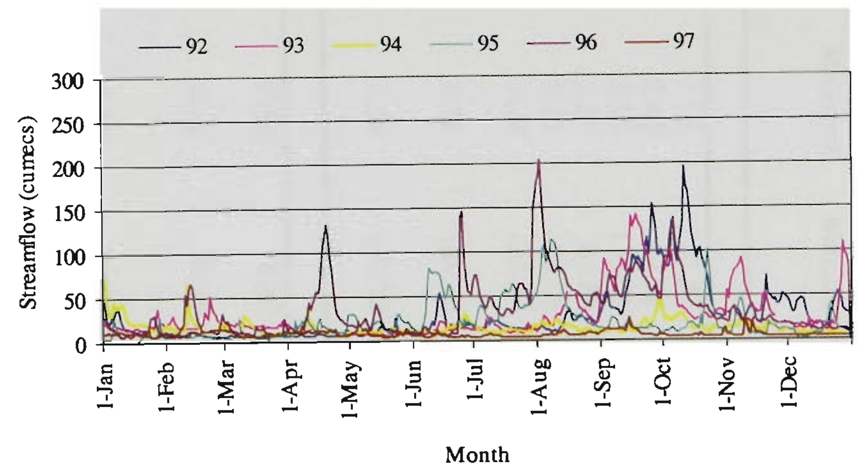
(e) Station 200



(f) Station 142



(g) Station 135



(h) Station 143

Figure 3.7 (Cont.) Mean Daily Streamflow

Table 3.6 Summary of Low Flow Period Streamflow Range (m^3/s) for 1992 –1997

Location	Station No. as shown in Fig. 3.4	Minimum	Maximum	Mean
Yarra River				
Yarra River at Millgrove	212	1.19	42.37	4.44
Yarra River at Yarra Grange	653	2.42	87.32	11.42
Yarra River at Yarra Glen	206	2.55	133.52	12.72
Yarra River at Yering Gorge	147	2.27	83.89	12.26
Yarra River at Warrandyte	200	2.52	93.88	13.80
Yarra River at Fitzsimons Lane	142	3.20	125.27	14.99
Yarra River at Banksia Street, Heidelberg	135	3.77	188.30	16.63
Yarra River at Chandler Hwy, Kew	143	1.59	146.20	15.24
Tributaries				
Little Yarra River at Yarra Junction	214	0.53	8.89	1.52
Woori Yallock Creek upstream of Warburton Highway	215	0.31	38.19	2.33
Watts River downstream of Maroondah dam	144	0.00	29.99	1.16
Olinda Creek at Lilydale Lake	602	0.05	4.74	0.16
Brushy Creek at Mooroolbark	665	0.02	4.39	0.08
Watsons Creek at Kangaroo Ground South	608	0.00	9.53	0.16
Diamonds Creek at Eltham	618	0.02	41.21	0.49
Plenty River at Mernda	616	0.08	51.46	0.30
Darebin Creek at Ivanhoe	611	0.10	12.44	1.03
Merri Creek at Fitzroy	149	0.01	45.42	0.95

3.4 Sewage Treatment Plant Data

The STPs in the Yarra River catchment have been through a transition period to meet the effluent standards set by EPAVIC, as discussed in Section 3.2.3. The transition period began in 1997, and the effluent data of STPs since 1997 were not available at the time of this study. Therefore, the effluent data from the 13 STPs prior to 1997 (Table 3.2) were used in this study.

Table 3.7 Summary of High Flow Period Streamflow Range (m^3/s) for 1992-1997

Location	Station No. as shown in Fig. 3.4	Minimum	Maximum	Mean
Yarra River				
Yarra River at Millgrove	212	1.40	93.60	10.37
Yarra River at Yarra Grange	653	3.55	234.13	27.58
Yarra River at Yarra Glen	206	3.41	140.87	30.68
Yarra River at Yering Gorge	147	2.40	200.81	28.22
Yarra River at Warrandyte	200	2.41	209.10	30.61
Yarra River at Fitzsimons Lane	142	3.13	195.46	31.33
Yarra River at Banksia Street, Heidelberg	135	2.90	270.29	38.11
Yarra River at Chandler Hwy, Kew	143	3.15	203.32	32.99
Tributaries				
Little Yarra River at Yarra Junction	214	0.72	21.61	2.32
Woori Yallock Creek upstream of Warburton Highway	215	0.76	62.93	5.52
Watts River downstream of Maroondah dam	144	0.00	48.53	3.60
Olinda Creek at Lilydale Lake	602	0.10	8.91	0.36
Brushy Creek at Mooroolbark	665	0.05	3.89	0.16
Watsons Creek at Kangaroo Ground South	608	0.00	11.77	0.38
Diamonds Creek at Eltham	618	0.03	37.47	1.25
Plenty River at Mernda	616	0.01	30.17	1.00
Darebin Creek at Ivanhoe	611	0.12	16.51	1.55
Merri Creek at Fitzroy	149	0.03	41.62	1.77

The effluent data were supplied by Yarra Valley Water Pty Ltd. Data were from 1992 to 1997, with sampling done on a fortnightly basis. A summary of the mean effluent concentration (based on available data) for BOD, total nitrogen (TN) and total phosphorus (TP) discharged from the 13 STPs are shown in Table 3.8. The effluent from STPs were discharged either to the Yarra River or its tributaries. Note that these effluent data were measured to meet the EPAVIC compliance and not for the modelling exercise described in this thesis or for any other river water quality modelling work.

Table 3.8 Summary of Mean Effluent Concentration (mg/L)

STP	BOD	TN	TP
Wesburn	4.6	6.9	14.1
Yarra Junction	3.7	6.4	9.8
Woori Yallock	3.7	7.2	10.7
Monbulk	4.8	5.3	16.5
Symons Road	5.2	2.4	5.7
Ferres Road	4.1	1.6	4.1
Seville	5.1	7.0	13.6
Healesville	2.4	3.4	15.8
Bluegum Drive	5.5	7.8	16.5
Lilydale	10.0	1.0	15.8
Brushy Creek	3.7	3.7	11.8
Whittlesea	5.6	2.4	12.0
Craigieburn	5.4	3.7	9.1

3.5 River Water Quality Data

As discussed in Section 3.2.4, both EPAWQ and SWWQ data were considered for use in this study. Both EPAWQ and SWWQ monitoring programs sampled data on a fortnightly basis and was based on grab sampling. Similar to the STP effluent data, water quality data were not measured for the purpose of water quality modelling. Although the EPAWQ monitoring began in 1974, data records were not complete until 1992. The SWWQ monitoring began in 1993, but also contains missing patches of data. Therefore, EPAWQ data were used in the development of YRWQM, in particular for calibration and verification of the model parameters, since they are almost complete from 1992-1997, and also located on the main Yarra River. However, SWWQ data were used to estimate headwater, incremental and tributary concentrations for YRWQM. A detailed analysis was conducted on water quality of the selected flow events for use in YRWQM. This analysis provided a greater understanding of the governing water quality processes in the Yarra River. Selection of flow events is discussed in Section 3.6, while the water quality analysis of the selected flow events is presented in Section 3.7.

3.6 Selection of Flow Events

Selection of several flow events was required for the development of YRWQM. These events were used throughout this study for model assembly (Chapter 4), pre-calibration uncertainty/sensitivity analysis (Chapters 5), model calibration, verification and post-calibration uncertainty/sensitivity analysis (Chapter 6), and assessment of river water quality management strategies (Chapter 7). These events were also used to compute incremental, headwater and point load flows and concentrations for use in YRWQM.

Flow events were selected based on the data availability on flow in the river and tributaries, effluent data from STPs and water quality measurements. As discussed earlier in Section 3.5, water quality data used in this study for calibration were from EPAWQ monitoring stations, and therefore the flow events selected were primarily depended on EPAWQ monitoring period. Since a steady state water quality model (i.e. QUAL2E) was used in this study, the events with 'fairly' constant streamflow over a period equivalent to the travel time from Upper Yarra Dam (UYD) to Dights Falls (DFS) were selected. The travel time from UYD to DFS is about 3 days. Water quality data at the EPAWQ monitoring stations were measured once a fortnight and therefore the flow events selected based on steady river flow criterion should also coincide with the water quality measurements. These selected events should also have STP effluent data. In some cases, effluent data from STPs data were unavailable and interpolation of missing data was required. This interpolation of effluent concentration does not affect the overall input since effluent quality does not fluctuate widely (Personal communication with Julie Baud of Yarra Valley Water Pty Ltd, 2000). The overall flow event selection procedure is listed below.

1. First, flow events were selected by considering 'fairly' steady flows in the river and tributaries over a period of 3 days.
2. The flow events obtained from Step 1 were matched with water quality measurement data. The events that coincided were then considered in Step 3.

3. Finally, the events obtained from Step 2 were matched with the effluent data from STPs. If data from STPs were missing for these events, they were interpolated from available data.

Ten flow events as shown in Table 3.9 were selected based on the above criteria, and as expected, they were all low flow events. Events were defined by their dates. Although the 10 selected events were low flows, two groups were considered in subsequent modelling work, as 'lower range' and 'higher range'. This grouping was based on an arbitrary flow of 10 m^3/s at the gauging station 143 (which is close to DFS). Flows below 10 m^3/s were considered as 'lower range' flows and vice versa. The flow events above 40 m^3/s at gauging station 143 were not steady over a period of 3 days.

Table 3.9 Flow Events Selected

Event dates	Event ID	Flow* (m^3/s)	Comments
Mar 18-20, 1992	18/3/92	6.1	
Nov 1-3, 1995	3/11/95	7.4	
Feb 18-20, 1992	18/2/92	7.5	Lower range flows
Jun 11-13, 1996	11/6/96	7.8	
Apr 2-4, 1997	2/4/97	9.4	
Jan 21-23, 1992	21/1/92	9.5	
Sep 6-8, 1994	6/9/94	11	
Oct 19-21, 1993	19/10/93	25	Higher range flows
Nov 2-4, 1992	2/11/92	31	
Jul 26-28, 1995	26/7/95	40	

* Flow at gauging station 143

3.7 Water Quality Data Analysis

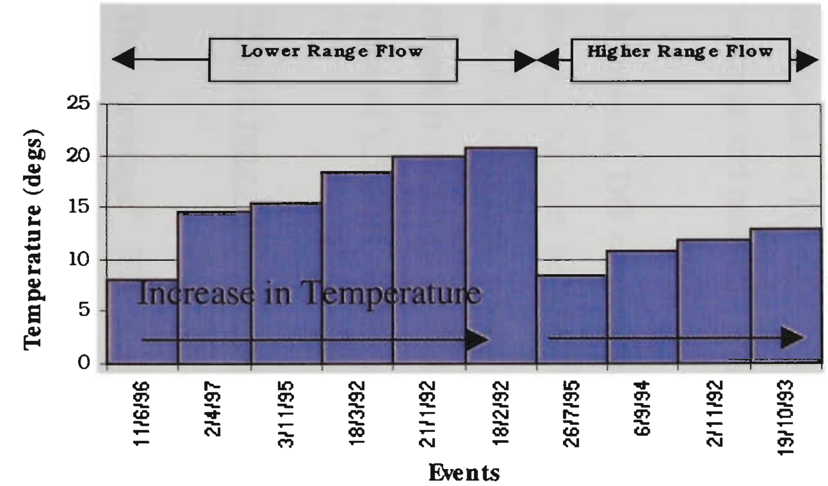
Three main processes control water quality in rivers namely physical, biological and chemical, as discussed in Section 2.2. Depending on the assimilative capacity of the river system (i.e. the ability of the river to digest incoming pollutants) and different flow conditions in the river, certain processes dominate water quality in the river. The selected flow events were analysed to understand the processes affecting Yarra River water quality for flows up to about 40 m^3/s at DFS. As different flow conditions result in different water quality, the analysis was done relative to the two flow conditions of 'lower range' and 'higher range' (Table 3.9). Note that due to data limitations, this analysis was done only to give a preliminary indication of the water quality trend in Yarra River.

Three major nonconservative water quality constituents, namely dissolved oxygen (DO), TN and TP were considered in Yarra River water quality modelling in this study. DO was considered, since it is the major indicator of river health. Both TN and TP were found to be the major nutrients which pose threats to environmental health of the Yarra River, and the Port Phillip Bay with undesirable algal blooms (Harris *et al.*, 1996 and EPA Victoria, 1999). Turbidity was also considered in this analysis, since it measures the level of sedimentation activity in the river, which in turn affects water quality.

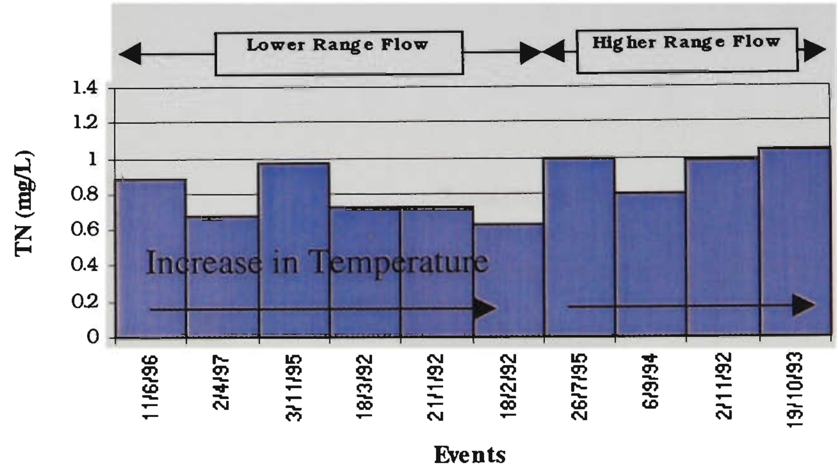
For both 'lower range' and 'higher range' flows, the events were ranked based on increase in temperature. The 'average' TN, TP and DO concentrations of the flow events are shown in Figure 3.8, in which the flow events are arranged according to increase in temperature. Since the increase in turbidity is not a function of the temperature, the average turbidity concentrations of flow events are plotted in Figure 3.9 according to increase in flow in the flow events. The 'average' water quality concentrations of each flow event in Figures 3.8 and 3.9 refer to the mean of the six EPAWQ monitoring measurements along Yarra River.

<u>'Lower range' flow events</u>

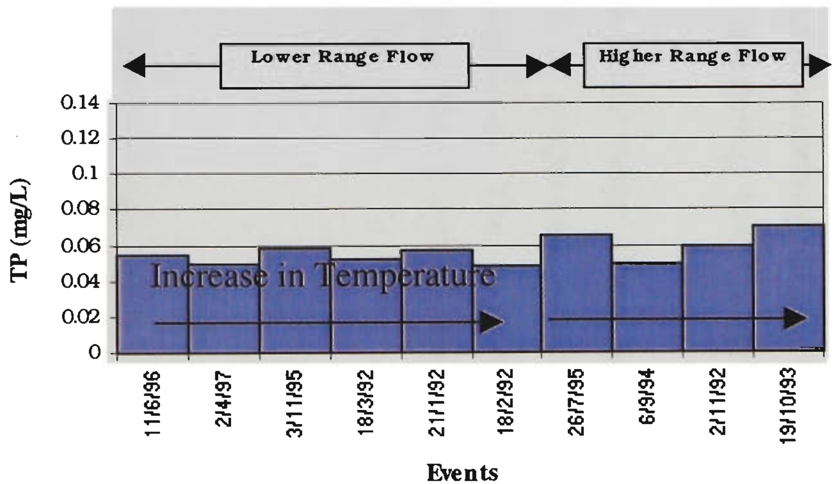
In general, TN concentration is lower for 'lower range' flows as compared to 'higher range' flows. The 'average' TN concentration ranged between 0.63-0.97 mg/L (Figure 3.8), which exceeds the recommended value of 0.1-0.75 mg/L specified in ANZECC (1999) in some cases. The TN concentration generally decreases as temperature increases, as indicated in Figure 3.8 except for the event on 3/11/95. This event had the highest TN concentration but with moderately high temperature. Analysis of turbidity



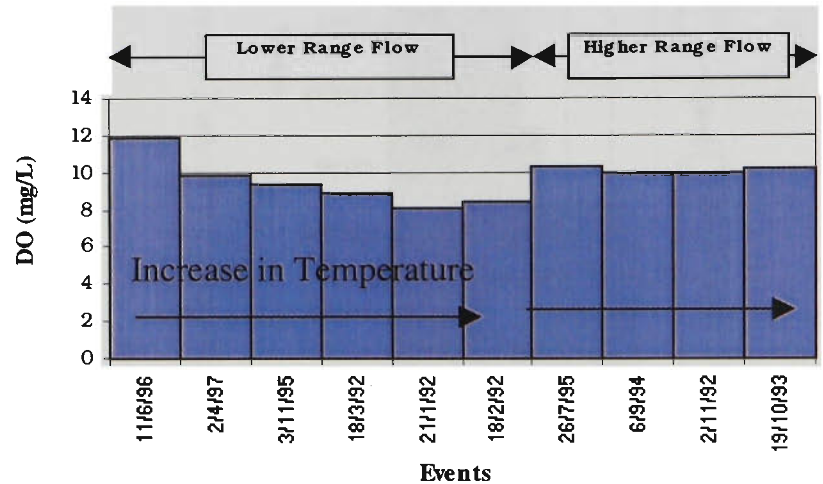
(a) Temperature



(b) TN



(c) TP



(d) DO

Figure 3.8 Average Water Quality Concentrations in Figure 3.9 indicated that turbidity level for this event was the highest compared to other 'lower range' events. This suggests that the high TN for this event were due to low settling and re-suspension activity.

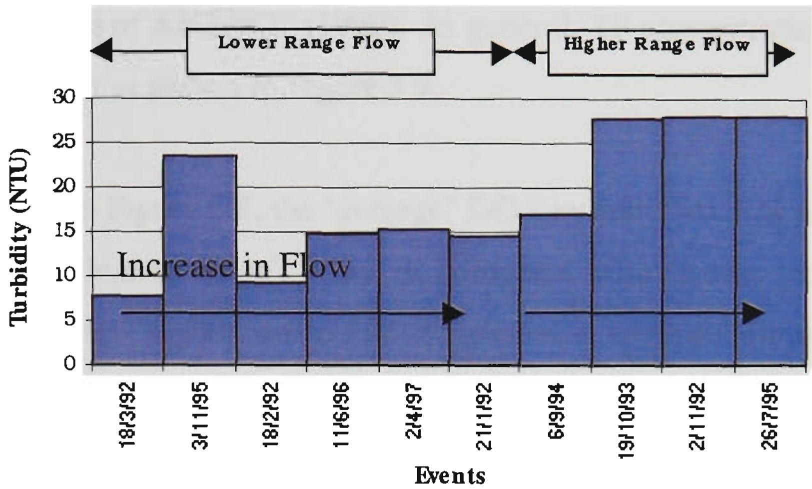


Figure 3.9 Average Turbidity Concentrations

The 'average' TP concentrations of all 'lower range' flow events are almost the same concentration, as shown in Figure 3.8. The range for the average TP concentration was 0.05-0.06 mg/L, which was within the recommended guidelines of 0.01–0.10 mg/L of ANZECC (1999). This relatively small variation in 'average' TP concentrations compared to TN could be due to soil particles adsorbing phosphorus more readily than nitrogen and retaining phosphorus in river bed during 'lower range' flows.

In general, DO concentration is lower for 'lower range' flows than for 'higher range' flows. As can be seen from Figure 3.8, as temperature increases, the biochemical activity increases which caused DO to decrease. The 'average' DO concentration ranged from 8.1 to 11.9 mg/L for 'lower range' flows, which is above the recommended value of 7.5 mg/L at 25 degrees of ANZECC (1999).

<u>'Higher range' flow events</u>

The 'average' TN concentration of 'higher range' flow events exceeds the recommended value of ANZECC (1999) in almost all cases. The trend observed for 'lower range' flows (i.e. as temperature increased, TN decreased) was not found with ‘higher range’ flows. The turbidity plays a greater role for high TN concentrations as indicated in Figure 3.9.

The ‘average’ TP concentration ranged from 0.03-0.07 mg/L, which was within the recommended values of ANZECC (1999). In general, TP concentrations are increasing as turbidity increases as shown in Figure 3.9.

As can be seen from Figure 3.8, the ‘average’ DO concentration was relatively constant and better during ‘higher range’ flows as compared with ‘lower range’ flows. The reason could be that ‘higher range’ flows generally enhance dilution capacity and reaeration. The ‘average’ DO concentration ranged from 7.7-10.3 mg/L, which was above the recommended values of ANZECC (1999).

3.8 Previous Water Quality Modelling on Yarra River Catchment

Two “in-house” computer models are available to model water quality in the Yarra River. They are the ‘Yarra Catchment Model’ developed by EPAVIC in 1995 (EPA Victoria, 1995b) and ‘FILTER’ developed in 1999 by Argent and Mitchell (1999).

The ‘Yarra Catchment Model’ was developed to simulate the water quality of Yarra River when uniform effluent license limit (10/10/1) was launched on Yarra River STPs. This model was also used as a tool to foresee any potential impact on proposed effluent license limits. However, the model does not adequately simulate the processes such as decay and transport (i.e. advection and dispersion) mechanisms. These shortcomings in the model can potentially underestimate the assimilative capacity of the river to recover its balance due to incoming pollution. This in turn overestimate the levels of effluent license limit that would actually required.

The ‘FILTER’ model was developed to estimate the amount of pollutant load reaching Port Phillip Bay. This model was not specifically developed for Yarra River water quality simulation. There were 8 catchments including the Yarra River catchment considered in 'FILTER'. This model mainly use to estimate the nonpoint source load carried from each of these catchments into Port Phillip Bay, which could not be used as a simulation modelling tool to study various management strategies for Yarra River.

3.9 Summary

The Yarra River catchment located in Victoria (Australia) is a valuable asset to all Melbourne residents. Over the years, the river quality has declined due to population increase. However, the water quality management strategies introduced by the Environment Protection Authority of Victoria (EPAVIC), such as control of point source discharges from sewage treatment plants (STPs) have improved river water quality.

Due to different landuse activities in the catchment, the Yarra River has been characterised by three segments, namely Upper, Middle and Lower Yarra. The Upper Yarra segment has the pristine water quality since the area is protected for the sole purpose of harvesting water. The Middle Yarra segment has high agricultural activities, which cause major diffused source pollution in terms of high nutrient runoff. The urban Melbourne area has the poorest water quality and is known as the Lower Yarra segment.

An extensive data analysis was conducted on streamflow, effluent from STPs and river water quality data for 3 main purposes:

- Identification of seasonal flow regimes
- Flow event selection
- Understand the water quality processes in the river under low flow conditions

The analysis of mean daily streamflow data indicated that variation in flow regime exists between seasons. It was found that periods between November to June can be grouped as low flows, while periods between July to October as high flows.

Ten low flow events were selected for this study, by considering steady state flows over a period of 3 days (travel time of the catchment from upstream to downstream was considered to be 3 days), considering the availability of river water quality and STP effluent data. Although these events were low flows, they were arbitrarily subdivided into 'lower range' and 'higher range' flows. These events were to be used for the assembly of the Yarra River Water quality Model (YRWQM), calibration and verification of YRWQM, and uncertainty and sensitivity analysis of model parameters of YRWQM.

A preliminary water quality data analysis was conducted for 'lower range' and 'higher range' flow events using EPAWQ data. It was found that dissolved oxygen (DO) concentration decreases as temperature increases under 'lower range' flows, whereas it was relatively constant under 'higher range' flows. Total nitrogen (TN) concentration for 'lower range' flow events generally decreases as temperature increases, but not for 'higher range' flow events. It was found that the variation of total phosphorus (TP) between flow events was relatively small. This could be perhaps due to soil particles adsorbing phosphorus more readily compared to nitrogen. Furthermore, in general as turbidity increases, TP concentration increases.

CHAPTER 4

DEVELOPMENT OF YARRA RIVER WATER QUALITY MODEL USING QUAL2E

4.1 Introduction

Successful model development involves the following steps.

1. Data collection and selection of modelling software
2. Assembly of the model
3. Pre-calibration uncertainty and sensitivity analysis of model parameters
4. Calibration and verification of model parameters
5. Post-calibration sensitivity analysis of model parameters

After selecting the appropriate software, the assembly of the river water quality model can be considered as the major step in the overall model development. It involves the collection of the required data, the transformation of certain raw data into a format that can be used by the river water quality modelling software and then the entry of those data into the computer software. The raw data transformation includes the development of hydraulic power functions from rating tables at streamflow gauging stations, the derivation of decay rates from water quality data, and the selection of the appropriate reaeration methods considering depth and velocity relationships.

The assembly of the Yarra River Water Quality Model (YRWQM) using QUAL2E software (Brown and Barnwell, 1987) is described in this chapter. This chapter begins with a description of mathematical algorithms used in QUAL2E. The discretisation of the Yarra River for use in QUAL2E is then discussed. Finally, the inputs required for development of YRWQM are presented.

4.2 QUAL2E River Water Quality Modelling Software

Some QUAL2E river water quality modelling applications and the reasons for selecting QUAL2E for development of YRWQM were discussed in Section 2.4.4. The mathematical equations and processes considered by QUAL2E are discussed in this section. As stated in Section 2.4.4, QUAL2E is an one-dimensional river water quality modelling software. It uses the general conservation principle of hydrologic and mass balance to determine flow and water quality concentration of a river system. The use of QUAL2E for a hypothetical river network is shown conceptually in Figure 4.1. The actual hypothetical stream is shown in Figure 4.1a, with two tributaries. This river system is discretised into several reaches (i.e. R1 to R5) as shown in Figure 4.1b. In this case, the reach division is based on the confluence points. There can be other criteria that can be used for reach division, such as the location of water quality sampling points and sewage treatment plants (STPs). These reaches are further subdivided into smaller computational elements *(i)*, as shown in Figure 4.1b, with one small grid enlarged. Each element *(i)* is considered completely mixed and the transport of matter from element *(i)* to *(i+1)* is sequentially linked via the advection and dispersion transport mechanisms. The advection mechanism models the **movement** of the mass as it flows downstream, while the dispersion mechanism accounts for the **spreading** of the mass in lateral, longitudinal and vertical directions.

Computations in QUAL2E are performed from upstream to downstream, allowing for mixing at junctions. The conservation principle of hydrologic and mass balance, which determines the flow and mass at the downstream end of an element is shown below in a descriptive sense:

Hydrologic balance

Downstream flow = (upstream flow) + (inflows from lateral flows, tributaries and STPs within the element) – (outflow from diversions from the element)

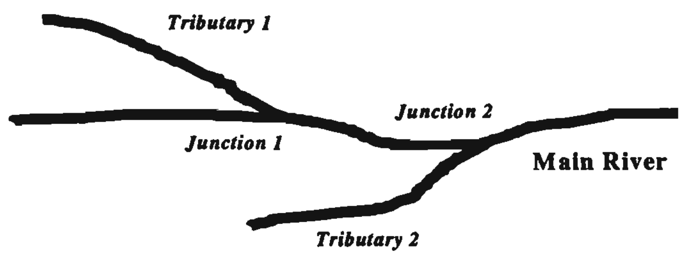


Figure 4.1a Hypothetical Stream

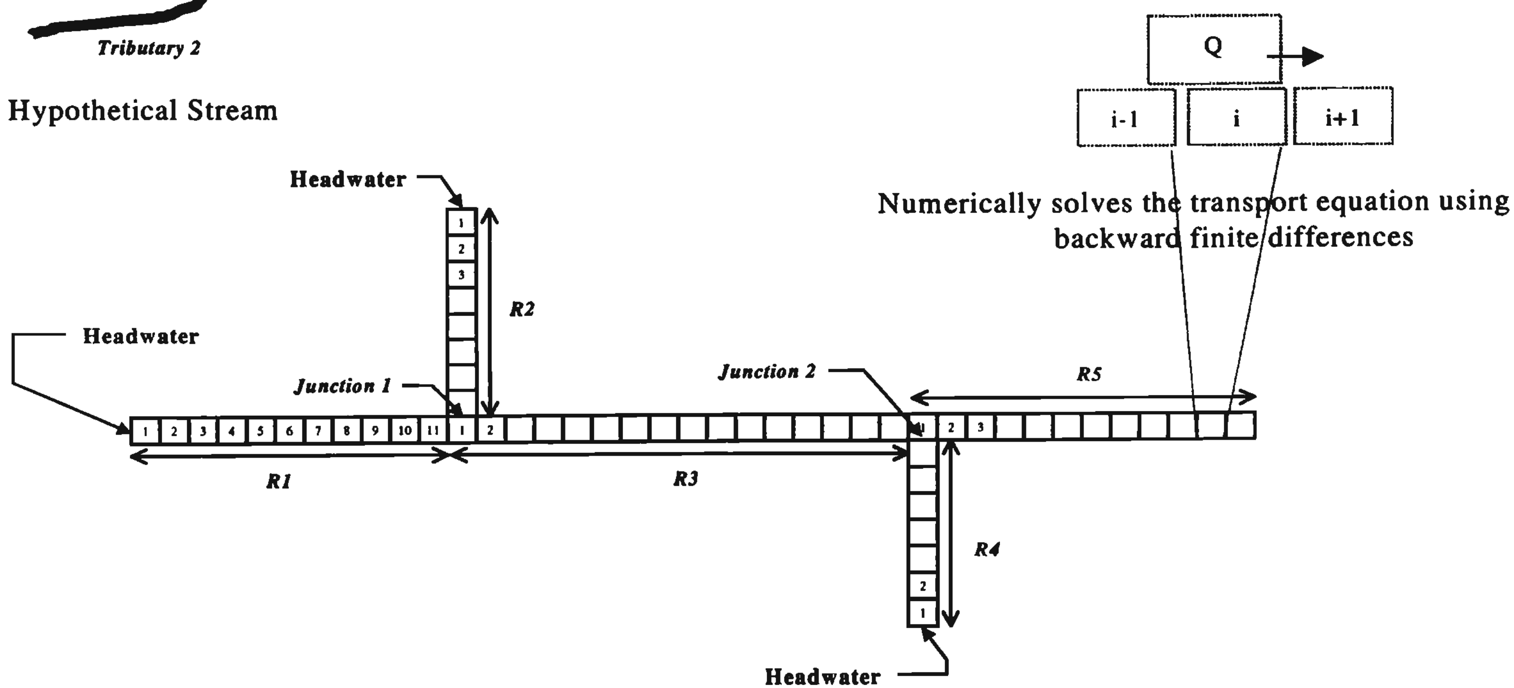


Figure 4.1b Discretised stream network

Figure 4.1 QUAL2E Conceptual Representation of Stream Network

Mass balance

> Mass accumulation = (upstream mass) + (mass carried from lateral flows, tributaries and STPs within the element) – (mass loss from diversions from the element) + (source within the element) – (sinks within the element).

In determining the final mass out of the element, the internal reaction of mass is considered via sources and sinks of mass. Sources are processes which can increase the quantity of a water quality mass, while sinks are processes which decreases the quantity. For example, reaeration and nitrification represent a source and a sink respectively for dissolved oxygen (DO).

By considering the transport (movement and spreading), the physical and internal changes in mass components along the longitudinal direction in the river, the 'transport–mass' formulation given in Equation 4.1 can be used to determine the changes in water quality concentration in rivers. QUAL2E determines the accumulation or loss of water quality concentration by solving this one-dimensional mass transport equation using backward implicit finite difference method.

$$\underbrace{(A\,dx)\frac{\partial c}{\partial t}}_{\substack{\text{Change in}\\ \text{concentration}\\ \text{with time}}} = \underbrace{\frac{\partial\left(AD\frac{\partial c}{\partial x}\right)}{\partial x}\partial x}_{\substack{\text{Transport}\\ \text{mechanism}\\ \text{advection}}} - \underbrace{\frac{\partial(AUc)}{\partial x}\partial x}_{\substack{\text{Transport}\\ \text{mechanism}\\ \text{dispersion}}} + \underbrace{(A\,dx)\frac{dc}{dt}}_{\substack{\text{Internal}\\ \text{source and}\\ \text{sinks}}} + \underbrace{S}_{\substack{\text{External}\\ \text{pollutant}\\ \text{input/output}}} \tag{4.1}$$

where		
	A	is the cross sectional area (m^2)
	c	is the concentration of pollutant (mg/L)
	t	is the time (day^{-1})
	x	is the distance (m)
	D	is the dispersion coefficient ($m^2\ day^{-1}$)
	U	is the mean velocity ($m\ day^{-1}$)

S is the external pollutant input/output (kg/d)

The first two terms on the right hand side of Equation 4.1 deal with advection and dispersion transport mechanisms and these two transport mechanism have been briefly discussed earlier in this section. The third term deals with the internal source and sink processes of various water quality concentrations in the river system. Any additional input/output of various water quality concentrations is accounted for in the last term, for example effluent discharge from STPs.

The $\frac{dc}{dt}$ term of internal sources and sinks represents the rate of change of individual water quality constituents and is modelled in QUAL2E using first order equations, where the rate of change is directly proportional to the concentration. These internal sources and sinks are due to physical and biochemical processes in the river system (Section 2.2). The reduction or increase in the concentration of water quality constituent is governed by their individual decay rates. The nonconservative water quality constituents that were considered in this study are all nitrogen (N) and phosphorus (P) forms, carbonaceous biochemical oxygen demand (CBOD) and DO. Their rates of change in concentration (which also represent their corresponding processes) are determined through Equations 4.2-4.9. In all these equations, both physical and biochemical processes are considered for each nonconservative water quality constituents.

$$\frac{d(Org-N)}{dt} = \alpha_1 \rho G - k_3 N - k_1 N \tag{4.2}$$

$$\frac{d(NH_3)}{dt} = k_3 N - k_4 A + \frac{k_5}{d} - F \alpha_1 \mu G \tag{4.3}$$

$$\frac{d(NO_2)}{dt} = k_4 A - k_6 C \tag{4.4}$$

$$\frac{d(NO_3)}{dt} = k_6 C - (1-F) \alpha_1 \mu G \tag{4.5}$$

$$\frac{d(Org-P)}{dt} = \alpha_2\rho G - k_7P - k_8P \quad 4.6$$

$$\frac{d(Diss-P)}{dt} = k_7P + \frac{k_9}{d} - \alpha_2\mu G \quad 4.7$$

$$\frac{d(CBOD)}{dt} = -k_{10}B - k_{11}B \quad 4.8$$

$$\frac{d(DO)}{dt} = k_2(Os-O) + (\alpha_3\mu - \alpha_4\rho)G - k_{10}B - \frac{k_{12}}{d} - k_4A\alpha_5 - k_6O\alpha_6 \quad 4.9$$

where:

k_1= Org-N settling (day^{-1})

k_2= Reaeration rate coefficient (day^{-1})

k_3= Hydrolysis of org-N to NH_3 (day^{-1})

k_4= Biological oxidation of NH_3 (day^{-1})

k_5= Benthos rate for NH_3 (mg m^{-2} day^{-1})

k_6= Oxidation rate of NO_2 (day^{-1})

k_7= Org-P decay rate (day^{-1})

k_8= Org-P settling rate (day^{-1})

k_9= Benthos rate for diss-P (mg m^{-2} day^{-1})

k_{10}= Deoxygenation rate for CBOD (day^{-1})

k_{11}= Settling rate for CBOD (day^{-1})

k_{12}= Sediment oxygen demand (mg m^{-2} day^{-1})

ρ= Algae respiration rate (day^{-1})

d = Mean stream depth (m)

G = Algae biomass concentration (mg/L)

N = Org-N concentration (mg/L)

A = NH_3 concentration (mg/L)

C = NO_2 concentration (mg/L)

P = Org-P concentration (mg/L)

B = CBOD concentration (mg/L)

O = DO concentration (mg/L)

O_s = Saturation concentration of DO (mg/L)

α_1= Nitrogen algae biomass (mg mg^{-1})

α_2 = Phosphorus algae biomass (mg mg^{-1})

α_3=Rate of O_2 production of algae photosynthesis (mg mg^{-1})

α_4 = rate of O_2 uptake by algae respiration (mg mg^{-1})

α_5 = O_2 uptake by NH_3 oxidation (mg mg^{-1})

α_6 = O_2 uptake by NO_2 oxidation (mg mg^{-1})

μ = Local specific growth of algae (day^{-1})

F = Fraction of algae nitrogen uptake from NH_3(mg $m^{-2}day^{1}$)

Equations 4.2-4.5 consider both physical and biochemical processes of water quality constituents within the N group (i.e. Org-N, NH_3, NO_2 and NO_3) via their decay rates. The concentration of org-N is governed by the processes of microbial transformation (from Org-N to NH_3) and settling, determined by their decay rates of k_3 and k_1 respectively. The NH_3 concentration is increased based on microbial transformation from Org-N (via the decay rate k_3), decreased based on nitrification process (via the decay rate k_4) and increased via benthos release, which has been accumulated from settled organic matter (via the decay rate k_5). The NO_2 concentration level is increased due to conversion from NH_3 (via the decay rate k_4), and reduced due to the second stage of nitrification to NO_3 (via the decay rate k_6). In the above and subsequent discussions on Equations 4.2 to 4.9, sources/sinks due to algae have been omitted, because algae is not considered in this study, as explained later in Section 4.4.3.

Equations 4.6-4.7 are used to determine the rate of change in concentration of the phosphorus group (i.e. Org-P and Diss-P). The Org-P concentration can be reduced through conversion to Diss-P (via the decay rate k_7) and settling (via the decay rate k_8). The Diss-P concentration is increased due to conversion from Org-P (via the decay rate k_7), and benthos release (via the decay rate k_9).

The CBOD concentration can be reduced through two processes, namely biological decay (via the decay rate k_{10}) and settling (via the decay rate k_{11}), as shown in Equation 4.8.

The increase in DO concentration can be through oxygen input from atmosphere (via the decay rate k_2). The decrease in DO concentration is resulted from various biochemical processes within the water column and river bed, such as biological decay of CBOD (via the decay rate k_{10}), decaying of organic matter in the benthos layer (via the decay rate k_{12}) and nitrification (via the decay rates k_4 and k_6), as shown in Equation 4.9.

The decay rates are temperature dependent. These decay rates are adjusted in QUAL2E using the following expression

$$k_T = k_{20}\,\theta^{(T-20)} \qquad 4.10$$

where k_T is adjusted decay rate at temperature T°C

k_{20} is decay rate at 20°C

θ is temperature correction factor

The temperature correction factors are different for different decay rates and they are discussed in Section 4.4.1.

4.3 Discretisation of Yarra River and its Tributaries

The model boundary of YRWQM was considered from the Upper Yarra Dam (UYD) to Dights Falls (DFS), as indicated in Figure 4.2. Upstream of UYD is protected from human interaction and has high water quality (EPA Victoria, 1999), and therefore was not considered in YRWQM. Downstream of DFS is affected by tidal influence, and the nutrient levels in this area of the river are delivered from both Yarra River and estuary of Port Phillip Bay. Modelling of such conditions requires extensive data and these data were not available at the time of the model construction, and therefore, downstream of DFS was not considered in YRWQM.

The Yarra River and its major tributaries were first plotted (based on known Australia Map Grid (AMG) coordinates system) using MapInfo™ software to enhance the accuracy of locating various information on the Yarra River. The AMG coordinates for various stations within the study area, such as streamflow gauging stations, STPs, EPA water quality monitoring sites (EPAWQ) and StreamWatch water quality monitoring sites (SWWQ) were then used to map them onto the Yarra River.

Five major tributaries, namely Worri Yallock Creek, Olinda Creek, Brushy Creek, Plenty River and Merri Creek, were considered in YRWQM, since these creeks receive discharges from STPs. The Yarra River together with the above tributaries were initially discretised into a number of reaches based on the locations of STPs, the confluences of tributaries to Yarra River and the locations of the EPAWQ stations. When the reaches defined based on the above criteria were long, they were further sub-divided. In total, 29 reaches were considered for the Yarra River and aforementioned tributaries, and each reach was assumed to have uniform pollution loading, and hydraulic and hydrological characteristics. These reaches are as shown in Figure 4.2. Each colour lines in Figure 4.2 representing a reach. Each reach was then sub-divided

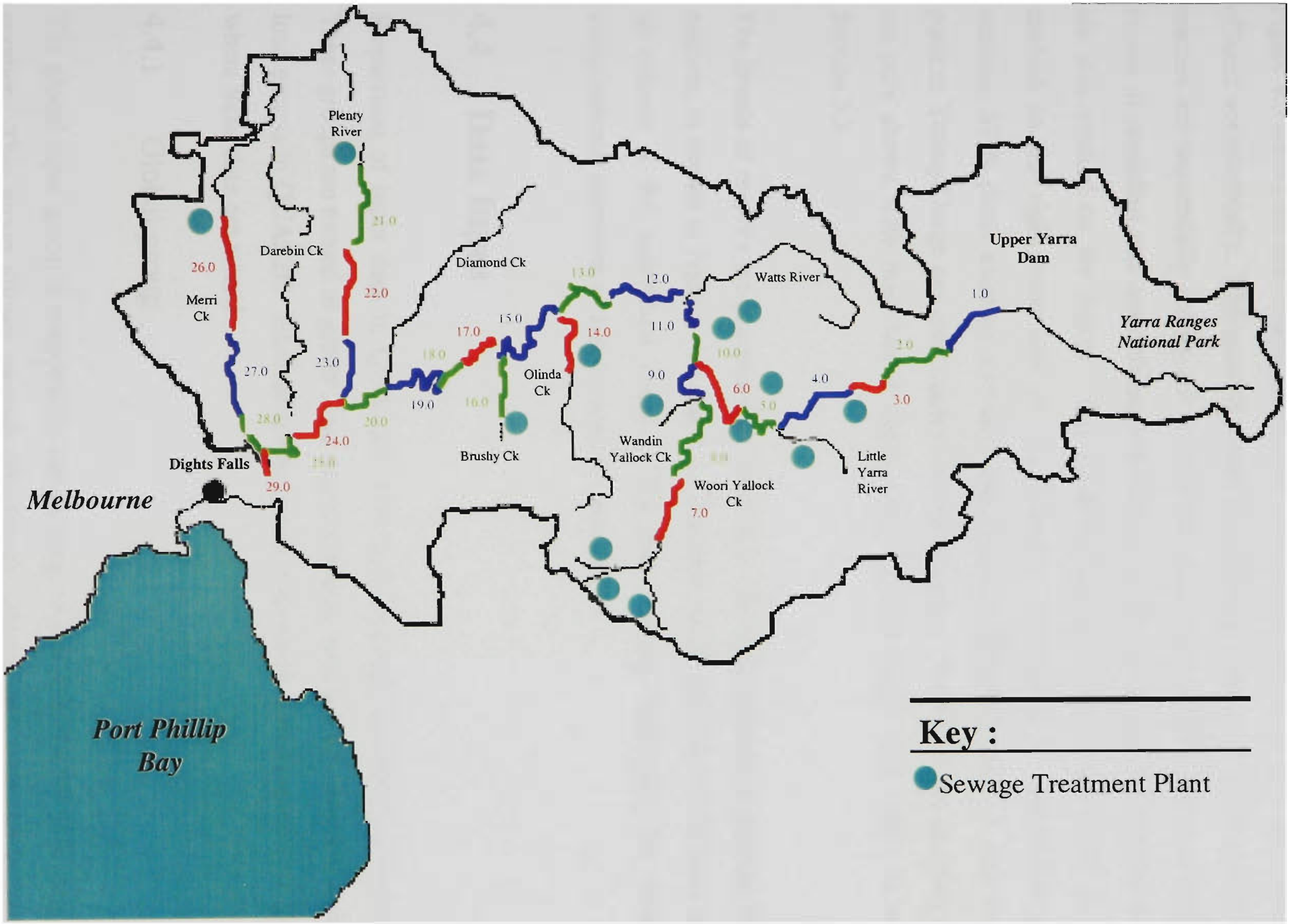


Figure 4.2 Reach Discretisation of Yarra River and Tributaries

into a number of computational elements of 1-km length, which provide sufficient resolution for water quality modelling (McCutcheon, 1989). Each computational element was assumed completely mixed.

Figure 4.3 shows the discretised Yarra River and its 5 major tributaries that receive STP effluent schematically. The reach boundary is denoted with small open circles and the reaches are sequentially numbered from UYD (upstream) to DFS (downstream). The names of modelled and unmodelled tributaries (which have gauging stations on them) are also marked on the figure. The modelled distance (km) from UYD to DFS is marked on the right hand side of the diagram. Locations of streamflow gauging stations, STPs (both existing and decommissioned), EPAWQ stations and extraction point at Yering Gorge are also marked on the diagram. Note that the gauging stations are only shown with their last 3 digits since they all begin with '229', as stated in Section 3.3.

The details of reaches are presented in Table 4.1. The first column represents the reach number, as shown in Figures 4.2 and 4.3. Distance measured from UYD (km) is shown in column 2 for individual reach. The last column represents the number of computational elements of 1-km length of each reach.

4.4 Data Input

Preparation of input data in QUAL2E were done through 6 main parameter groups. These groups are named as global, hydraulic, reaction, incremental, headwater and point load groups in QUAL2E. Each of these groups represents a compartment in QUAL2E where input data are stored.

4.4.1 Global group

The global input group is responsible for all data, which do not vary from one reach to another. This group allows various simulation options (e.g. nutrient modelling and algae on/off), assigns units, specifies temperature correction factors for decay rates,

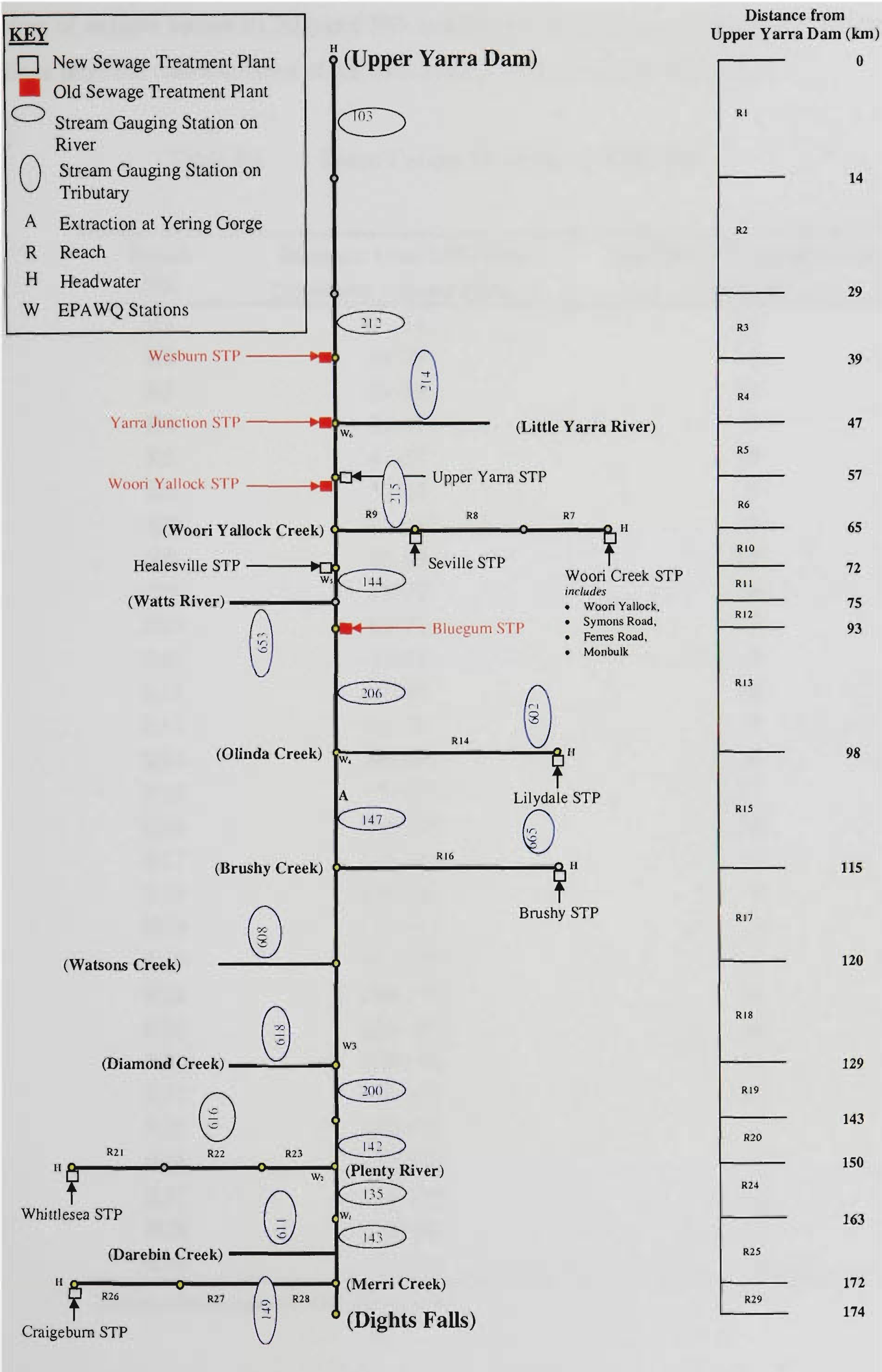


Figure 4.3 Schematic Diagram of Discretised Yarra River and Tributaries

factors of oxygen uptake by NH_3 and NO_2 oxidation (i.e. α_5 and α_6 in Equation 4.9) and defines physical characteristics of the river system such as number of reaches.

Table 4.1 Reach Details Modelled in YRWQM

Reach No.	Distance from DFS (km) (upstream – downstream)	Number of Computational Elements
R1	0 -14	14
R2	14-29	15
R3	29-39	10
R4	39-47	8
R5	47-57	10
R6	57-65	8
R7	**65-105**	**7**
R8	**65-98**	**19**
R9	**65-79**	**14**
R10	65-72	7
R11	72-75	3
R12	75-93	18
R13	93-98	5
R14	**98-106**	**8**
R15	98-115	17
R16	**115-125**	**10**
R17	115-120	5
R18	120-129	9
R19	129-143	14
R20	143-150	7
R21	**150-197**	**16**
R22	**150-181**	**18**
R23	**150-163**	**13**
R24	150-163	13
R25	163-172	9
R26	**172-203**	**4**
R27	**172-199**	**14**
R28	**172-185**	**13**
R29	172-174	2

Bold reaches represents tributaries reaches

Default values specified in QUAL2E users manual (Brown and Barnwell, 1987) were used for all global group entries, except the physical characteristics of the river system, where specific values related to YRWQM was used. It was reasonable to assume the default values for these parameters, as these parameters were not sensitive to water quality responses in YRWQM (Chapter 5). Table 4.2 presents examples of some of the values used in the global group, where the first column represents the name of the input entry and the corresponding value used is shown in second column. Table 4.3 presents the temperature correction factors used for decay rates.

Table 4.2 Examples for Some of the Default Entries Used in the Global Group

Input data	Value
Oxygen uptake by NH_3 oxidation (α_5)	3.43 mg per unit of NH_3 oxidation
Oxygen uptake by NO_2 oxidation (α_6)	1.14 mg per unit of NO_2 oxidation

Table 4.3 Temperature Correction Factors for Decay Rates

Decay rate	Symbol	Temperature Correction Factor
CBOD decay	$CBOD_d$	1.047
Org-N decay	Org-N_d	1.047
NH_3 decay	$NH_{3\text{-}d}$	1.083
NO_2 decay	$NO_{2\text{-}d}$	1.047
Org-P decay	Org-P_d	1.047
CBOD settling	$CBOD_s$	1.024
Org-N settling	Org-N_s	1.204
Org-P settling	Org-P_s	1.024
SOD	SOD	1.060
NH_3 benthos	NH_3ben	1.074
Diss-P benthos	Diss-Pben	1.074

4.4.2 Hydraulic group

The hydraulic input group is responsible for all data related to computation of flows (or hydraulics) and modelling of longitudinal dispersion.

QUAL2E provides two methods to model the hydraulics of river reaches, namely the *Manning* equation (Chow, 1959) and the *Leopold-Maddox* empirical power functions (Thomann and Mueller, 1987). These methods basically compute depth and velocity for various flows in the river reaches. In this study, the power functions were used because the *Manning* equation requires river longitudinal slope, side slopes of reach cross-sections (assuming a trapezoidal channel) and *Manning* roughness coefficients of the reaches, which could not be estimated accurately, because of the complexity of the river reaches.

4.4.2.1 Power functions for study reaches

The *Leopold-Maddox* empirical power functions relate depth and velocity to flow and are given in Equations 4.11 and 4.12.

$$D = a\,Q^{b} \tag{4.11}$$

$$V = c\,Q^{d} \tag{4.12}$$

where	D	is stream depth (m)
	V	is stream velocity (m/s)
	a, b, c and d	is empirical constants

Stage-discharge rating curves and cross-section details of gauging stations were used to determine the empirical coefficients a, b, c and d of power functions. Not all streamflow gauging stations shown in Figure 4.3 have the information on rating curves and cross-section details to develop power functions. Therefore, only those gauging stations that have above information were considered for development of the power functions and these gauging stations are 103, 212, 653, 602, 147, 200, 142, 135, 143 and 149 (Figure 4.3).

Three groups of reaches were considered in estimating power functions, as follows.

(a) Yarra River reaches and tributaries that have gauging stations on them. These gauging stations were assumed to have both rating curve and cross-sectional data information.

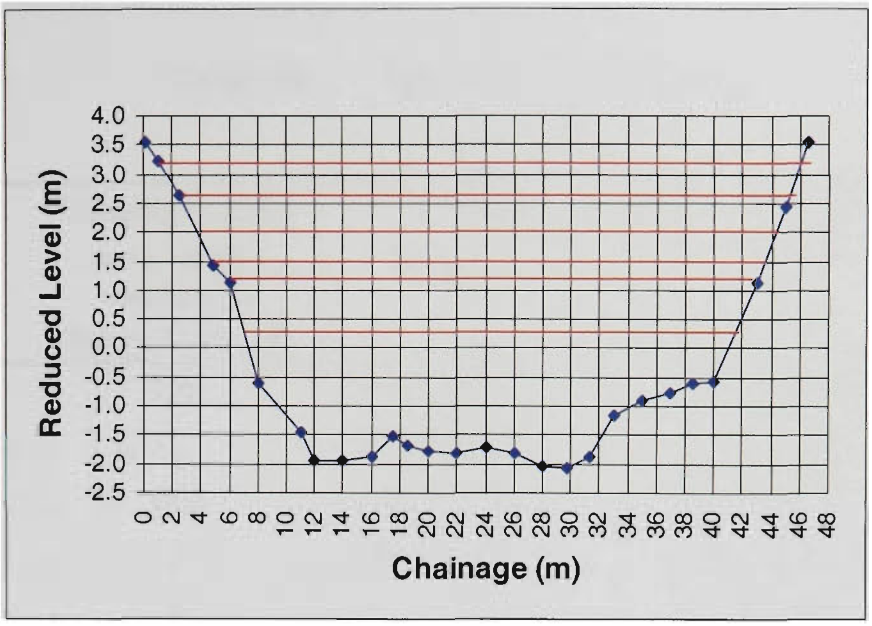
(b) Yarra River reaches that have no gauging stations on them.

(c) Tributary reaches that have no gauging stations on them.

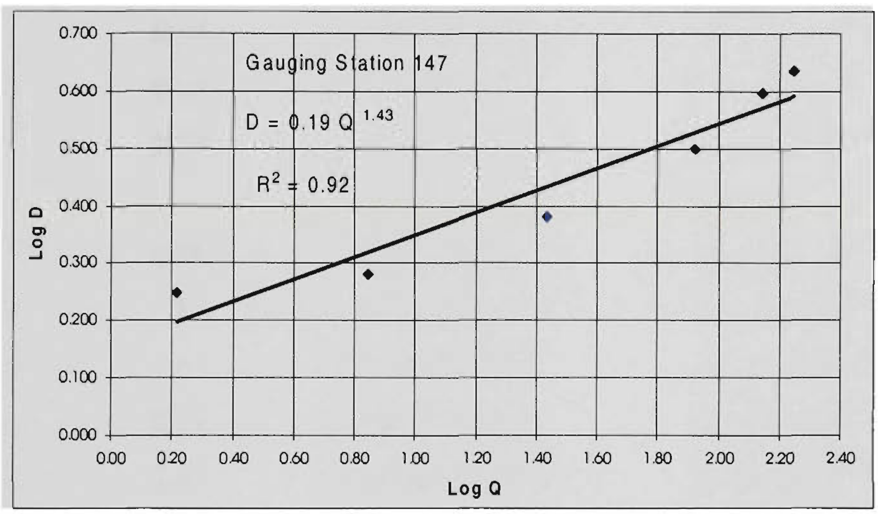
(a) Reaches with gauging stations

To derive power functions for this group, first a gauging station (which is within the reach) was identified for the reach. Then, several elevations from both rating curves and cross-sectional profile were considered in developing the power functions. However, in all cases, it was found that 6 elevations were adequate to determine the power functions. These elevations are marked on cross-sectional profile plot and water area is computed corresponding to each elevation. The depth was then determined as the ratio to the area to the top water surface width. The discharge corresponding to this elevation was then obtained from the rating curve and the velocity was estimated as the ratio of discharge to the water area. These relationships (i.e. depth-discharge and velocity-discharge) were plotted on log-log scale and empirical coefficients were determined. Figure 4.4 shows these relationships for Yering Gorge gauging station (147) with its river cross-section. These relationships were developed for use in Reach 15 (Figure 4.3). Similar figures are provided in Figures A1-1-A1-9 of Appendix A1 for other reaches in group (a). The power functions for depth and velocity are shown in Table 4.4.

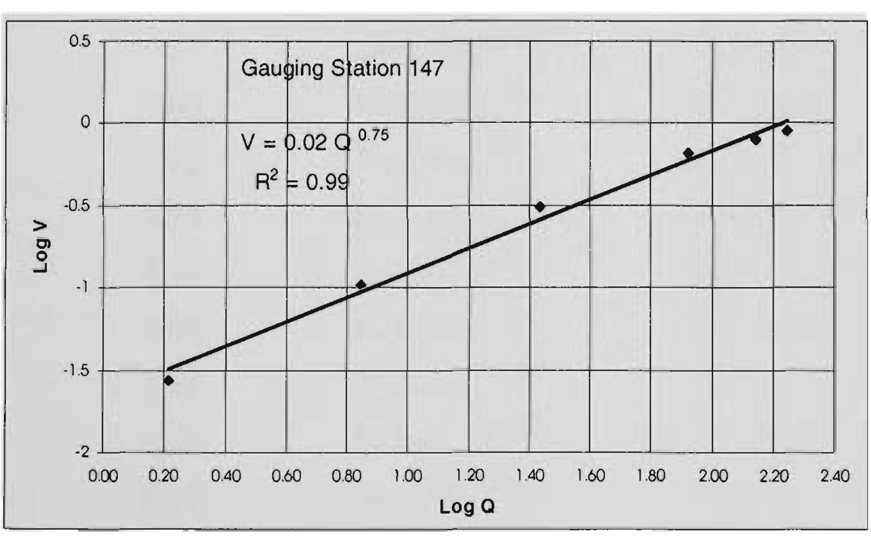
Note that Reach 24 has a negative exponent for velocity, which means as flow increases, velocity decreases. Negative velocity exponents have also experienced by Brush (1961).



(a) Cross-Section for Gauging Station 147



(b) D-Q Relationship



(b) V-Q Relationship

Figure 4.4 Cross-Section for Gauging Station 147 and Power Function for Reach 15

Table 4.4 Derived Power Functions

Reach	Gauging stations used, nearby reaches or Interpolation (I)	Hydraulic Power Functions $D=a\ Q^b$	$V=c\ Q^d$	Group
R1	103	$D=4.32\ Q^{0.07}$	$V=0.02\ Q^{0.915}$	(a)
R2	I	$D=2.23\ Q^{0.06}$	$V=0.05\ Q^{0.74}$	(b)
R3	212	$D=0.96\ Q^{0.19}$	$V=0.05\ Q^{0.73}$	(a)
R4	I	$D=0.89\ Q^{0.13}$	$V=0.12\ Q^{0.26}$	(b)
R5	I	$D=0.71\ Q^{0.16}$	$V=0.22\ Q^{0.14}$	(b)
R6	I	$D=0.55\ Q^{0.32}$	$V=0.33\ Q^{0.15}$	(b)
R7	R14	$D=0.31\ Q^{0.44}$	$V=0.56\ Q^{0.09}$	(c)
R8	R14	$D=0.31\ Q^{0.44}$	$V=0.56\ Q^{0.09}$	(c)
R9	R14	$D=0.31\ Q^{0.44}$	$V=0.56\ Q^{0.09}$	(c)
R10	I	$D=0.40\ Q^{0.45}$	$V=0.41\ Q^{0.15}$	(b)
R11	653	$D=0.15\ Q^{0.71}$	$V=0.44\ Q^{0.13}$	(a)
R12	I	$D=0.42\ Q^{0.47}$	$V=1.23\ Q^{0.21}$	(b)
R13	I	$D=0.26\ Q^{0.23}$	$V=0.15\ Q^{0.30}$	(b)
R14	R14	$D=0.31\ Q^{0.44}$	$V=0.56\ Q^{0.09}$	(a)
R15	147	$D=0.19\ Q^{1.43}$	$V=0.02\ Q^{0.75}$	(a)
R16	602	$D=0.31\ Q^{0.44}$	$V=0.56\ Q^{0.09}$	(c)
R17	I	$D=1.67\ Q^{0.11}$	$V=0.10\ Q^{0.36}$	(b)
R18	I	$D=0.70\ Q^{0.20}$	$V=0.09\ Q^{0.60}$	(b)
R19	200	$D=0.92\ Q^{0.24}$	$V=0.04\ Q^{0.65}$	(a)
R20	142	$D=2.40\ Q^{0.08}$	$V=0.02\ Q^{0.87}$	(a)
R21	R28	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(c)
R22	R28	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(c)
R23	R28	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(c)
R24	135	$D=0.05\ Q^{0.85}$	$V=4.77\ Q^{-0.39}$	(a)
R25	143	$D=0.60\ Q^{0.28}$	$V=0.06\ Q^{0.60}$	(a)
R26	R28	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(c)
R27	R28	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(c)
R28	149	$D=0.87\ Q^{0.16}$	$V=0.15\ Q^{0.59}$	(a)
R29	R25	$D=0.60\ Q^{0.28}$	$V=0.06\ Q^{0.60}$	(c)

(b) Yarra River reaches with no gauging stations

Determining power functions for reaches in this group (e.g. Reach R2 in Figure 4.3), a linear interpolation of the (already developed) power functions of the two closest gauging stations on either side was used. These reaches are labelled 'I' in the second column of Table 4.4. For these reaches, the power functions were developed using 6 low flow events described in Chapter 3 (Table 3.9) and therefore limited to low flows but covers flow ranges upto 40 m^3/s at DFS. The 6 flow events included 3 'lower range' flows and 3 'higher range' flows. This was considered adequate for this study. If YRWQM is to be used for high flows beyond 40 m^3/s at DFS, it is necessary to re-derive these functions.

The interpolation procedure is shown in Figure 4.5. In this figure, the power functions are to be derived for Reach 2 (at the mid-point of the reach) from power functions at gauging stations GS_1 and GS_3, which are also shown in Figure 4.5. For each flow event (flows of Q_1 and Q_3 at gauging station GS_1 and GS_3), the depth D_1 and D_3 and velocity V_1 and V_3 can be obtained from the respective power functions at gauging stations GS_1 and GS_3 respectively, and the depth D_2 and V_2 can be calculated for this flow event using linear interpolation for reach 2 considering the length. The discharge corresponding to this flow event (Q_2) at the mid-point of Reach 2 can be estimated from the flow at upstream gauging stations (GS_1), and any measured tributary and other inflows and outflows (such as STP inflow and diversion outflows) between GS_1 and mid-point of Reach 2, and the incremental flow. The incremental flow accounts for all ungauged flows between any two points in the river and the method of estimation of incremental flow is discussed in Section 4.4.4. For each flow event, a set of D_2, V_2 and Q_2 was obtained and the power functions were then determined by plotting the relationships between depth and velocity with discharge on log-log scale. These power functions are shown in Table 4.4. Although it is acknowledged that there can be errors in power functions derived by this method, this was perhaps the best method that can be used due to lack of data for these reaches. The derived rating curves for group (b) reaches are shown in Figures A2-1-A2-9 of Appendix A2.

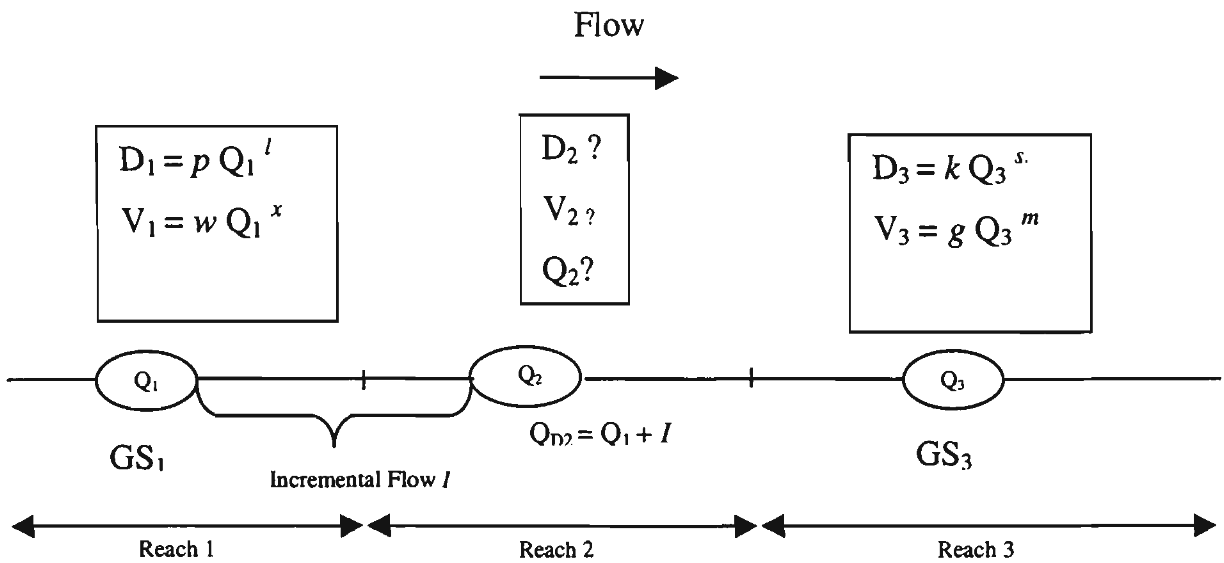


Figure 4.5 Derivation of Power Functions for Group (b) Reaches

(c) Tributary reaches with no gauging stations

For reaches in tributaries that have no gauging stations, the power functions of the nearby reaches were adopted. In this case, an assumption was made that nearby reaches have similar morphometry. This may not be the best approach to this problem again, however, due to no information available, this was considered to be the best method for group (c) reaches. The tributary reaches that fall into this group are,

- Reaches 7-9 (Woori Yallock Creek); these reaches were considered to be similar to Reach 14
- Reach 16 (Brushy Creek); this reach was considered to be similar to Reach 14
- Reaches 21-23 (Plenty River); these reaches were considered to be similar to Reach 28
- Reaches 26 and 27 (Merri Creek); these reaches were considered to be similar to Reach 28

In addition to above tributary reaches, this method was also used for Reach 29 (the last reach of Yarra River), since the method adopted for group (b) reaches could not be used for this case. Table 4.4 shows the similar reaches used for group (c) together with the power functions.

4.4.2.2 Longitudinal dispersion

Longitudinal dispersion exists in rivers due to horizontal and vertical gradients of velocity (Thomann and Mueller, 1987) and this was considered in YRWQM. The longitudinal dispersion is modelled in QUAL2E through Equation 4.13.

$$D_L = 3.82\ K\ n\ U\ D^{5/6} \qquad 4.13$$

where	D_L	is dispersion coefficient (m^2/d)
	K	is dispersion constant (m^2/d)
	D	is mean depth of the stream (m)
	n	is Manning's n roughness coefficient
	U	is mean velocity (m/d)

The required input for QUAL2E was the dispersion constant (*K*) and *Manning's* n coefficient. The recommended typical range for *K* is from 6 to 6000 based on previous studies (Brown and Barnwell, 1987). As Fischer *et al.* (1979) stated, this coefficient was not sensitive to most applications. Pre-calibration uncertainty and sensitivity analysis conducted in Chapter 5 also found that this coefficient was insensitive. Therefore, a value of 60 was used in this study, as this value was used in an example in QUAL2E user's manual (Brown and Barnwell, 1987), which was based on Willamette River study in USA. The *Manning's* n coefficient of 0.02 was used, as it was the default value in QUAL2E. Although this value seems to be low, it was considered to be satisfactory, since it was not used for hydraulic calculations but only for modelling dispersion. Furthermore, as found in Chapter 5, *Manning's* n coefficient is not a sensitive parameter.

4.4.3 Reaction group

The reaction group consists of parameters related to decay of water quality constituents and reaeration rate.

4.4.3.1 Decay rate constants

As discussed in Section 2.4.2.1, QUAL2E can simulate various nonconservative water quality constituents. In this study, only N, P, CBOD and DO were considered. Although algae as chlorophyll-a is considered to be one of the important water quality constituents in effecting the overall DO and P interactions, it was not considered in this study for the following reasons.

(1) The available EPAWQ data showed that the concentration of chlorophyll-a measured at the six water quality measuring points were all less than the recommended water quality standards of 0.1 ug/L (ANZECC, 1999) and therefore, it is not of major concern to Yarra River.

(2) The effluent chlorophyll-a data for Yarra River STPs were not available.

(3) A QUAL2E run of YRWQM using STP effluent data of the Willamette river basin study (USEPA, 1997b) showed that there were no differences on NH_3 and DO concentration when algae was simulated and not simulated.

(4) As indicated in USEPA (1997b), with tertiary level of wastewater treatment, effluent should not have significant chlorophyll-a concentration. This could be the reason that chlorophyll-a was not measured at STPs of Yarra River catchment, as in point (2) above.

Faceal Coliform was also not considered, since it was not a major factor contributing to the Yarra River water quality (EPA Victoria, 1999).

Several decay rates are required for modelling N, P, CBOD and DO are shown in Table 4.5. The decay rates responsible for these nonconservative water quality constituents were determined through a preliminary analysis of Yarra River water quality data and were compiled into ranges of decay rates. These ranges were required for calibration of YRWQM using Genetic Algorithm (GA) discussed in Chapter 6.

Table 4.5 Decay Rate Ranges for YRWQM Calibration

Decay rate	Symbol	Range (per day)	Source
CBOD decay	$CBOD_d$	0.0042-3.5	Yarra Field data and Bowie et al. (1985)
Org-N decay	Org-N_d	0.006-0.42	Yarra Field data and Bowie et al. (1985)
NH_3 decay	$NH_{3\text{-}d}$	0.001-0.72	Yarra Field data and Bowie et al. (1985)
NO_2 decay	$NO_{2\text{-}d}$	0.001-0.7	Yarra Field data and Bowie et al. (1985)
Org-P decay	Org-P_d	0.001-1.0	Yarra Field data and Bowie et al. (1985)
CBOD settling	$CBOD_s$	0.001-1.53	Yarra Field data and Bowie et al. (1985
Org-N settling	Org-N_s	0.001-2.63	Yarra Field data and Bowie et al. (1985)
Org-P settling	Org-P_s	0-0.14	Yarra Field data and Bowie et al. (1985)
SOD	SOD	0-2	Bowie et al. (1985)
NH_3 benthos	NH_3ben	0.001-1.8	Bowie et al. (1985)
Diss-P benthos	Diss-Pben	0.001-1.7	Bowie et al. (1985)

These decay rates were determined in two ways, as follows:

(a) When data were available for water quality constituents, their decay rates were computed using 6 'lower range' flow events (Section 3.6) at different reaches, to arrive at several different rates. These decay rates were then collectively grouped into a range, which were compared against the range compiled in Bowie *et al.* (1985). This comparison produced a final range for each decay rate. Details of the procedure used in determining decay rates from flow events is discussed after (b) below.

(b) When data were not available for water quality constituents, their ranges were obtained from Bowie *et al.* (1985), which had summarised the rates from many different studies. These water quality constituents were NH_3ben, Diss-Pben and sediment oxygen demand (SOD), . These ranges are shown in Table 4.5.

'Lower range' flow events of Section 3.6 (i.e. flows less than 10 m^3/s at DFS) were used to determine the preliminary estimates of the decay rates, since the incremental (or lateral) flow and concentration can be ignored for these events. This will simplify the analysis, yet accurate enough to obtain preliminary estimates of the decay rates. In total, 5 reaches were considered for this estimation where upstream and downstream water quality station data for the 6 'lower range' flow events were available from EPAWQ and SWWQ, yielding a maximum of 30 points. For some reaction coefficients, the number of points were less than 30 due to unavailability of data.

The preliminary estimates of decay rates for nonconservative water quality constituents were computed using the first order reaction assumption, i.e. the rate of change in concentration is directly proportional to the concentration (Thomann and Mueller, 1987). This relationship is shown by

$$C = C_o \exp(-Kt) \tag{4.14}$$

where	C	is downstream concentration (mg/L)
	C_o	is upstream concentration (mg/L)
	K	is decay rate for a nonconservative pollutant (day^{-1})
	t	is travel time (day)

The derivation of the above solution to the first order steady state linear differential equation can be found in Thomann and Mueller (1987). Equation 4.14 assumes constant flow between inputs and uniform cross-sectional area over the river section. The estimation of decay rates were simplified in this study to consider only one decay rate per one nonconservative water quality constituents. For example, the reduction of NO_2 concentration normally involves two decay rates, namely biological oxidation of NH_3 and oxidation rate of NO_2, as shown in Equation 4.4. In this study, only the oxidation rate of NO_2 is determined.

The estimation of the 8 decay rates listed in Table 4.5 considered the use of Equation 4.14 which takes into account of effluent discharge from STPs, tributary inflows and diversion at Yering Gorge. Figure 4.6 shows a typical example for deriving decay rates in a reach with inflows from a STP. Tributary inflows and diversions are modelled similarly through inflows and outflows.

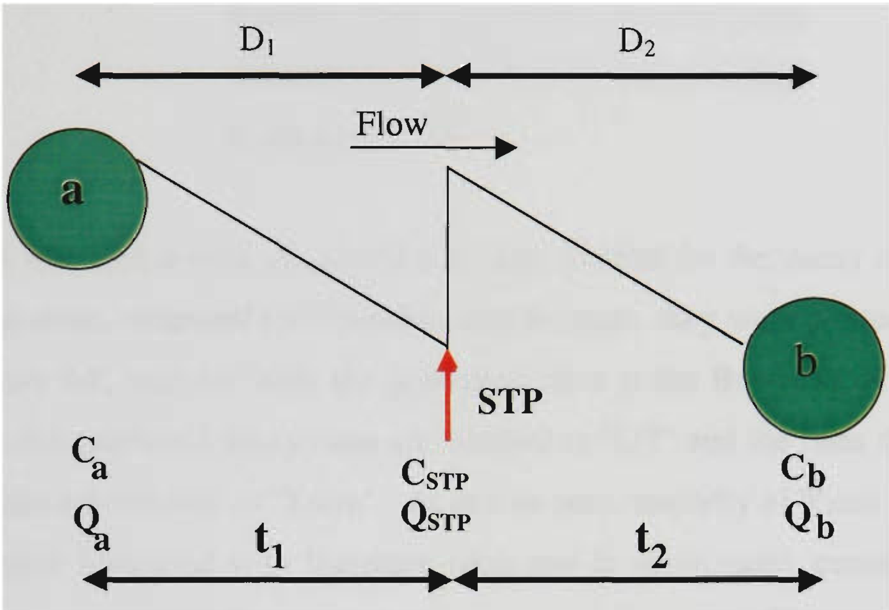


Figure 4.6 Example of Using First Order Reaction Equation and Mass Balance in Deriving Decay Rates

Consider the reach in Figure 4.6 with an upstream water quality station 'a' and a downstream water quality station 'b' and a STP. The STP is located at distances of D_1 and D_2 respectively from water quality stations 'a' and 'b'. The travel times t_1 and t_2 can be determined from the distance and velocity. Velocity was determined using the power functions estimated in Section 4.4.2.1 and the flow of each event. Combining Equation 4.14 and simple mass balance of all inflows/outflows and their concentrations, Equation 4.15 can be derived to estimate the decay rates for each nonconservative water quality constituents.

$$C_b = \left(\frac{Q_a \bullet C_a e^{-kt_1} + Q_{STP} \bullet C_{STP}}{Q_a + Q_{STP}} \right) e^{-kt_2} \qquad 4.15$$

where C_b is concentration measured at station b (mg/L)

C_a is concentration measured at station a (mg/L)

C_{STP} is effluent concentration discharged from STP (mg/L)

Q_a	is streamflow at station a (m^3/s)
Q_b	is streamflow at station b (m^3/s)
Q_{STP}	is flow discharged from STP (m^3/s)
t_1	is travel time from station a to STP (day)
t_2	is travel time from STP to station b (day)
k	is decay rate (day^{-1})

Equation 4.15 was then solved using trial and error method for the decay rate k. Once the decay rates were computed for 5 reaches and 6 events, they were plotted. They are shown in Figure 4.7, together with the published rates in the Bowie *et al.* (1985). In each diagram, the published decay rates are labelled as 'LIT' and the rates derived from Yarra River data are labelled as 'Yarra'. As can be seen, majority of Yarra River decay rates were higher compared with literature rates and in some cases, contains outliers. This could be due to limited data used to estimate the decay rates. The criterion used to select the appropriate decay rate range for use in the calibration of YRWQM was based on decay rate concentrations of both 'LIT' and 'Yarra' data sets in Figure 4.7, with some subjective engineering judgement. The selected decay rate range used for calibration for YRWQM is also marked on each of the plots in Figure 4.7. Note that the decay rate that shown in Figures 4.7 (i) to (k) did not have Yarra River data to determine the rates, hence literature rates are shown. These chosen ranges for the 11 decay rates are also given in Table 4.5.

4.4.3.2 Derivation of reaeration rates

One of the most important health indicators of the river is the DO level. The DO concentration is reduced through decaying of oxygen demand matter originated from effluent discharge and organic matter deposited in the streambed. On the other hand, DO concentration is increased through processes such as photosynthesis by plants and oxygen inputs from the atmosphere. The rate of oxygen input into the stream from the atmosphere is known as the reaeration rate coefficient.

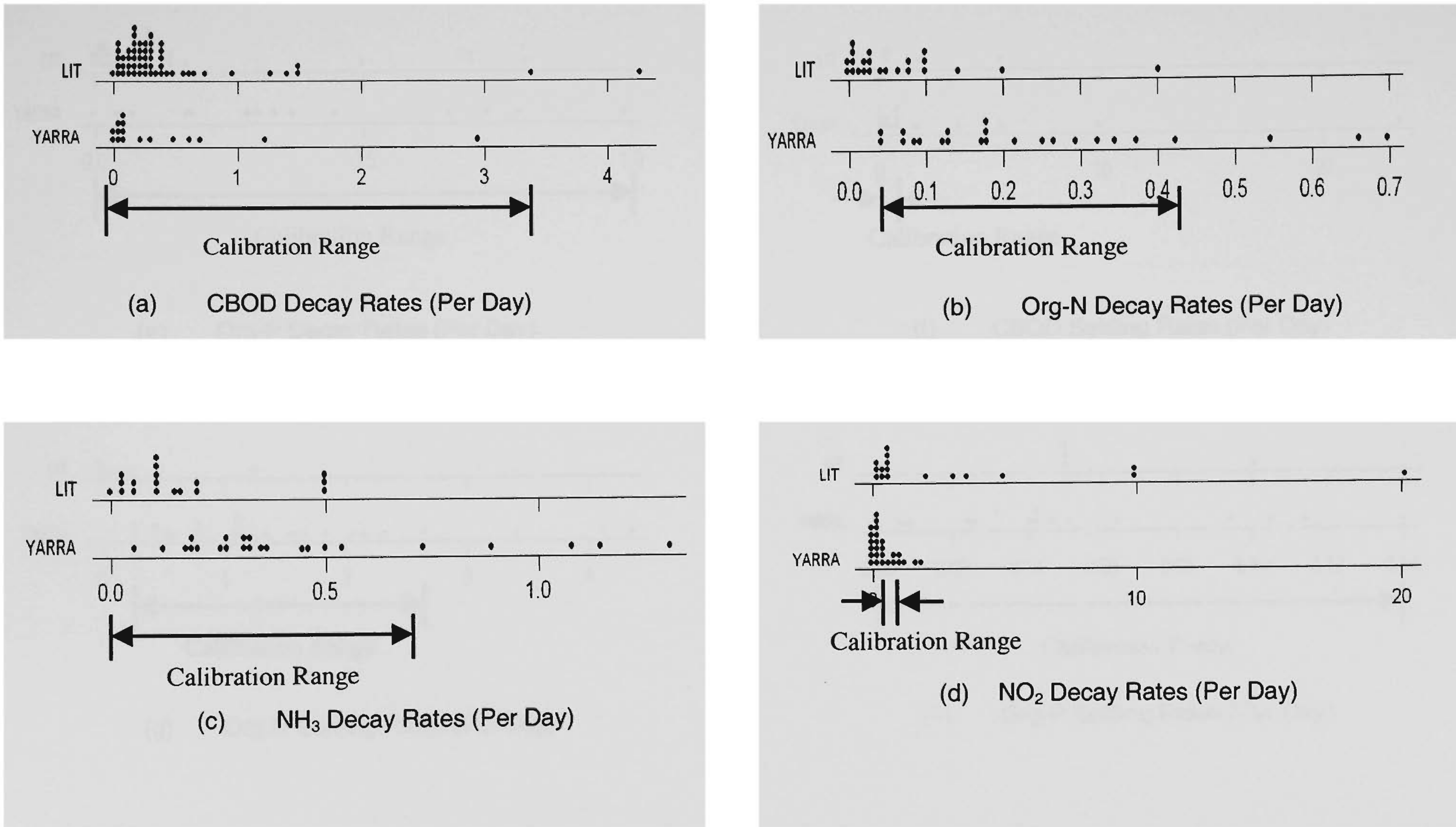
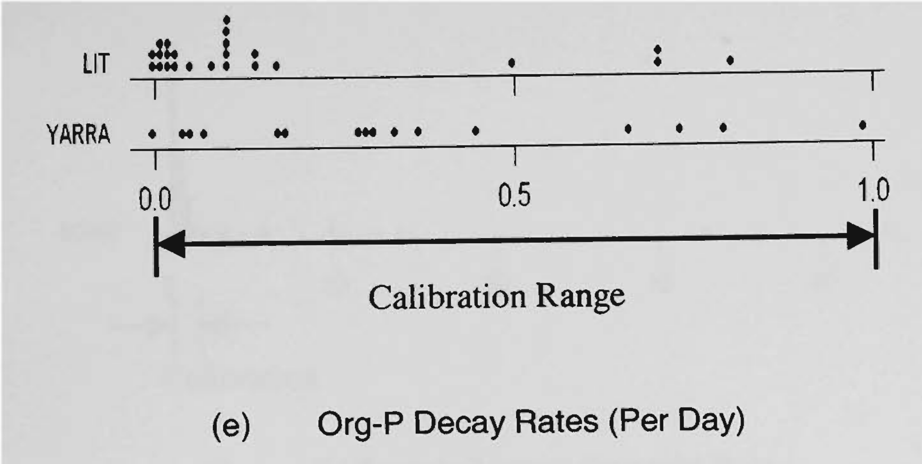
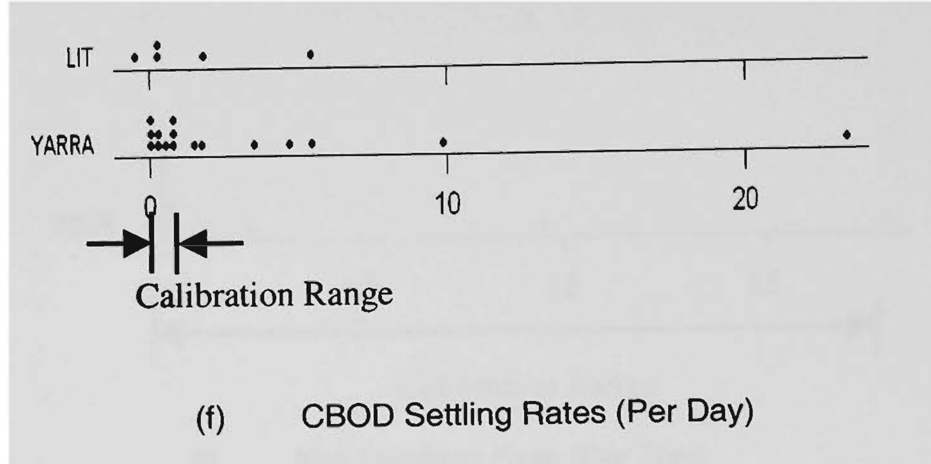
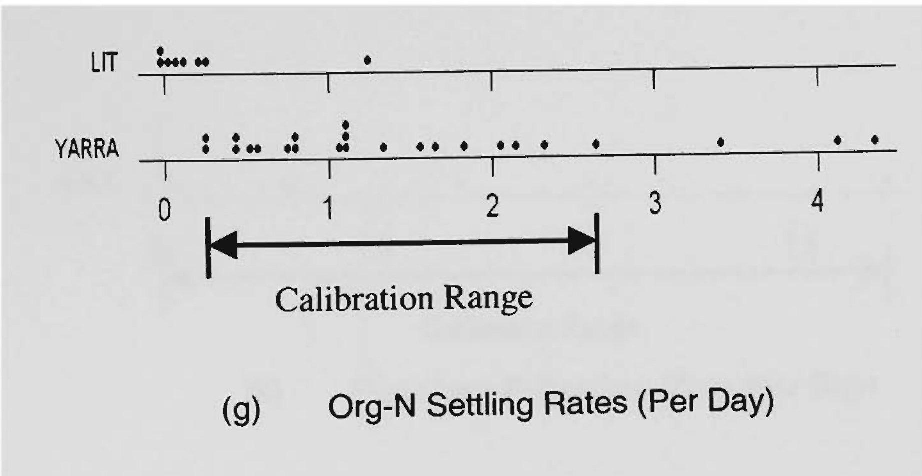


Figure 4.7 Decay Rate Ranges for YRWQM Calibration







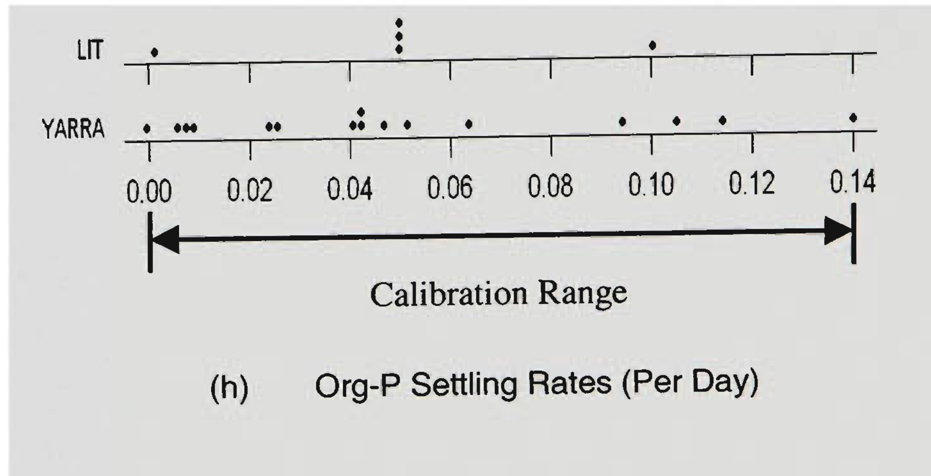
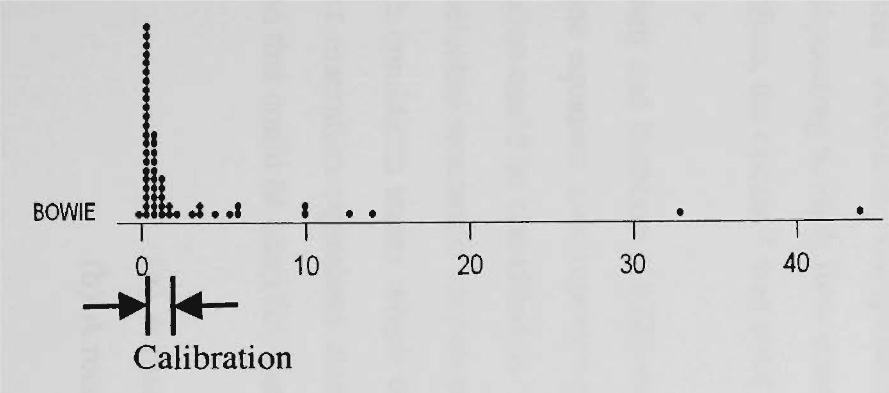
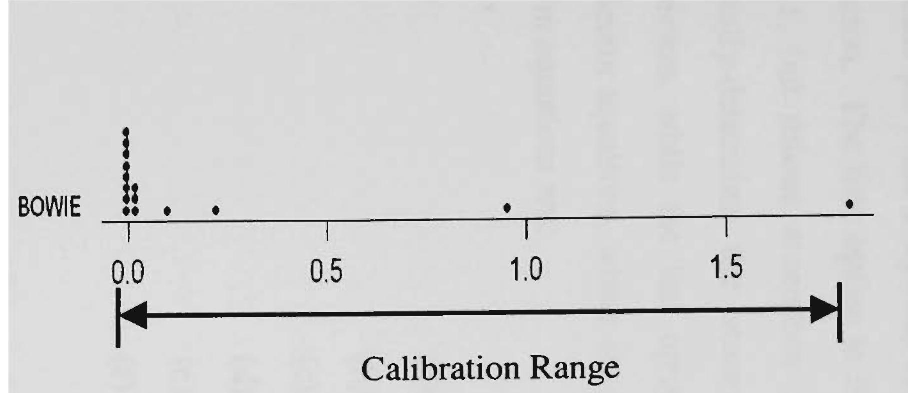


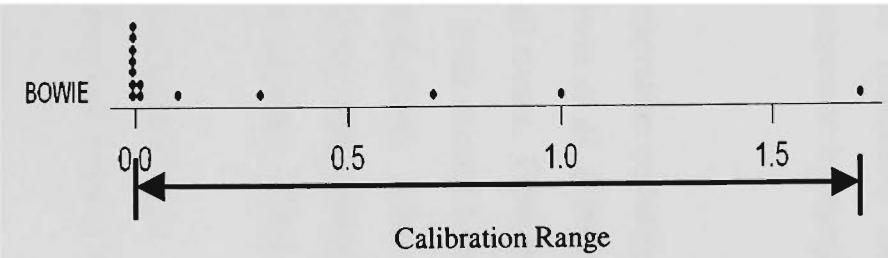
Figure 4.7 (Cont.) Decay Rate Ranges for YRWQM Calibration



(i) Sediment Oxygen Demand Rate (Per Day)



(j) NH_3 Benthos Rate (Per Day)



(k) Dissolved-P Benthos Rate (Per Day)

Figure 4.7 (Cont.) Decay Rate Ranges for YRWQM Calibration

QUAL2E provides 3 options to determine reaeration rate coefficients and modelling reaeration. The first option is through power functions similar to the ones in Section 4.4.2.1, but relates reaeration rate coefficient to flow. The second option is to externally-determine the reaeration rates coefficients and enter them as input parameters, while the last option is to use the 6 built-in empirical reaeration rate coefficient equations, where the reaeration rates are computed within QUAL2E. These built-in equations are:

(a) Churchhill
(b) O'Connor and Dobbins
(c) Owens, Edwards and Gibbs
(d) Thackston and Krenkel
(e) Langbien and Durum
(f) Tsivoglou -Wallace

The option 1 could not be used in this study, as no measured reaeration rate data were available to determine power functions. Although option 2 can be used by externally computing reaeration rate through the use of Covar (1976) method (discussed below), it requires extensive computations to determine reaeration rates for each reach corresponding to each flow event. This can be overcome by simply using option 3 and therefore, the option 3 was used in this study.

Bennett and Rathbun (1972) reviewed thirteen reaeration equations and concluded that no one equation was superior over another. Bowie *et al.* (1985) stated that no single equation could be classified as 'best' for use in all rivers. Furthermore, they stated that the selected reaeration empirical equations for a river should have similar velocity and depth conditions under which the equations were derived. After reviewing methods to select reaeration equations using depth and velocity relationships, two methods were found that could be used for Yarra River and its tributaries. They are:

(a) A method introduced by Covar (1976)
(b) A recent method by Moog and Jirka (1998)

The first method, developed by Covar (1976), compiled three commonly used reaeration rate equations (i.e. Churchill, O'Connor and Dobbins, and Owens, Edwards and Gibbs) into one system, where the reaeration rate can be estimated from depth and velocity. This method allows the selection of the most appropriate reaeration rate equation for the river to cover a wide range of depth and velocity, considering all above 3 equations in one plot.

The second method, developed by Moog and Jirka (1998), suggested the use of a number of representative values and multiplying factors to 10 empirical reaeration rate equations (such as Churchill and O'Connor and Dobbins), based on the river slope. Since data on river slope was not available for Yarra River, this method could not be used in this study. Therefore, the Covar (1976) method was used in this study to select the appropriate reaeration rate equation for use in YRWQM. The use of the Covar method for YRWQM is explained below.

Depths and velocities were computed for each reach corresponding to 10 selected flow events in Section 3.6 (Table 3.9) and plotted on velocity–depth plot of Covar (1976). These depths and velocities were plotted separately for 'lower range' and 'higher range' flows respectively of main Yarra River and they are shown in Figures 4.8 and 4.9. Figures 4.10 and 4.11 show similar plots for tributaries. In these figures, the reach number was plotted for each pair of depth and velocity corresponding to each flow event. For both Yarra River and tributaries, there were no significant differences between the reaeration methods that are applicable to 'lower range' and 'higher range' flows. As can be seen from Figures 4.8 and 4.9, O'Connor-Dobbins was the most popular method for majority of the reaches. Only Reaches 11-13 showed Owens-Gibbs under both flow conditions.

For most tributary reaches, the O'Connor-Dobbins method shows as a better method. Table 4.6 shows the selected reaeration method for use in YRWQM based on the Covar method.

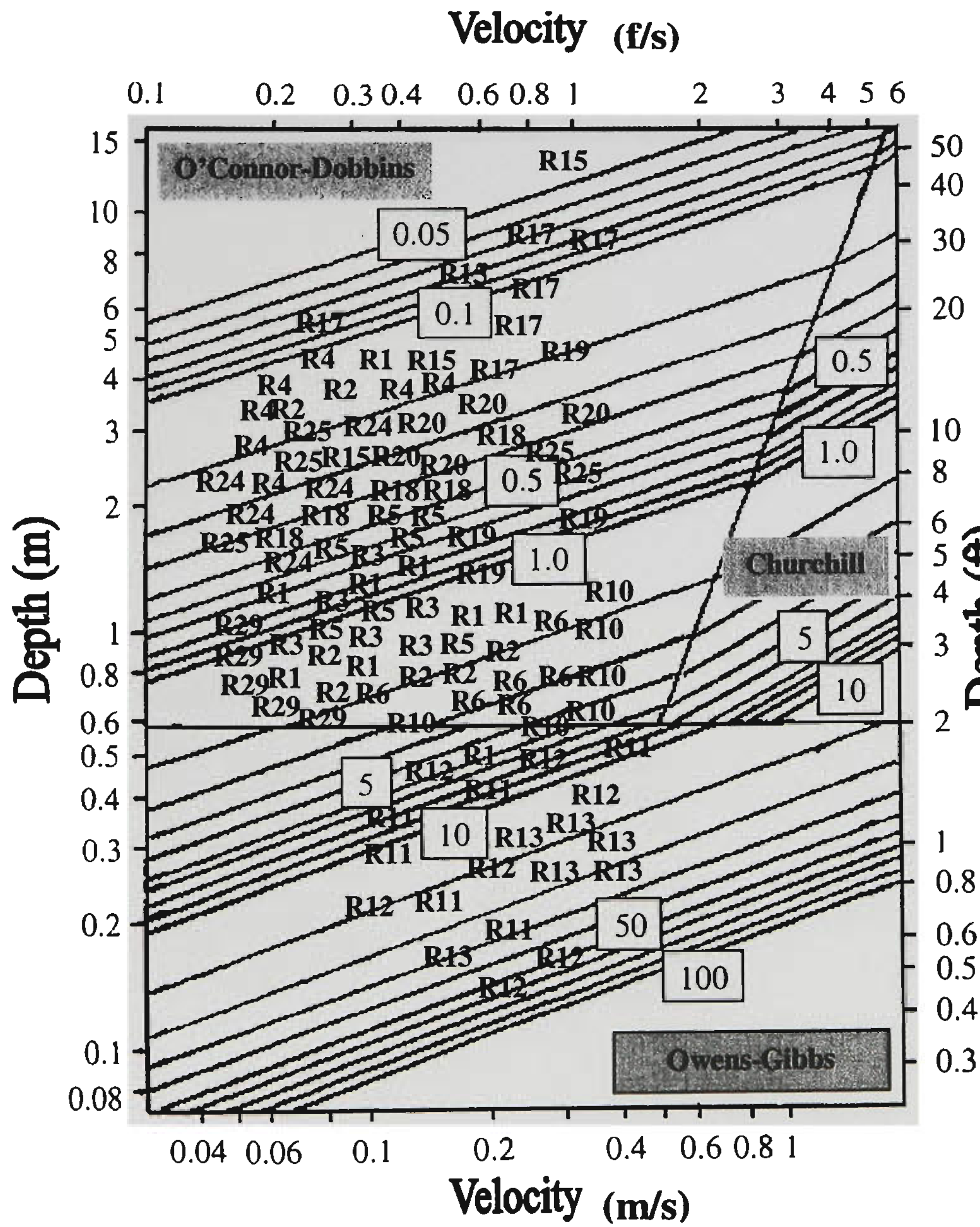


Figure 4.8 Selection of Reaeration Method for 'Lower Range' Flows for Main Yarra River

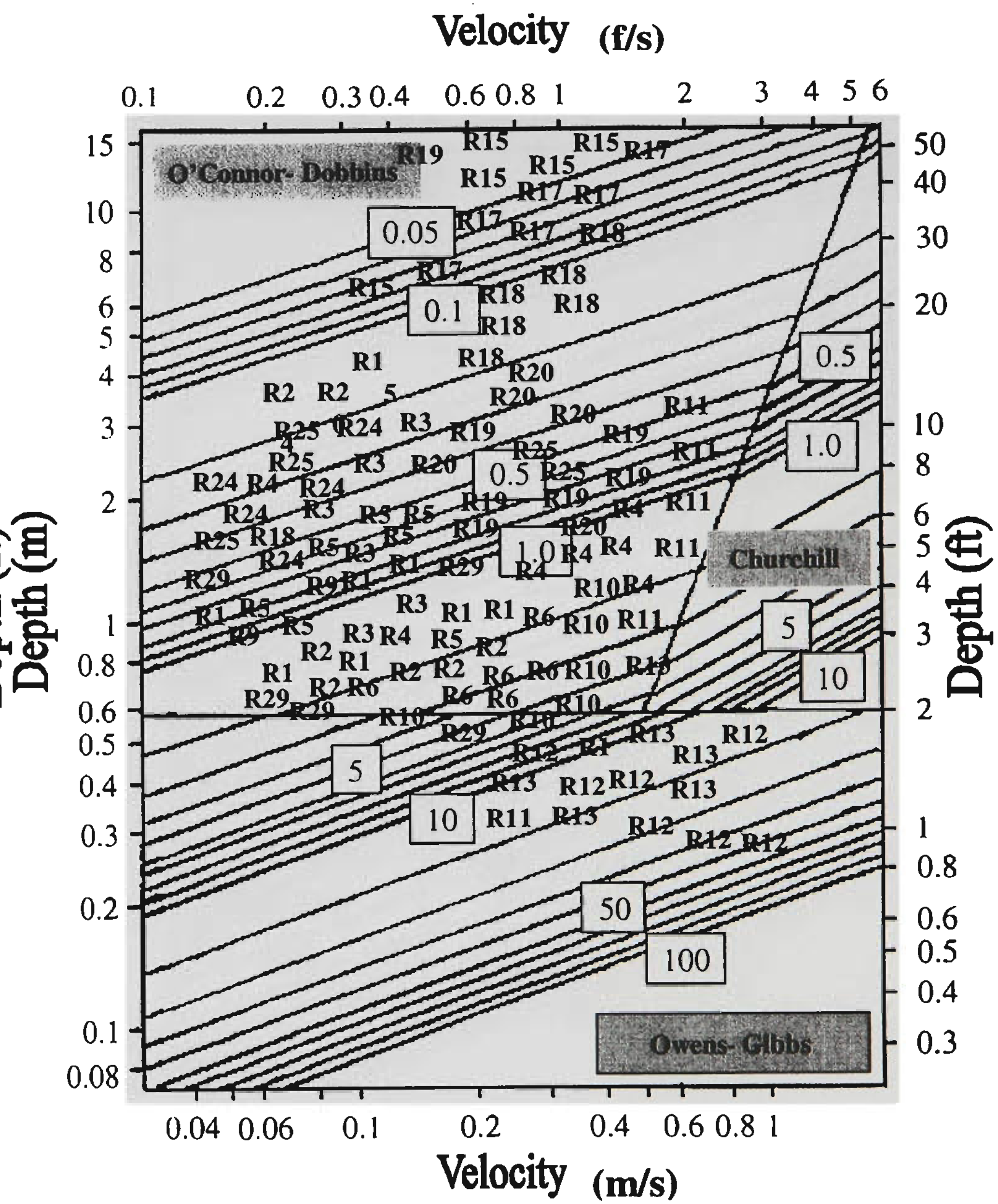


Figure 4.9 Selection of Reaeration Method for 'Higher Range' Flows for Main Yarra River

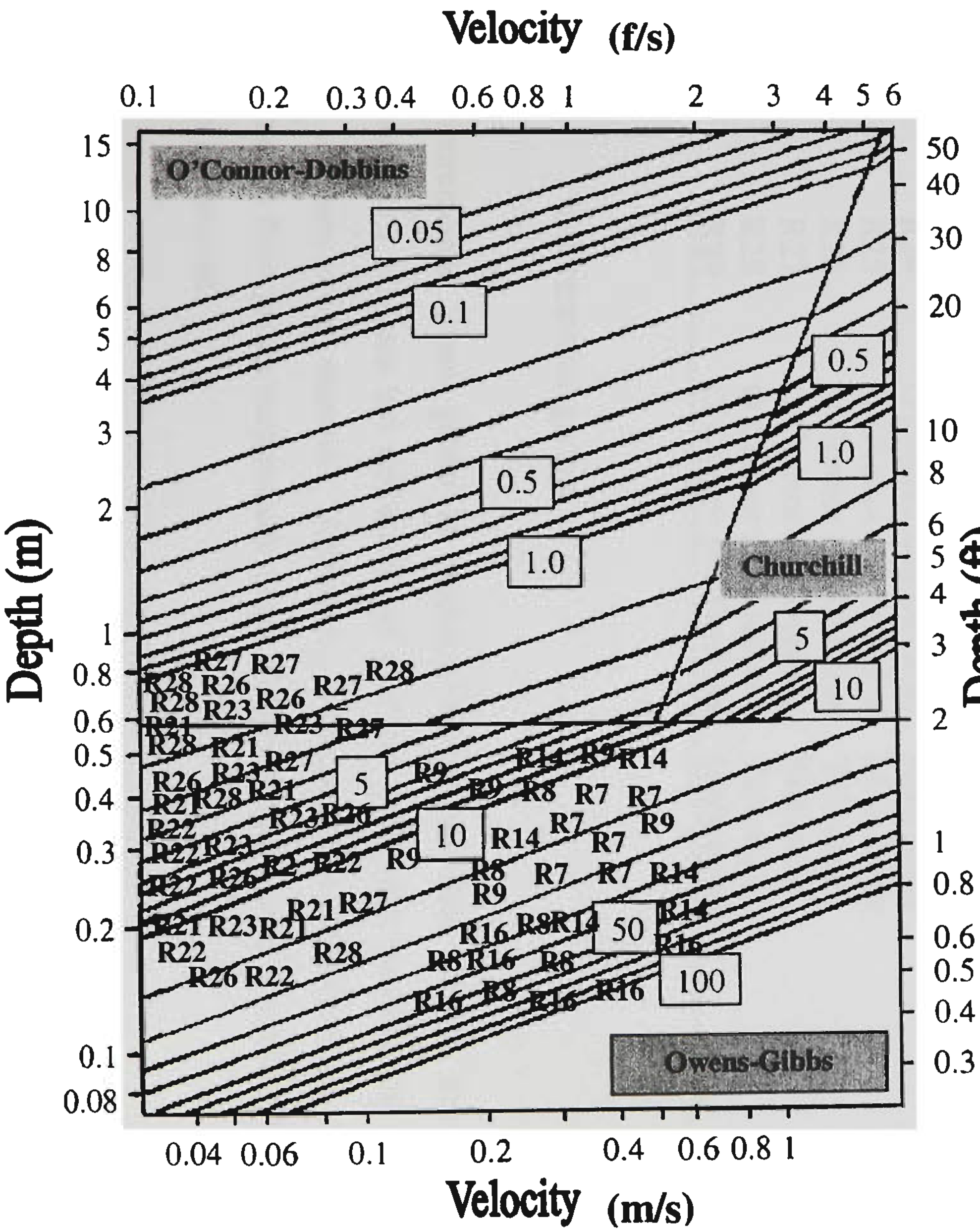


Figure 4.10 Selection of Reaeration Method for 'Lower Range' Flow for Tributaries

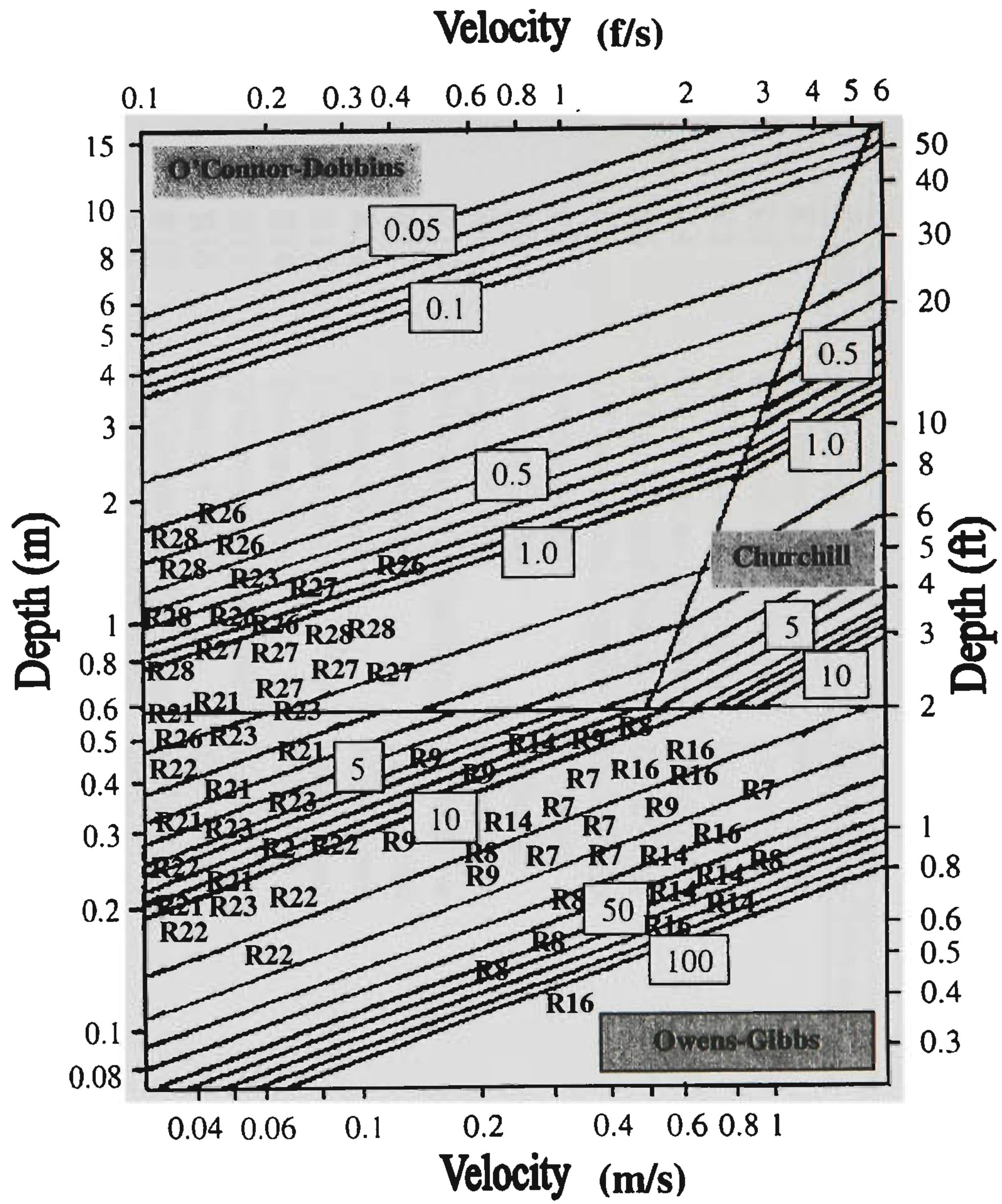


Figure 4.11 Selection of Reaeration Method for 'Higher Range' Flow for Tributaries

Table 4.6 Selected Reaeration Method for YRWQM

Reach	Reaeration Method	Main River/Tributary
R1	O'Connor and Dobbins	Main River
R2	O'Connor and Dobbins	Main River
R3	O'Connor and Dobbins	Main River
R4	O'Connor and Dobbins	Main River
R5	O'Connor and Dobbins	Main River
R6	O'Connor and Dobbins	Main River
R7	Owens-Gibbs	Tributary
R8	Owens-Gibbs	Tributary
R9	Owens-Gibbs	Tributary
R10	O'Connor and Dobbins	Main River
R11	Owens-Gibbs	Main River
R12	Owens-Gibbs	Main River
R13	Owens-Gibbs	Main River
R14	Owens-Gibbs	Tributary
R15	O'Connor and Dobbins	Main River
R16	Owens-Gibbs	Tributary
R17	O'Connor and Dobbins	Main River
R18	O'Connor and Dobbins	Main River
R19	O'Connor and Dobbins	Main River
R20	O'Connor and Dobbins	Main River
R21	O'Connor and Dobbins	Tributary
R22	O'Connor and Dobbins	Tributary
R23	O'Connor and Dobbins	Tributary
R24	O'Connor and Dobbins	Main River
R25	O'Connor and Dobbins	Main River
R26	O'Connor and Dobbins	Tributary
R27	O'Connor and Dobbins	Tributary
R28	O'Connor and Dobbins	Tributary
R29	O'Connor and Dobbins	Main River

4.4.4 Incremental group

The incremental input group is responsible for all data related to flows and concentrations that are not delivered via a single point, e.g. runoff from farmlands. They are non-point sources. Although, in theory this definition is correct, the incremental flow group may include point sources (usually small) which cannot be (or are not) measured.

4.4.4.1 Incremental flow

The incremental or nonpoint source flows are generally delivered to the river from urban and rural runoff. In this study, the incremental flows are assumed to be uniformly distributed along the river reach and computed based on the difference in observed flows at two gauging stations. The method also accounts for STP effluent flow, tributary inflows and diversions.

Consider the example of a river section with 2 reaches and a STP discharging effluent to the river, as shown in Figure 4.12. Although only a STP is considered in this example, any inflow/outflow within reaches can be handled with this method. The distance of STP to the two gauging stations are x and y, as shown in Figure 4.12.

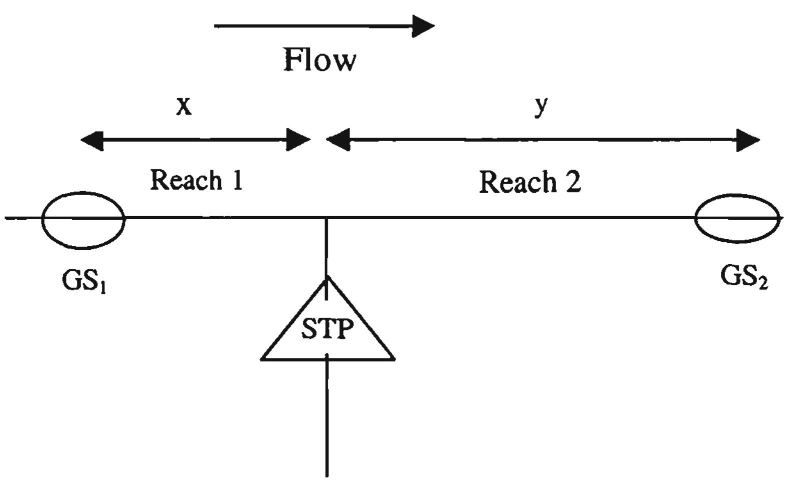


Figure 4.12 Incremental Flow Estimation

To determine incremental flow for Reaches 1 and 2, the following equations were used. These equations are based on continuity equation and assumes that the incremental flows are proportional to the length of the river

$$q_1 = [\frac{(Q_2 - Q_1 - Q_{STP})}{x + y}]\, x \qquad 4.16$$

$$q_2 = [\frac{(Q_2 - Q_1 - Q_{STP})}{x + y}]\, y \qquad 4.17$$

where	q_1	is total incremetal flow for Reach 1 (m^3/s)
	q_2	is total incremetal flow for Reach 2 (m^3/s)
	Q_1	is streamflow measured at gauging station GS_1 (m^3/s)
	Q_2	is streamflow measured at gauging station GS_2 (m^3/s)
	x	is length of reach 1 (m)
	y	is length of reach 2 (m)
	Q_{STP}	is STP effluent flow (m^3/s)

Table 4.7 shows the statistics of estimated incremental flows for each river reach determined from the 10 events (Table 3.9) to give an overall indication of the magnitude of flow used for this parameter group. However, the actual incremental flows of events were used in calibration and verification of YRWQM in Chapter 6. These incremental flows were also used for deriving power functions for group (b) reaches of Section 4.4.2. The zero flows shown in Table 4.7 are obtained mainly from the 6 'lower range' flow events, which proves that the nonpoint source flows are negligible for extremely low flows. Even the 'higher range' flows were not from storm events. Therefore, it can be said that the estimated incremental flows are mainly from base flow or groundwater flow, which generally should have good water quality.

4.4.4.2 Incremental concentration

Based on the discussion in Section 4.4.4.1, the estimation of incremental concentration may not be necessary, since the incremental flow is mainly contributed from groundwater for low flow events considered in this thesis, and can be considered to be of high quality. However, to prove this assumption, an estimation of incremental concentration was conducted for one of the highest flow event within the 'higher range' flow group of Table 3.9 (i.e. 26/7/95). This estimated incremental concentration was also used for uncertainty and sensitivity analysis in Chapter 5.

The estimation of the incremental concentration requires all inputs for parameter groups to be known with only the incremental concentration as the unknown. All inputs used in this computation (i.e. incremental flow, headwater flow and concentration, and point load flow and concentration) are for the 26/7/95 event, except for decay rate parameters where mean values were used. These mean values were determined in Section 5.4.1.

Table 4.7 Summary of Estimated Incremental Flow (m^3/s)

Reach	Min	Max	Mean
R1	0.008	0.106	0.055
R2	0.004	0.140	0.057
R3	0.006	0.312	0.126
R4	0.011	0.373	0.097
R5	0.006	0.376	0.096
R6	0.000	0.373	0.095
R7	0.000	0.000	0.000
R8	0.000	0.005	0.000
R9	0.003	0.134	0.034
R10	0.002	0.381	0.096
R11	0.011	0.366	0.095
R12	0.000	0.099	0.019
R13	0.000	0.568	0.086
R14	0.000	0.571	0.089
R15	0.000	0.334	0.069
R16	0.010	0.284	0.080
R17	0.000	0.284	0.096
R18	0.003	0.284	0.066
R19	0.000	0.164	0.050
R20	0.000	0.054	0.011
R21	0.000	0.087	0.008
R22	0.000	0.069	0.013
R23	0.000	0.060	0.009
R24	0.000	0.035	0.007
R25	0.000	0.013	0.003
R26	0.000	0.025	0.002
R27	0.000	0.007	0.001
R28	0.000	0.008	0.001
R29	0.000	0.117	0.020

Several YRWQM simulation runs were used to determine the incremental concentration by comparing computed nutrient (N and P) concentrations at the six EPAWQ measured points with the observations for this event. This was done by trial and error by changing incremental concentration of the reaches. The best match between computed nutrients with the observation yield the incremental concentration for the reaches. Temperature, DO and CBOD was not estimated, however average values were adopted from nearby water quality measurement stations for temperature and DO concentration.

CBOD concentration was not considered for incremental concentration, as it is mainly originated from point sources.

The trial and error estimation of incremental concentrations was conducted in six sequential simulations from upstream to downstream reaches beginning with EPAWQ point 6, as shown in Figure 4.13. This figure shows the river section between Upper Yarra headwater (H), and the EPAWQ point 6, with its four reaches. The next simulation deals with the river section from EPAWQ point 6 to EPAWQ point 5, and so on. In each of the 6 simulations, at least two sub-simulations were required to model TKN first and then TN, since TKN affect TN. The TP concentration can be modelled with TKN or TN, since TP is independent of TKN and TN. Once the set of incremental concentration for N and P were estimated for reaches 1 to 4 they were kept constant for the second simulation of river section between EPAWQ point 6 and EPAWQ point 5. This procedure was followed for all six river sections. The estimated incremental concentrations of respective river reaches is shown in Table 4.8 for event 26/7/95. The first column of this table shows the reach, the second column shows the water quality constituent and the last column shows estimated incremental concentration. Reaches downstream of Reach 24 were not shown in Table 4.8, since these reaches are downstream of EPAWQ point 1.

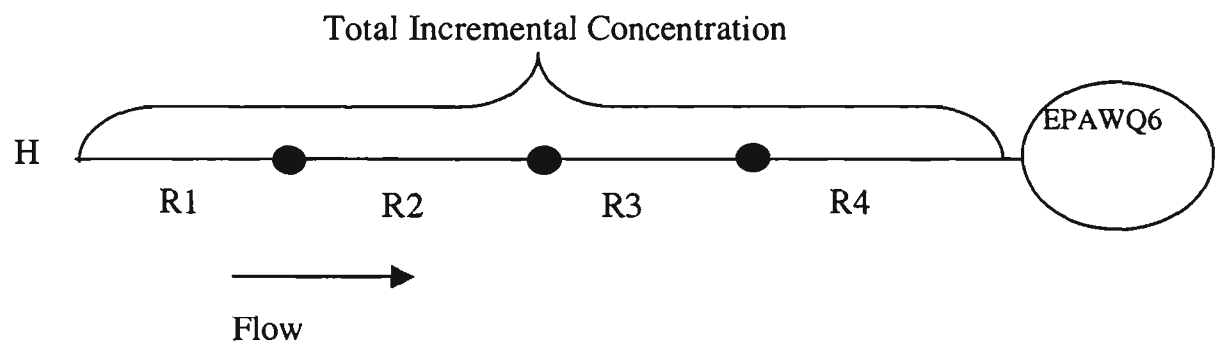


Figure 4.13 Incremental Concentration Estimation

As can be seen from Table 4.8, the incremental concentration was considered low, relative to headwater concentration of Table 4.10. This has proven the assumption that incremental concentration can be neglected in this study.

Table 4.8 Estimated Incremental Concentrations for Event 26/7/95

Reach	Water Quality Constituents	Incremental Concentration (mg/L)
R1-4		
	Org-N	0.050
	NH_3	0.003
	NO_2	0.001
	NO_3	0.024
	Org-P	0.002
	Diss-P	0.001
	Temperature (°C)	10
	DO	9
R5-10		
	Org-N	0.076
	NH_3	0.001
	NO_2	0.001
	NO_3	0.100
	Org-P	0.005
	Diss-P	0.001
	Temperature (°C)	10
	DO	8
R11-13		
	Org-N	0.294
	NH_3	0.006
	NO_2	0.003
	NO_3	0.330
	Org-P	0.001
	Diss-P	0.002
	Temperature (°C)	10
	DO	8
R14-18		
	Org-N	0.100
	NH_3	0.010
	NO_2	0.001
	NO_3	0.005
	Org-P	0.002
	Diss-P	0.005
	Temperature (°C)	11
	DO	8

Table 4.8 (Cont.) Estimated Incremental Concentrations for Event 26/7/95

Reach	Water Quality Constituents	Incremental Concentration (mg/L)
R19-23		
	Org-N	0.365
	NH_3	0.001
	NO_2	0.001
	NO_3	0.020
	Org-P	0.001
	Diss-P	0.008
	Temperature (°C)	11
	DO	6
R24		
	Org-N	0.500
	NH_3	0.012
	NO_2	0.005
	NO_3	0.050
	Org-P	0.002
	Diss-P	0.005
	Temperature (°C)	12
	DO	6

4.4.5 Headwater group

The most upstream end of the main river and tributaries are defined as headwater. Flows and concentrations delivered from headwaters are stored in the headwater input group. As can be seen from Figure 4.3, there are six headwater reaches that required headwater flow and concentration and they are:

- Upper Yarra,
- Woori Yallock Creek,
- Brushy Creek,
- Olinda Creek,
- Plenty River and
- Merri Creek.

4.4.5.1 Headwater flow

Based on personal communication with Ian Watsons of Melbourne Water Corporation (personal communication, 1999), a constant regulated flow of 0.116 m^3/s released from UYD was used as the headwater flow for Upper Yarra as it is the operational practice. The other headwater flows of tributaries used the data from nearby gauging stations. In these 5 headwaters located in the tributaries, adjustments were made to the measured flow at gauging stations by accounting for the effluent flow discharged from STPs upstream of gauging stations. As can be seen from Figure 4.3, the STPs on tributaries are upstream of the gauging stations. The location of the gauging stations was shown in Figure 4.3. The headwater flows were computed for all 10 selected flow events (Table 3.9) and the summary statistics are shown in Table 4.9. It should be noted that although the summary statistics are given in Table 4.9, the actual headwater flows of events were used in calibration and verification of YRWQM.

Table 4.9 Summary of Statistics of Headwater Flows

Headwater	Headwater flow (m^3/s)			Gauging Stations Used
	Min	Max	Mean	
Upper Yarra	0.116	0.116	0.116	Regulated Flow from UYD
Worri Yallock Creek	0.810	6.134	2.454	215
Olinda Creek	0.000	0.562	0.117	602
Brushy Creek	0.002	0.111	0.023	665
Plenty River	0.004	2.671	0.430	616
Merri Creek	0.012	4.607	0.681	149

4.4.5.2 Headwater concentration

The headwater concentrations were estimated for all headwaters except for Upper Yarra headwater using the water quality data from both EPAWQ and SWWQ stations shown in Figure 3.6. The data obtained from SWWQ stations were used as the principal source for estimating headwater concentration, because these monitoring networks have more stations located in the tributaries than EPAWQ. However, when data at SWWQ stations were not available, then close-by EPAWQ water quality stations were used. The headwater water quality concentrations were estimated for selected 10 flow events

(Table 3.9). The concentrations considered were all forms of N, P temperature, CBOD and DO.

The headwater concentrations for Upper Yarra was not estimated, but have used the measured water quality concentration of SWWQ station 17 (Figure 3.6), which is the closest to Upper Yarra headwaters. An assumption was made in this case that the water quality measured at station 17 was similar to the water quality in the Upper Yarra headwaters. This assumption is justified since there are no deteriorating pollutant sources (both point and non-point) in the area upstream of station 17.

The remaining five headwater concentrations were estimated using available SWWQ data. No water quality stations are located at the headwaters and majority of the stations that can be used for estimation of headwater concentrations are located near or at the confluence. Therefore, adjustments were made based on simple mass balance to determine the approximate headwater concentrations since water quality concentration obtained at these stations are considered higher than those of actual headwaters, because of input from effluent discharge and other pollutant sources. The estimation of headwater concentration is illustrated through Figure 4.14.

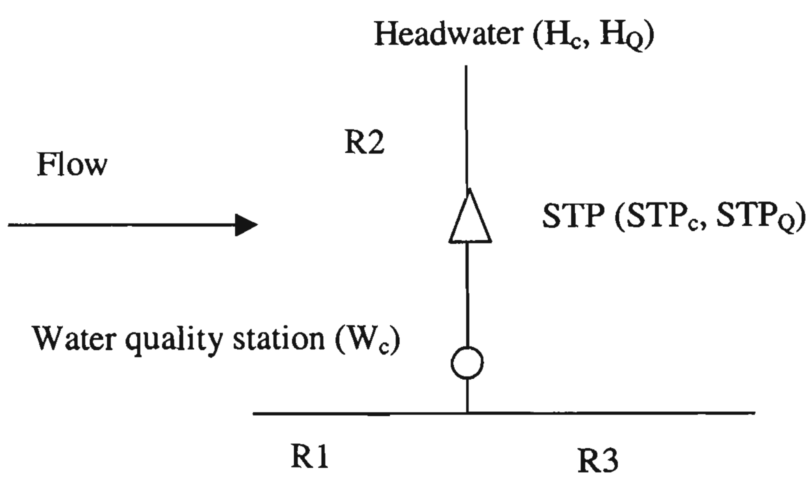


Figure 4.14 Estimation of Headwater Concentration

Figure 4.14 shows a segment of a river system with three reaches (i.e. R1, R2 and R3), which includes a tributary reach (R2). The water quality measurement point in this case is just below the STP. The symbols given in Figure 4.14 are defined after Equation

4.18. The headwater concentration can be determined using the mass balance equation (i.e. Equation 4.18).

$$Hc = \frac{Wc(H_Q + STP_Q) - (STP_c \bullet STP_Q)}{H_Q} \qquad 4.18$$

where H_c is headwater concentration (mg/L)

H_Q is headwater flow (m^3/s)

STP_Q is effluent flow (m^3/s)

STP_c is effluent concentration (mg/L)

W_c is water quality concentration measured at station (mg/L)

A summary statistics of the headwater concentrations are shown in Table 4.10. In general, the upstream headwaters (i.e. Upper Yarra and Worri Yallock Creek) have better water quality than downstream headwaters (i.e. Olinda Creek, Brushy Creek, Plenty River and Merri Creek). Total nitrogen (i.e. sum of all forms of nitrogen) and total phosphorus (sum of all forms of phosphorus) concentrations for all headwaters are above the recommended value of 0.75 mg/L and 0.06 mg/L of ANZECC (1999) respectively. The temperature is increasing from the Upper Yarra (upstream) towards downstream at Merri Creek. DO concentration of all headwaters is generally above the recommended value of 7.5 mg/L of ANZECC (1999). Again, it should be noted that although the summary statistics are given in Table 4.10, the actual headwater concentrations of events were used in calibration and verification of YRWQM.

Table 4.10 Summary of Estimated Range of Headwater Concentrations

Headwater	Water Quality Constituents	Headwater Concentration (mg/L)		
		Min	Max	Mean
Upper Yarra	CBOD	0.200	1.800	0.680
	Org-N	0.118	2.451	0.575
	NH_3	0.002	0.112	0.045
	NO_2	0.001	0.121	0.013
	NO_3	0.110	0.740	0.250
	Org-P	0.004	0.117	0.031
	Diss-P	0.002	0.102	0.023
	Temperature (°C)	8.120	14.000	9.234
	DO	9.000	16.700	13.300
Woori Yallock Creek	CBOD	0.520	1.800	0.696
	Org-N	0.212	0.882	0.439
	NH_3	0.004	0.104	0.033
	NO_2	0.001	0.301	0.043
	NO_3	0.720	2.300	1.331
	Org-P	0.005	0.067	0.029
	Diss-P	0.003	0.053	0.013
	Temperature (°C)	9.000	14.900	9.590
	DO	5.500	16.100	10.936
Olinda Creek	CBOD	0.000	3.200	2.300
	Org-N	0.241	1.100	0.740
	NH_3	0.008	1.000	0.139
	NO_2	0.004	0.170	0.035
	NO_3	0.120	3.600	1.059
	Org-P	0.001	0.203	0.082
	Diss-P	0.007	0.540	0.103
	Temperature (°C)	9.700	14.000	10.410
	DO	6.200	16.200	7.670
Brushy Creek	CBOD	1.400	6.900	4.581
	Org-N	0.107	2.058	1.343
	NH_3	0.015	3.900	0.833
	NO_2	0.004	1.270	0.174
	NO_3	0.010	6.000	3.687
	Org-P	0.001	3.400	0.855
	Diss-P	0.030	4.900	1.688
	Temperature (°C)	10.000	15.300	12.750
	DO	5.780	9.000	6.664

Table 4.10 (Cont.) Summary of Estimated Range of Headwater Concentrations

Headwater	Water Quality Constituents	Headwater Concentrations (mg/L)		
		Min	Max	Mean
Plenty River	CBOD	0.500	2.600	1.690
	Org-N	0.006	3.883	1.045
	NH_3	0.011	2.017	0.347
	NO_2	0.002	0.504	0.058
	NO_3	0.030	2.080	0.505
	Org-P	0.001	3.404	0.468
	Diss-P	0.004	2.116	0.254
	Temperature (°C)	8.200	15.100	11.030
	DO	4.000	11.300	6.770
Merri Creek	CBOD	1.000	5.600	2.020
	Org-N	0.560	1.860	0.876
	NH_3	0.052	1.115	0.190
	NO_2	0.010	0.516	0.065
	NO_3	0.020	1.260	0.597
	Org-P	0.001	0.166	0.054
	Diss-P	0.001	0.165	0.076
	Temperature (°C)	10.500	16.700	13.230
	DO	4.460	12.500	8.648

4.4.6 Point load group

4.4.6.1 Point load flow

The point load flows are defined as flows that are discharged into the river system or out of the system from a single point. In case of YRWQM, the point loads are due to STPs, diversions and tributaries that were not modelled as a river reach (referred to as 'unmodelled' tributary in this study). These are shown in Table 4.11. The discharges from STPs are important from concentration point of view, while unmodelled tributaries are important from flow point of view. As stated in Section 3.2.3, 13 old STPs were considered in YRWQM development, however, three of the old STPs (i.e. Ferres, Monbulk, and Symons) are located on the Woori Yallock Creek were combined as Woori Creek STP, since they are very close to each other. Diversion flow at Yering

Gorge was a major extraction point, therefore it was modelled as a loss of flow in Reach 15.

Table 4.11 Categories of Point Load Flows

Sewage Treatment Plants (STPs)	Diversions	Unmodelled Tributaries
Wesburn	Yering Gorge	Watts River
Yarra Junction		Watsons Creek
Woori Yallock		Diamond Creek
Woori Creek*		Darebin Creek
Seville		
Healesville		
Bluegum		
Lilydale		
Brushy		
Whittlesea		
Craigieburn		

* Ferres, Monbulk, and Symons STPs were combined as Woori Creek STP for this study.

Point load flows were either obtained from effluent flow data supplied by Yarra Valley Water Pty Ltd in case of STPs, or obtained from the gauging stations/diversion data. Diversion data were provided by Melbourne Water Corporation. A summary of the point load flows obtained from various data sources for the 10 selected flow events is shown in Table 4.12. Again, although the summary statistics are given in Table 4.12, the actual point load flows of events were used in calibration and verification of YRWQM.

4.4.6.2 Point load concentration

The point load concentrations from STPs were obtained from STP effluent data supplied by Yarra Valley Water Pty Ltd. The concentrations of unmodelled tributaries were obtained from the closest available water quality sampling station on its tributary, as shown in Figure 3.6. It is not necessary to estimate water quality concentrations of diversions, since they are modelled as losses and do not affect the water quality in the river system. However, the diversion flows have to be modelled. Summary statistics for point load concentrations for the 10 selected flow events are shown in Tables 4.13 and 4.14 for STPs and unmodelled tributaries respectively. Although the summary statistics are given in Tables 4.13 and 4.14, the actual point load concentrations of events were used in calibration and verification of YRWQM

Table 4.12 Summary of Point Load flows

Point Load	STP/Gauging Station Data Used	Point Load Flows (m^3/s)		
		Min	Max	Mean
Wesburn	STP	0.003	0.240	0.025
Yarra Junction	STP	0.081	2.598	1.504
Woori Yallock	STP	0.002	0.005	0.003
Woori Creek	STP	0.001	0.004	0.002
Seville	STP	0.001	0.002	0.001
Healesville	STP	0.003	0.008	0.005
Bluegum	STP	0.001	0.002	0.001
Lilydale	STP	0.046	0.085	0.065
Brushy	STP	0.098	0.118	0.107
Whittlesea	STP	0.003	0.006	0.004
Craigieburn	STP	0.021	0.031	0.026
Watts River	653	0.000	5.455	1.390
Watsons Creek	608	0.029	0.341	0.157
Diamond Creek	618	0.000	1.428	0.375
Darebin Creek	611	0.000	1.665	0.432
Yering Gorge	Diversion flow data	1.157	3.970	2.049

Table 4.13 Summary of Point Load Concentrations for STPs

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Wesburn	CBOD	2.000	6.000	2.900
	Org-N	0.900	4.000	1.760
	NH_3	0.100	3.500	1.277
	NO_2	0.007	0.110	0.060
	NO_3	0.020	7.900	2.377
	Org-P	3.500	8.400	5.621
	Diss-P	1.500	3.600	2.409
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500

Table 4.13 (Cont.) Summary of Point Load Concentrations for STPs

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Yarra Junction	CBOD	0.600	2.000	0.740
	Org-N	0.166	1.600	0.325
	NH_3	0.005	0.230	0.034
	NO_2	0.001	0.050	0.006
	NO_3	0.230	7.000	0.924
	Org-P	0.016	5.250	0.557
	Diss-P	0.003	2.250	0.231
	Temperature (°C)	11.700	16.000	15.140
	DO	5.500	5.500	5.500
Woori Yallock	CBOD	1.000	7.000	2.950
	Org-N	0.300	8.000	2.450
	NH_3	0.500	9.000	3.960
	NO_2	0.040	0.370	0.130
	NO_3	0.280	6.400	3.263
	Org-P	1.820	8.400	4.879
	Diss-P	0.780	3.600	2.091
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500
Woori Creek	CBOD	2.000	6.600	4.150
	Org-N	0.003	3.300	1.537
	NH_3	0.001	6.223	1.345
	NO_2	0.001	0.820	0.118
	NO_3	0.023	9.700	3.195
	Org-P	0.002	6.510	2.798
	Diss-P	0.001	2.790	1.339
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500
Seville	CBOD	1.000	9.000	3.250
	Org-N	0.800	4.000	2.500
	NH_3	0.500	9.700	3.390
	NO_2	0.040	0.200	0.122
	NO_3	0.160	9.600	4.655
	Org-P	2.030	7.700	5.195
	Diss-P	0.870	4.830	2.369
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500

Table 4.13 (Cont.) Summary of Point Load Concentrations for STPs

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Healesville	CBOD	1.000	5.000	2.900
	Org-N	0.800	3.000	1.580
	NH_3	0.200	11.000	1.470
	NO_2	0.010	0.200	0.085
	NO_3	0.060	8.600	2.816
	Org-P	0.259	5.320	3.148
	Diss-P	0.111	2.280	1.349
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500
Bluegum	CBOD	2.000	9.000	4.000
	Org-N	1.800	3.300	2.456
	NH_3	0.100	13.000	7.044
	NO_2	0.020	0.480	0.184
	NO_3	1.300	11.000	4.956
	Org-P	4.600	7.000	5.792
	Diss-P	1.740	3.000	2.457
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500
Lilydale	CBOD	1.200	11.000	7.320
	Org-N	3.200	10.000	6.190
	NH_3	1.600	18.000	7.690
	NO_2	0.380	1.900	0.726
	NO_3	0.100	2.000	0.691
	Org-P	0.010	2.030	0.681
	Diss-P	0.084	0.870	0.331
	Temperature (°C)	11.100	16.000	15.510
	DO	5.500	5.500	5.500
Brushy	CBOD	2.000	8.000	3.500
	Org-N	0.500	4.000	1.800
	NH_3	0.800	11.000	5.610
	NO_2	0.010	0.810	0.265
	NO_3	0.450	15.000	3.875
	Org-P	0.050	4.900	2.819
	Diss-P	0.420	2.100	1.151
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500

Table 4.13 (Cont.) Summary of Point Load Concentrations for STPs

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Whittlesea	CBOD	3.000	10.000	5.300
	Org-N	0.200	4.200	2.080
	NH_3	0.500	14.000	5.730
	NO_2	0.040	3.800	0.714
	NO_3	0.300	8.600	4.880
	Org-P	0.350	7.700	2.151
	Diss-P	0.150	3.300	0.929
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500
Craigieburn	CBOD	2.000	10.000	5.200
	Org-N	1.000	8.800	3.390
	NH_3	0.400	8.300	3.020
	NO_2	0.000	0.590	0.231
	NO_3	0.500	5.300	2.817
	Org-P	0.490	6.720	2.879
	Diss-P	0.210	3.350	1.311
	Temperature (°C)	16.000	16.000	16.000
	DO	5.500	5.500	5.500

Table 4.14 Summary of Point Load Concentrations for Unmodelled Tributaries

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Watts River	CBOD	0.500	2.000	1.130
	Org-N	0.095	0.685	0.287
	NH_3	0.004	0.038	0.021
	NO_2	0.001	0.016	0.005
	NO_3	0.200	0.670	0.397
	Org-P	0.012	0.043	0.027
	Diss-P	0.002	0.018	0.008
	Temperature (°C)	6.800	16.300	12.410
	DO	6.530	12.500	9.253

Table 4.14 (Cont.) Summary of Point Load Concentration for Unmodelled Tributaries

Point load	Water Quality Constituent	Point Load Concentrations (mg/L)		
		Min	Max	Mean
Watsons Creek	CBOD	0.800	4.200	2.250
	Org-N	0.141	2.383	0.627
	NH_3	0.002	0.028	0.012
	NO_2	0.000	0.502	0.054
	NO_3	0.020	0.530	0.142
	Org-P	0.004	0.125	0.034
	Diss-P	0.002	0.034	0.009
	Temperature (°C)	8.100	16.000	13.080
	DO	6.600	11.200	8.748
Diamond Creek	CBOD	1.400	4.000	2.820
	Org-N	0.325	2.367	0.799
	NH_3	0.009	0.092	0.027
	NO_2	0.002	0.021	0.008
	NO_3	0.140	0.370	0.236
	Org-P	0.026	0.191	0.067
	Diss-P	0.006	0.039	0.016
	Temperature (°C)	7.900	16.000	13.100
	DO	5.000	11.600	7.720
Darebin Creek	CBOD	1.000	4.600	2.520
	Org-N	0.396	1.599	0.904
	NH_3	0.001	0.077	0.021
	NO_2	0.002	0.021	0.008
	NO_3	0.040	1.000	0.471
	Org-P	0.006	0.200	0.061
	Diss-P	0.019	0.150	0.064
	Temperature (°C)	7.800	18.000	13.760
	DO	6.850	11.900	9.215

Temperature data were not available for STPs, but based on discussions with Julie Baud of Yarra Valley Water Pty Ltd (personal communication, 1999), a constant temperature was assumed for these STP effluent and shown in Table 4.13. Again, no data on effluent DO discharge from STPs were available for the 10 flow events, except for Whittlesea STP on few occasions. The effluent DO concentration of Whittlesea STP was found to be around 5-6 mg/L. Therefore, a constant 5.5 mg/L of DO effluent concentration was adopted for all STPs (including Whittlesea STP) in this study. In general, the Lilydale STP discharges the highest level of TN concentration with ammonia contributing the greatest. The Brushy Creek STP discharges the highest level of TP concentration.

On average, the TN concentration measured at Watts River is the only unmodelled tributary within the recommended limit of 0.75 mg/L of ANZECC (1999). The average TP concentrations for both Watts River and Watsons Creek are also within the recommended limit of 0.06 mg/L specified in ANZECC (1999).

4.5 Summary

Successful model development requires 5 essential stages:

- Data collection and model selection
- Assembly of the model
- Pre-calibration uncertainty and sensitivity analysis of model parameters
- Calibration and verification of model parameters
- Post-calibration uncertainty and sensitivity analysis of model parameters

The assembly of the model is one of the important stages in the overall model development. It was considered in this chapter, emphasising on data inputs. QUAL2E software was used to assemble the Yarra River Water Quality Model (YRWQM). The Yarra River catchment was first discretised into a number of reaches from Upper Yarra Dam (UYD) at the upstream end to Dights Falls (DFS) at the downstream end. The criteria used for discretisation were based on the locations of sewage treatment plants (STPs), the confluence of tributaries and the locations of the Environment Protection Authority Water Quality (EPAWQ) stations. When the reaches defined based on the above criteria was long, they were further sub-divided. In total, 29 reaches were considered for the Yarra River including main tributaries, and each reach was then subdivided into 1-km length computational elements, which provide sufficient resolution for water quality modelling.

Data preparation of QUAL2E was done through six parameter groups, namely global, hydraulic, reaction, incremental, headwater and point load. The global group is responsible for data which does not vary from reach to reach. The hydraulic, incremental, headwater and point load inputs are dependent on flow events. The reaction group is essentially the decay rates of major nonconservative water quality constituents and should not vary from event to event, unless the composition of effluent quality changes.

The global group allows various simulation options (e.g. nutrient modelling and algae on/off), assigns units, specifies temperature correction factors for decay rates and defines physical characteristics of the river system such as number of reaches. Default values as specified in QUAL2E user manual were used as the input, since this parameter group has been found as an insensitive parameter group in this study.

The hydraulic group is responsible for data inputs in determining depth and velocity for various flows. The power function method was selected in this study because less unknown variables are required for this method as compared to the Manning equation method. The power functions were determined for 29 reaches based on three groups. These groups are:

(a) Reaches that have gauging stations on them with stage-discharge rating curves and cross-sectional details,
(b) Reaches that do not have gauging stations on them,
(c) Tributaries that do not have gauging stations on them.

For group (a) reaches, the constants and exponents were obtained by considering cross-sectional details and stage-discharge rating curves at the gauging station within the reach. A linear interpolation of the power functions of the two closest gauging stations was used for group (b) reaches. For tributaries that have no gauging stations (i.e. group (c) reaches), the estimated power functions from a nearby reach were used.

The reaction parameter group is responsible for decay and reaeration rates. A preliminary estimation analysis of decay rates was conducted in this study using water quality data of 6 selected 'lower range' flow events and the first order decay rate equation. These estimated decay rates and the rates compiled from the literature were used to select ranges of decay rates for use in calibration of YRWQM.

Several reaeration rate methods were available in QUAL2E to determine the rate of oxygen input in the river. The QUAL2E built-in reaeration rate method was selected because it can determine reaeration rates as a function of depth and velocity during simulation. To determine the most suitable method for use in YRWQM, an analysis using the 10 selected flow events was conducted by considering the velocity-depth plot developed by Covar (1976). The Covar plot provides the most suitable (or appropriate) reaeration method for the river reach under consideration. It was found that O'Connor and Dobbins, and Owens-Gibbs are the two most suitable methods for use in YRWQM.

Both flows and concentrations are required as input for the incremental parameter group. Any flows and concentrations that are not delivered via a single point is considered as incremental flows and concentrations. The estimation of incremental flows for all 10 selected flow events was based on flow volume balance considering flows at upstream and downstream gauging stations, STP effluent flow, tributary inflows and diversions. It was found that estimated incremental flows were mainly contributed from groundwater flows, since the flow events considered were low flows. Generally, the groundwater quality is high and therefore it was considered not necessary to estimate the incremental concentration. This was verified by computing incremental concentration for one of the highest flow events. It was also found that the incremental concentrations were of better water quality compared to the headwater concentrations.

The headwater parameter group is responsible for flows and concentrations of the most upstream end of the main river and its tributaries. The measured flow from the closest gauging station located near the headwaters together with appropriate adjustments for effluent inflow was used as the headwater flow for the five tributaries. A constant regulated flow was used as the headwater flow for main Yarra River. The concentration for headwaters was estimated from the closest EPAWQ and StreamWatch Water Quality (SWWQ) stations by considering the incoming concentration from STPs, through mass balance.

The point load parameter group is responsible for input of STPs, diversions and unmodelled tributaries. The point load flows and concentrations discharged from STPs were supplied as STP effluent data. The point load flows and concentrations measured at the closest gauging stations and water quality measurement stations on the tributary were used as the data entry for unmodelled tributaries. A major extraction at Yering Gorge was considered as negative point flow.

CHAPTER 5

PRE-CALIBRATION UNCERTAINTY AND SENSITIVITY ANALYSIS OF MODEL PARAMETERS

5.1 Introduction

Due to complex natural processes in river systems, developing a mathematical model involves certain assumptions, simplifications and approximations in representing the actual river system. The extent of these assumptions, simplifications and approximations in the model is termed error or uncertainty. The effect on the model output caused by this error is defined as sensitivity. Therefore, these two terms are related and hence they have been studied together in previous studies (e.g Lei and Schilling, 1996).

The types of uncertainties inherent in the model can be summarized into 4 groups:

- Model structure uncertainty
- Data uncertainty
- Operational uncertainty
- Model parameter uncertainty

The model structure uncertainty deals with the error associated with interpretation and transformation of natural river processes in the model algorithms. Generally this uncertainty can be reduced by selecting a standard model (Shanahan *et al.*, 1998), such as QUAL2E (Brown and Barnwell, 1987). Data uncertainty is mainly caused by the inaccuracy in data handling and sampling errors. This can be reduced through a well-structured data acquisition program to obtain good quality data and by conducting an extensive analysis of the data (obtained from the data acquisition program) so that the unexplainable data are not used in modelling. The operational uncertainty concerns with human errors associated with the actual usage of the model, and in general can be controlled through careful data input and model usage. The last uncertainty deals with the error associated with model parameters and is considered in this study.

The uncertainty/sensitivity analysis of model parameters can be conducted in two stages in developing a mathematical model, namely, pre-calibration and the post-calibration. The pre-calibration uncertainty/sensitivity analysis can identify both sensitive and insensitive parameters prior to model calibration. This allows modellers to expend less effort and time on the insensitive parameters and more effort on sensitive parameters, to obtain a good calibration during the calibration phase. The post-calibration sensitivity analysis can be used to quantify the effect of changes of input parameters from the calibrated values on the output results. In general, many studies (e.g. Brown, 1987; Melching and Anmangandla, 1993 and Drolc and Koncan, 1996) have used post-calibration sensitivity analysis of model parameters of river water quality models, but no such river water quality modelling studies were found in the literature related to pre-calibration.

As discussed in Section 2.7, many methods are available to determine uncertainty/ sensitivity of model parameters. Of the methods reviewed in Section 2.7, the 'one-at-a-time', the first order error analysis (FOEA) and the Monte Carlo simulation (MCS) are built-into QUAL2E. As concluded in Section 2.8, the MCS was used for the assessment of pre-calibration uncertainty/sensitivity analysis of model parameters of Yarra River Water Quality Model (YRWQM). The MCS is more appropriate for applications where no-prior knowledge is available on the likely values of the optimum parameters and therefore the analysis is conducted by considering the whole input parameters domain. The YRWQM uncertainty/sensitivity analysis falls into this category.

The MCS of QUAL2E performs uncertainty/sensitivity analysis by considering model inputs in six different parameter groups. Each of these groups consists of several parameters (within the group) as listed below.

1. Global parameter group	14 parameters
2. Hydraulics parameter group	6 parameters
3. Reaction parameter group	13 parameters
4. Incremental parameter group	8 parameters
5. Headwater parameter group	9 parameters
6. Point load parameter group	9 parameters

In this chapter, the objectives and overall methodology are first described. Event selection used for this study together with the estimation of various input data for MCS are then discussed. The output analysis used to determine sensitivity is then described followed by the results and discussions of the analysis. Finally, the conclusions drawn from this analysis are presented.

5.2 Objectives and Overall Methodology

Two major objectives were considered in this chapter and they were:

- To identify sensitive input parameter groups so that additional effort and attention can be given to these groups in collecting accurate and reliable data.
- To identify the most sensitive parameters within the reaction parameter group so that more time and effort can be put into sensitive parameters during the model calibration compared to the insensitive parameters. Attention was focused on the reaction parameter group because the parameters within this group can only be identified through model calibration, unlike the parameters in the other groups, which can be obtained by data transformation and analysis.

To achieve the above objectives, two flow events were first selected, one from each of the 'lower range' and 'higher range' flows of Section 3.6 (or Table 3.9). The event selection is described in Section 5.3. The input data that are required for MCS was then determined. These input data included mean, coefficient of variations (CV_{in}), and probability density function (PDF) of input parameters of each parameter group, and the number of simulation runs in MCS. Once these data inputs were determined, MCS studies related to the two objectives were undertaken. The MCS in QUAL2E samples input parameters randomly from their distributions, runs the simulation model and analyses the output distributions. Due to the QUAL2E program setting, the methodologies used to achieve the two objectives were slightly different from each other and are discussed below.

Methodology for first objective

One parameter group at a time was considered for this MCS study. The input parameters of this group were simultaneously generated randomly from their distributions within their CV_{in} values, while the input parameters of the other groups were held constant at their mean values with $CV_{in}=0$. Three CV_{in} values were considered in this study to investigate the effect of CV_{in} of input parameters on the sensitivity of output, and these CV_{in} values were obtained from QUAL2E (Brown and Barnwell, 1987). Details of the 3 CV_{in} values used are described in Section 5.4.2.

Methodology for second objective

First of all, the input parameters of all groups except the reaction parameter group were kept constant at their mean values with $CV_{in}=0$. Then, one reaction parameters (within the reaction parameter group) at a time was considered and values generated randomly from its distribution within its CV_{in}, while the other reaction parameters were held constant at their mean values with $CV_{in}=0$. Again, as for the first objective, three CV_{in} values were considered.

The results of MCS (i.e. QUAL2E simulation results obtained from various input parameters) were analysed using the relative deviation ratio (RDR) to study the output sensitivity for both objectives. Details of this analysis are described in Section 5.5.

5.3 Event and Reach Selection

As mentioned in Section 5.2, one 'lower range' and one 'higher range' flow event (Section 3.6, Table 3.9) were considered for pre-calibration uncertainty/sensitivity analysis. These flow events were selected in order to study the uncertainty/sensitivity of model parameters under different low flow events. The characteristics of the two selected flow events are given in Table 5.1.

Table 5.1 Characteristics of Flow Events Selected for Pre-Calibration Uncertainty/Sensitivity Analysis

Event	Streamflow at Downstream Boundary (m^3/s)	Flow Group
3/11/95	7.4	'lower range'
26/7/95	40.0	'higher range'

QUAL2E allows up to five user-specified locations in the river network for output analysis. Four critical reaches (i.e. Reaches 5, 15, 18 and 24), as indicated in Figure 5.1, were selected to assess the uncertainty/sensitivity results in YRWQM. These reaches were considered since they experience significant water quality deterioration through effluent discharges.

5.4 Inputs Required for Monte Carlo Simulation

To conduct MCS, four inputs are required, as listed below.

- Mean values of input parameters (χ),
- Coefficient of variation values of input parameters (CV_{in}),
- Probability density function of input parameters (PDF) and
- Number of simulation runs.

Each of these inputs required for the two objectives stated in Section 5.2 is described in Sections 5.4.1-5.4.4.

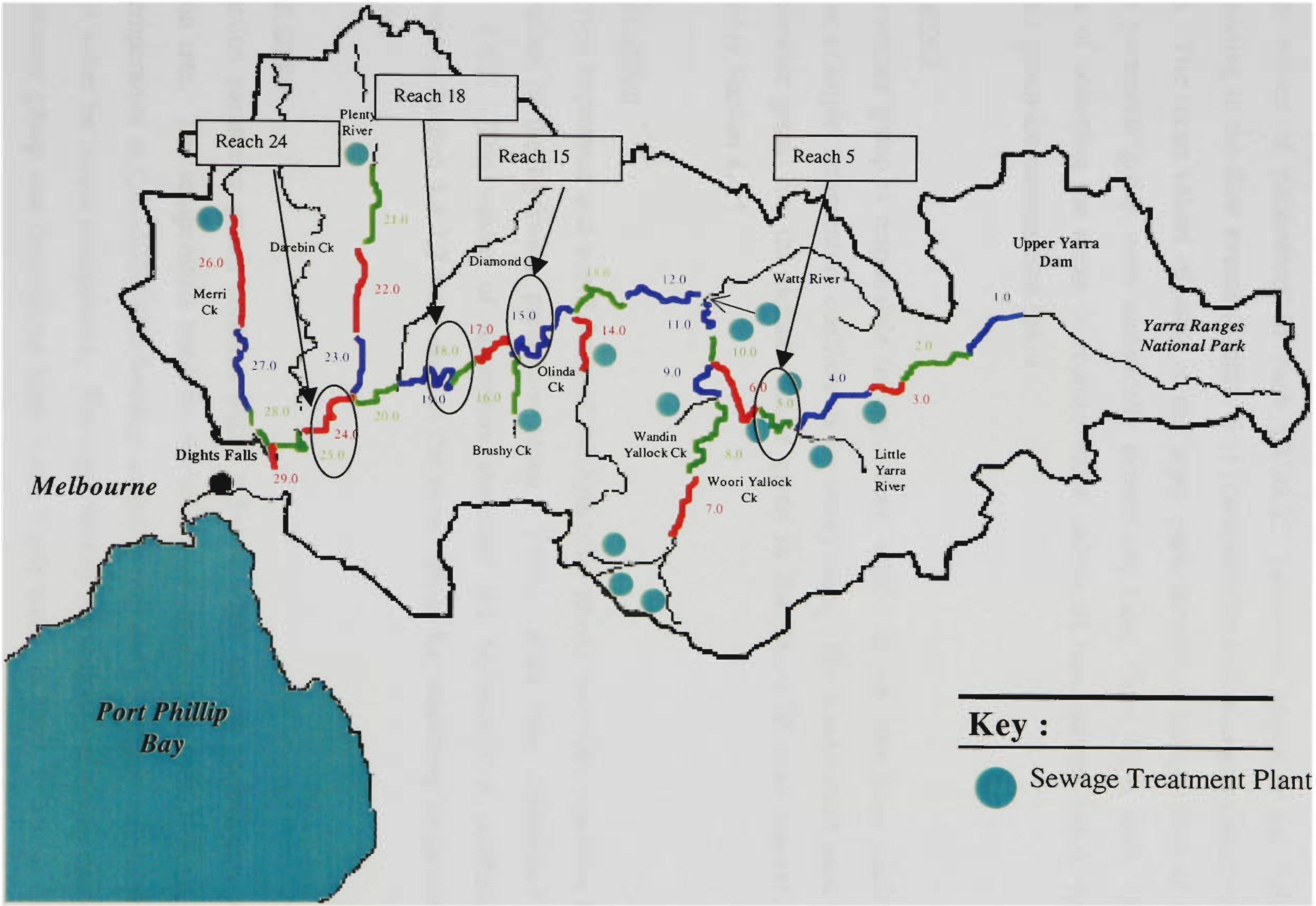


Figure 5.1 Selected River Reaches for Pre-Calibration Uncertainty and Sensitivity Analysis

5.4.1 Mean values of input parameters

In MCS, all inputs and outputs are measured relative to the mean value. Although QUAL2E defines these values as mean values, they are representative values. Majority of mean values of parameters in each QUAL2E parameter group is the values corresponding to the flow events (Section 5.3) considered in the uncertainty/sensitivity analysis. The mean values of most of the input parameters (i.e. decay rates) of the reaction parameter group were statistically determined from Yarra River data. The methods of obtaining the mean or representative values of input parameters of each parameter group are summarised below.

<u>Global group</u>

This parameter group is responsible for input data, which do not vary from reach to reach, for example, temperature coefficients for decay rates. The mean values used for this parameter group are the default values given in the QUAL2E user manual, as discussed in Section 4.4.1.

<u>Hydraulic group</u>

The derived exponents and coefficients of the hydraulic power functions represent the mean values in the hydraulic parameter group. These values were determined in Section 4.4.2. The values of dispersion constant and Manning's n coefficient determined in Section 4.4.2.2 were used as the mean values for modelling longitudinal dispersion.

<u>Reaction group</u>

The reaction parameter group contains decay rates of water quality constituents and reaeration rate. The temperature used for correction of decay rates (which is called initial temperature in QUAL2E) also belongs to this group and 20 degree was used as the mean value for initial temperature. The mean values for majority of decay rates in this parameter group was determined from Yarra River data. In total, 11 decay rates and reaeration rate are in the reaction group.

For decay rates of $CBOD_d$, $NH_{3\text{-}d}$, $NO_{2\text{-}d}$, Org-N_d, Org-P_d, $CBOD_s$, Org-N_s and Org-P_s, the mean values were computed from the values estimated from Yarra River data, as explained in Section 4.4.3. These mean values were found to be within the range published in Bowie *et al.* (1985). For decay rates of SOD, NH_3ben and Diss-Pben, the mean values were computed from decay rates obtained from other studies published in Bowie *et al.* (1985), as no observed data was available for Yarra River to compute these decay rates. The arithmetic average of the decay rates was used as the mean values for these 11 decay parameters.

The method used for the determination of reaeration rate (K2) in YRWQM was discussed in Section 4.4.3.2. It specified a method such as O'Connor and Dobbins for each river reach. During simulation run time, QUAL2E computes the reaeration rate. However, this method could not be used in this uncertainty/sensitivity analysis, since it requires the mean CV_{in} and PDF of the reaeration rate. Therefore, the reaeration rates were computed for all reaches of YRWQM considering 10 selected flow events in Section 3.6 (or Table 3.9). Power functions developed for each river reach (Section 4.4.2.1) were used to determine velocity and depth for each flow event at each river reach and the corresponding reaeration rate method in YRWQM was used to compute the reaeration rates. The arithmetic average of these reaeration rates was considered as the mean. Similar to the means of 8 decay rates estimated from Yarra River data, the reaeration rate mean obtained from Yarra River data was found to be within the range of the values obtained from previous studies published in Bowie *et al.* (1985). In this analysis, a single value of reaeration rate was considered for each river reach. These reaeration rate values are shown in Figure 5.2 (labelled 'Yarra') together with the values obtained from previous studies obtained from Bowie *et al.* (1985) (labelled 'LIT').

A summary of the mean values of decay rates and reaeration rates used for MCS is shown in Table 5.2. It also shows the CV_{in} values computed from the data used to compute the means, the number of data points used and the data source.

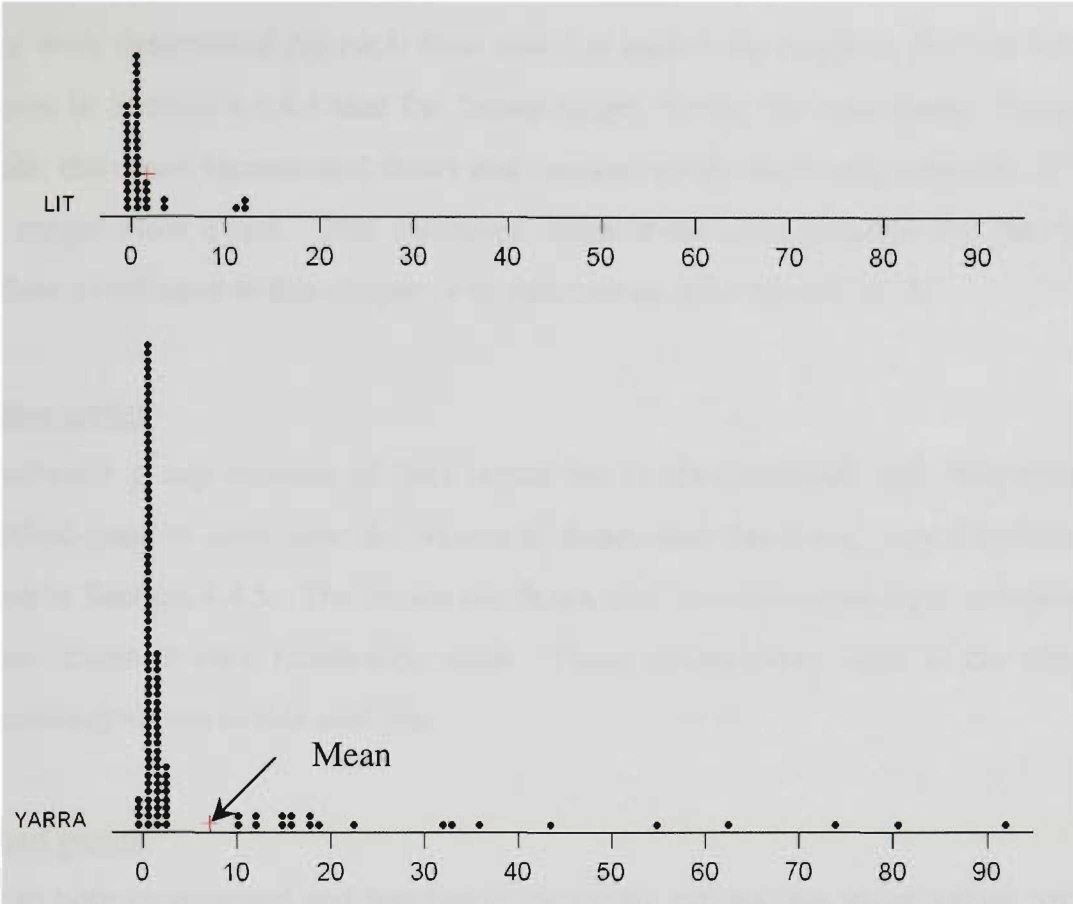


Figure 5.2 Mean Reaeration Rate Used in Monte Carlo Simulation

Table 5.2 Mean and Coefficient of Variation of Decay Rates and Reaeration Rate Estimated from Yarra River Data and Previous Studies

Parameter	Symbols	Number of points	Mean (Per day)	CV_{in}	Data Source
CBOD decay rate	$CBOD_d$	15	0.240	1.546	Yarra River Data
CBOD settling	$CBOD_s$	14	1.834	1.743	Yarra River Data
SOD	SOD	53	0.620	5.301	Bowie et al. (1985)
Reaeration rate	K2	112	6.861	2.720	Yarra River Data
Org-N decay	Org-N_d	23	0.178	0.500	Yarra River Data
Org-N settling	Org-N_s	23	1.110	1.087	Yarra River Data
NH_3 decay	$NH_{3\text{-}d}$	23	0.317	0.526	Yarra River Data
NH_3 benthos	NH_3ben	14	0.220	0.288	Bowie et al. (1985)
NO_2 decay	$NO_{2\text{-}d}$	23	0.437	1.198	Yarra River Data
Org-P decay	Org-P_d	23	0.167	1.457	Yarra River Data
Org-P settling	Org-P_s	15	0.042	1.076	Yarra River Data
Diss-P benthos	Diss-Pben	13	0.293	1.182	Bowie et al. (1985)

<u>Incremental group</u>

The incremental group consists of inputs for incremental flows and concentrations of each river reach. The values determined for each flow event represent the mean value, and they were determined for each flow event at each river reach in Section 4.4.4. It was shown in Section 4.4.4.1 that for 'lower range' flows, the incremental flows were negligible, therefore, incremental flows and concentrations were only considered for the 'higher range' flow event. The estimated incremental concentration for the 'higher range' flow event used in this chapter was determined in Section 4.4.4.2.

Headwater group

The headwater group consists of data inputs for headwater flows and concentrations. The method used to determine the values of headwater flows and concentrations was described in Section 4.4.5. The headwater flows and concentrations were computed for each flow event at each headwater reach. These values were used as the mean (or representative) values in this analysis.

Point load group

Similar to both incremental and headwater parameter groups, the mean values for point load group are also event dependent. The point load flows and concentration were computed for all STPs and unmodelled tributaries corresponding to flow events in Section 4.4.6 and these values were used as mean (or representative) value for MCS analysis.

5.4.2 Coefficient of variation of input parameters

The coefficients of variation (CV_{in}) of input parameters are required in MCS so that the uncertainty (or error) of the input parameters can be quantified relative to their mean values. They are also needed to sample input parameter values from the distribution. The coefficient of variation is defined as *the ratio of standard deviation to mean.* As stated in Brown and Barnwell (1987), CV_{in} values for input parameters were not widely available. An attempt was therefore made to determine appropriate CV_{in} values of the decay rates using Yarra River data when available and using other data published in Bowie *et al.* (1985) for decay rates in which Yarra River data were not available. Table 5.2 shows these computed CV_{in} values for 11 decay rates and reaeration rate.

Majority of CV_{in} values in Table 5.2 seemed to be extremely high, except for $NH_{3\text{-}d}$, Org-N_d and NH_3ben. Possible reasons could be the outliers of the data set, which cause large deviation from the mean value. Chadderton *et al.* (1982) stated that determination of standard deviation (and hence coefficient of variation) is a difficult task as it is a second order moment and requires extensive monitoring data. Due to the unrealistic CV_{in} values in Table 5.2, they were not used in this analysis, but the CV_{in} values suggested in Brown and Barnwell (1987) were used for the reaction parameter group. This means the CV_{in} values suggested by Brown and Barnwell (1987) were used for all 6 parameter groups.

Different CV_{in} ranges were suggested by Brown and Barnwell (1987) for different parameter groups and sometimes even within one parameter group, different CV_{in} values were suggested for different input parameters. Three CV_{in} values were considered for each input parameter in each parameter group. They correspond to mean, minimum and maximum of the 'typical' range suggested in Brown and Barnwell (1987). These values are shown in Table 5.3. The mean values of the 'typical' range were considered as the base CV_{in}.

Table 5.3 Input CV Used for MCS in YRWQM

Input Variable Name	Mean (or Base)	Minimum	Maximum	Number Used in Figures 5.6-5.13
Global Group				
Oxygen uptake by NH_3 Oxidation	0.100	0.050	0.200	1
Oxygen uptake by NO_2 Oxidation	0.100	0.050	0.200	2
Temp coefficient BOD Decay	0.030	0.020	0.050	3
Temp coefficient BOD Settling	0.030	0.020	0.050	4
Temp coefficient Reaeration	0.030	0.020	0.050	5
Temp coefficient SOD	0.030	0.020	0.050	6
Temp coefficient Org-N Decay	0.030	0.020	0.050	7
Temp coefficient Org-N Settling	0.030	0.020	0.050	8
Temp coefficient NH_3 Decay	0.030	0.020	0.050	9
Temp coefficient NH_3 Benthos	0.030	0.020	0.050	10
Temp coefficient NO_2 Decay	0.030	0.020	0.050	11
Temp coefficient Org-P Decay	0.030	0.020	0.050	12
Temp coefficient Org-P Settling	0.030	0.020	0.050	13
Temp coefficient Diss-P Benthos	0.030	0.020	0.050	14

Table 5.3 (Cont.) Input CV Used for MCS in YRWQM

Input Variable Name	Mean (or Base)	Minimum	Maximum	Number Used in Figures 5.6-5.13
Hydraulic Group				
Dispersion constant	0.200	0.100	0.500	15
Coefficient on flow for velocity	0.050	0.010	0.150	16
Exponent on flow for velocity	0.001	0.0005	0.010	17
Coefficient on flow for depth	0.050	0.010	0.150	18
Exponent on flow for depth	0.001	0.0005	0.010	19
Manning's roughness n	0.100	0.050	0.150	20
Reaction Group				
$CBOD_d$	0.175	0.100	0.250	21
$CBOD_s$	0.175	0.100	0.250	22
SOD	0.175	0.100	0.250	23
Reaeration (K2)	0.175	0.100	0.250	24
$Org\text{-}N_d$	0.175	0.100	0.250	25
$Org\text{-}N_s$	0.175	0.100	0.250	26
$NH_{3\text{-}d}$	0.175	0.100	0.250	27
NH_3ben	0.175	0.100	0.250	28
$NO_{2\text{-}d}$	0.175	0.100	0.250	29
$Org\text{-}P_d$	0.175	0.100	0.250	30
$OrgP_s$	0.175	0.100	0.250	31
Diss Pben	0.175	0.100	0.250	32
Initial Temperature	0.175	0.100	0.250	33
Incremental Group				
Incremental Flow	0.050	0.010	0.150	34
Incremental-DO	0.050	0.020	0.100	35
Incremental-Org-N	0.150	0.100	0.300	36
Incremental-NH_3	0.150	0.100	0.300	37
Incremental-NO_2	0.150	0.100	0.300	38
Incremental-NO_3	0.150	0.100	0.300	39
Incremental-Org P	0.150	0.100	0.400	40
Incremental-Diss-P	0.150	0.100	0.400	41
Headwater Group				
Headwater Flow	0.050	0.010	0.150	42
Headwater-DO	0.050	0.020	0.100	43
Headwater-CBOD	0.100	0.050	0.200	44
Headwater-Org -N	0.150	0.100	0.300	45
Headwater-NH_3-N	0.150	0.100	0.300	46
Headwater-NO_2	0.150	0.100	0.300	47

Table 5.3 (Cont.) Input CV Used for MCS in YRWQM

Input Variable Name	Mean (or Base)	Minimum	Maximum	Number Used in Figures 5.6-5.13
Headwater-NO_3	0.150	0.100	0.300	48
Headwater-Org P	0.150	0.100	0.400	49
Headwater Diss P	0.150	0.100	0.400	50
Point Load Group				
Point Load Flow	0.050	0.010	0.150	51
Point load-DO	0.050	0.020	0.100	52
Point load-CBOD	0.100	0.050	0.200	53
Point load Org N	0.150	0.100	0.300	54
Point load-NH_3	0.150	0.100	0.300	55
Point load-NO_2	0.150	0.100	0.300	56
Point load-NO_3	0.150	0.100	0.300	57
Point load-Org P	0.150	0.100	0.400	58
Point load Diss P	0.150	0.100	0.400	59

5.4.3 Probability density function

Two types of probability density function (PDFs) are available in QUAL2E, namely normal and lognormal, to obtain a statistical distribution for input parameters. Both distributions were initially considered to study the effect of input parameter distribution on output sensitivity.

5.4.4 Number of simulation runs

Number of QUAL2E simulation runs is required as input to MCS, so that the model parameters can repeatedly selected randomly from the input parameter distributions. The selection of the number of simulation runs can be subjective and depends on the required accuracy of the output distribution.

Burges and Lettenmaier (1975) used 2,000 runs in their water quality modelling study, and commented that it was adequate. They also stated that beyond 2,000 runs, there were minor changes in the mean and variance of output water quality responses. Both Scavia *et al.* (1981) and Song and Brown (1990) used 1,000 runs in their MCS. They commented that this number of runs was considered adequate relative to output results and computer time. Brown and Barnwell (1987) claimed that 2,000 simulations were sufficient to obtain accurate estimates of the output standard deviations in QUAL2E. Based on above findings, 2,000 runs were used in this study.

5.5 Output Analysis in Identifying Sensitive Parameters

QUAL2E produces the following outputs from MCS for each of the water quality constituents (e.g. DO) at each location of the river reach specified.

- Mean value
- Coefficient of variation (CV_{out})
- Minimum and maximum values
- Skewness coefficient
- Cumulative Frequency Distribution (CFD)

These outputs should be analysed to determine and interpret parameter sensitivity. Hamby (1994b) compiled an excellent and concise summary of the uncertainty analysis indicators used for most common uncertainty and sensitivity analysis methods and has been discussed in Section 2.7.2. Such indicators for MCS include the correlation coefficient (CC), regression analysis (RA) and relative deviation ratio (RDR). Of these three indicators, RDR was used in this study because both CC and RA require the listing of the randomly generated values of input parameters and QUAL2E does not provide such listing.

RDR was defined as the ratio of output CV to input CV. A large value of the RDR, indicates that the output response is sensitive to the input parameter (Hamby, 1994b). A large RDR can be due to either very small input CV or large output CV. Determination of sensitivity of the parameter based on the above broad statement can be subjective. Therefore, Hamby (1994b) suggested the following bench mark for RDR to quantitatively determine the sensitivity of parameters:

RDR > 1 input parameter was considered to be highly sensitive to the output, RDR < 1 input parameter causes little output variability, showing this parameter to be less sensitive,

RDR =1 input CV equals output CV, meaning that all input variability has passed through the model to produce equal output variability.

The above RDR criteria was used in this study in assessing the input parameter sensitivity or output responses.

5.6 Analysis and Results

Four output responses were examined in this parameter uncertainty/sensitivity analysis and they are:

- TKN
- TN
- TP
- DO

Reasons for selecting these four water quality constituents were discussed in Section 3.7. These water quality constituents were also used in model calibration discussed in Chapter 6. The results obtained from this chapter were used in Chapter 6, as prior knowledge so that model calibration can be done efficiently.

To conduct MCS, an appropriate PDF is required. As discussed in Section 5.4.3, QUAL2E allows modeller to select either the normal or the lognormal PDF for MCS. Therefore, the selection of PDF was considered first.

5.6.1 Normal and lognormal distributions

Melching and Anmangandla (1993), Lei and Schilling (1994) and Haan *et al.* (1998) studied the significance of assigning different PDFs for input parameters in their MCS studies. Melching and Anmangandla (1993) studied normal, lognormal, gamma and uniform PDFs for CBOD point loads in their water quality model. They found that these four PDFs provided very similar results for the output probability distribution of DO concentration. Six different PDFs (i.e. uniform, normal, lognormal, triangular, gamma and gumbel) were used to model input parameters of the urban rainfall-runoff model, HYSTEM-EXTRAN (Fuchs and Harms, 1986) by Lei and Schilling (1994). They found that in general, no significant differences between the 6 PDFs (especially between normal and lognormal PDFs) were observed on output responses of volume, peak flow and time to peak relative to four input parameters. The four input parameters considered were the percentage of the area which contributes to runoff at the beginning of the storm event, percentage of the area which contributes to runoff at the end of the storm event, wetting loss and depression storage. Haan *et al.* (1998) compared normal, lognormal, uniform and triangular PDFs for sampling input model parameters (i.e. retention parameter, Universal Soil Loss parameters, soil nitrogen, nitrogen extraction coefficient for runoff, and nitrogen extraction coefficient for leaching) in the non-point pollution source model, AGNPS (Young *et al.*, 1980), and found that the use of PDFs in MCS is less important than the actual mean and variance of the input parameters.

One parameter group at a time was sampled using normal and lognormal PDFs of input parameters using the mean values and the base CV_{in} of Table 5.3. MCS was done with these sampled parameters within QUAL2E. This exercise was done for both flow events. Only the output responses of TN, TP and DO were considered in this part of this study. Note that TKN was not considered as an output response in this case, because TKN is included in TN and also to reduce the number of similar plots without compromising the discussion. Although the analysis was conducted for two flow events and the output responses were analysed for 4 reaches, the results are shown only for one flow event and one river reach. Figures 5.3-5.5 show the cumulative frequency distributions (CFD) plots of TN, TP and DO for the 'lower range' flow event at Reach 15. These plots showed that the CFD of 3 output responses (i.e. TN, TP and DO) produced from the two PDFs were not significantly different for each of the six parameter groups. Plots for other reaches with both flow conditions produced similar

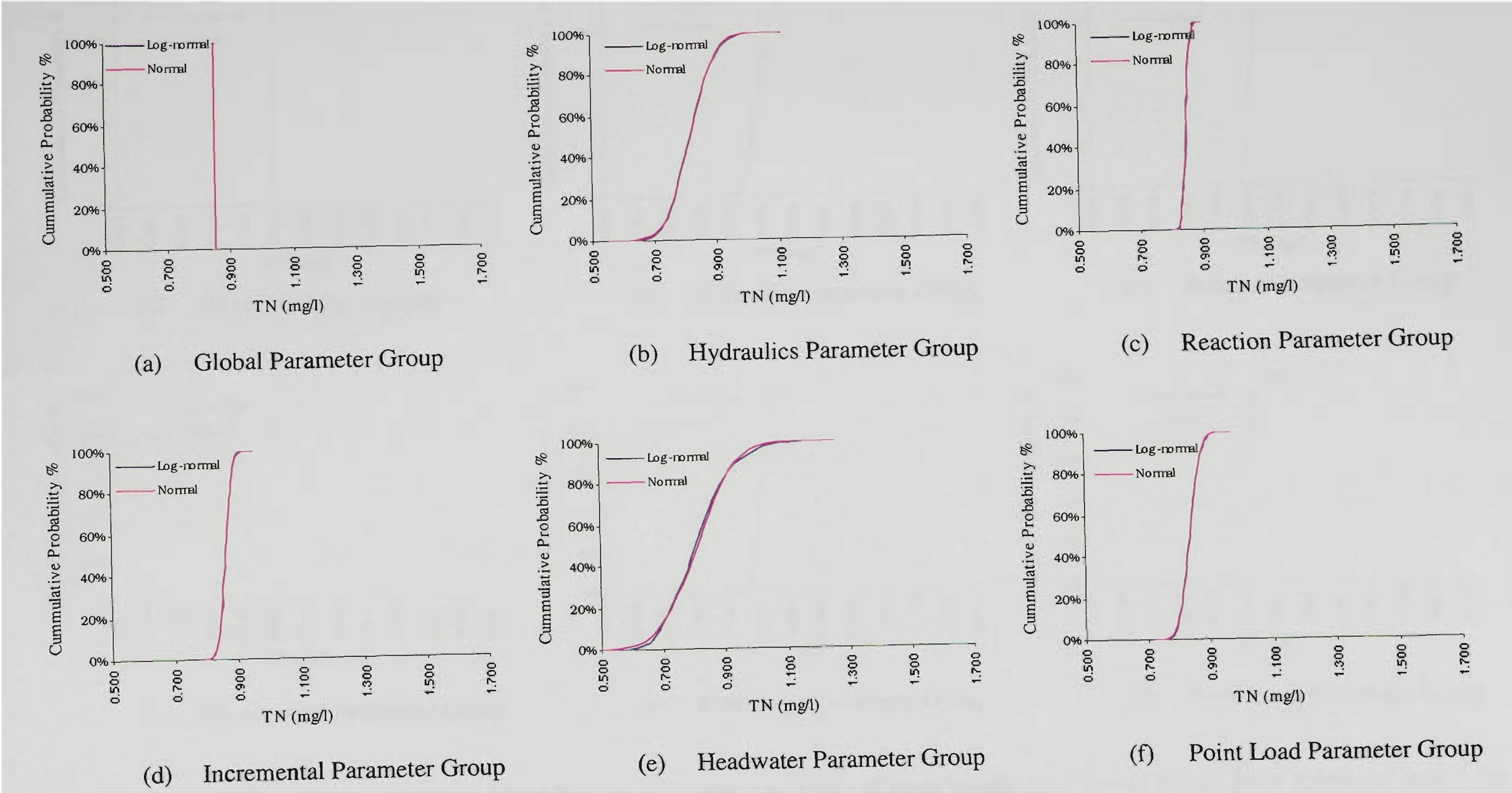


Figure 5.3 CDF for TN Concentration Using Normal and Lognormal PDFs of Input Parameters ('Lower Range' Flow Event and Reach 15)

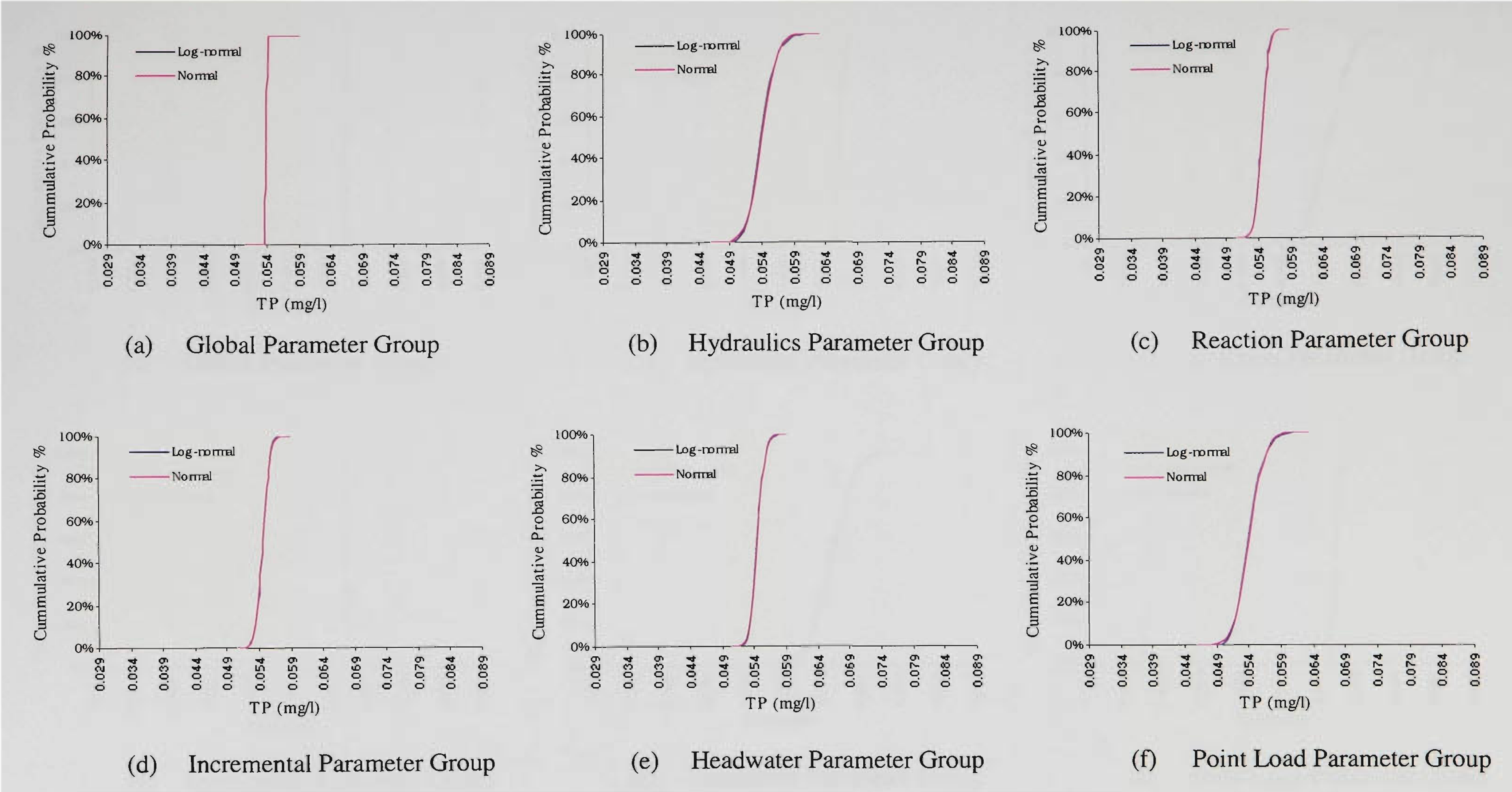


Figure 5.4 CDF for TP Concentration Using Normal and Lognormal PDFs of Input Parameters ('Lower Range' Flow Event and Reach 15)

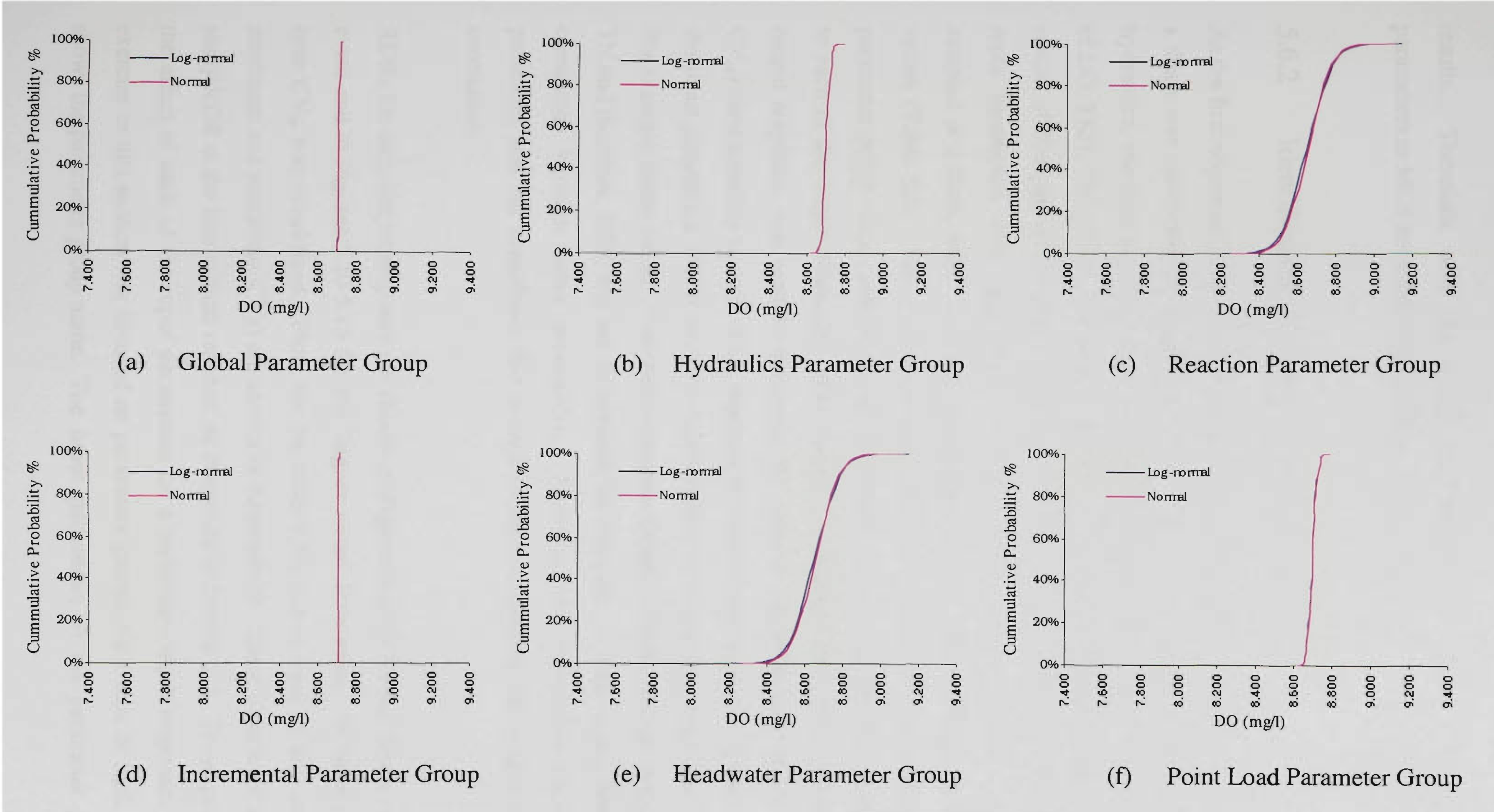


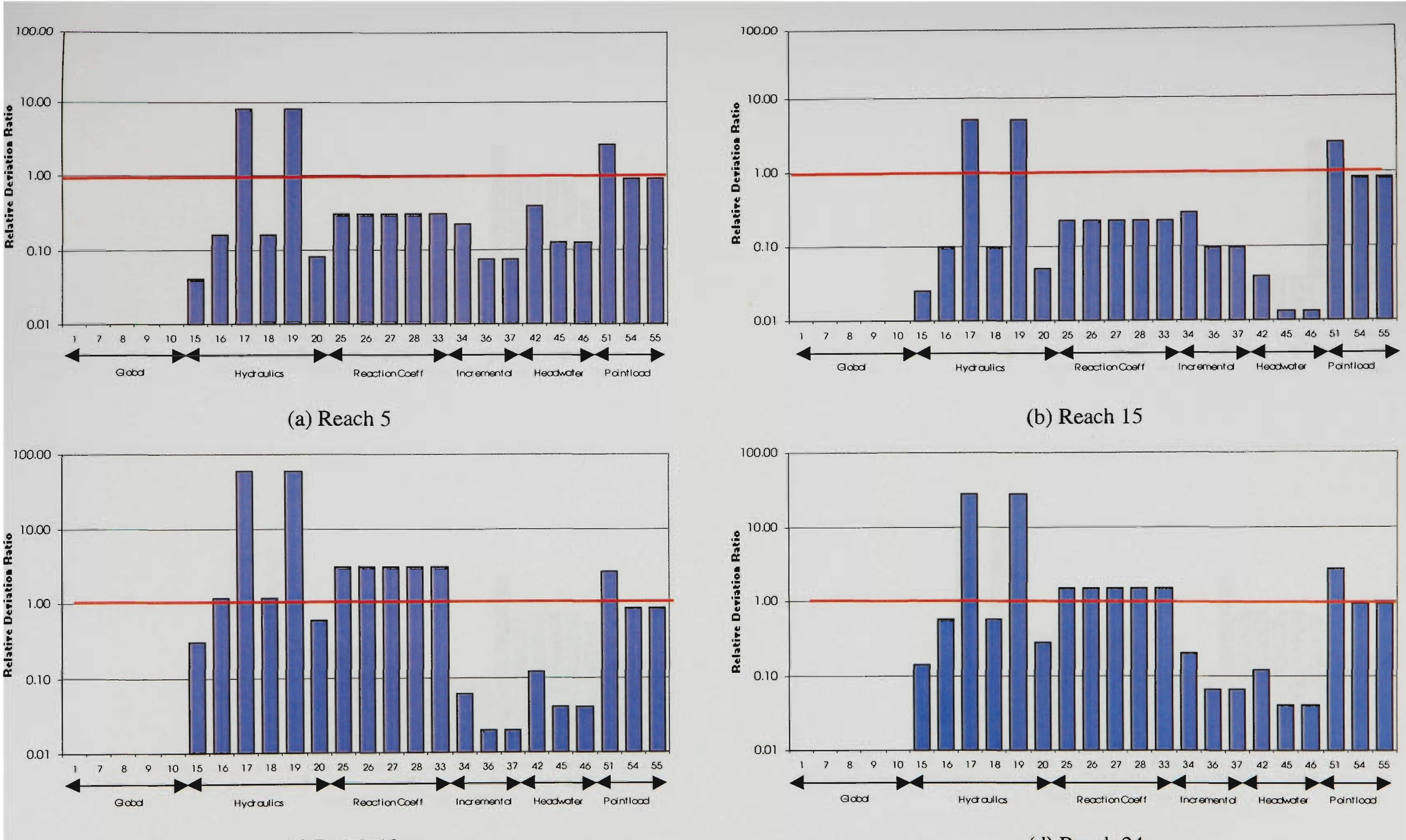
Figure 5.5 CDF for DO Concentration Using Normal and Lognormal PDFs of Input Parameters ('Lower Range' Flow Event and Reach 15)

results. Therefore, only the normal distribution was used for sampling model parameters in MCS for subsequent studies described in Section 5.6.2 to 5.6.4.

5.6.2 Identification of sensitive parameter groups

As the first objective of the pre-calibration uncertainty/sensitivity analysis (Section 5.2), a study was conducted to identify the sensitivity of the parameter groups (i.e. global, hydraulics, reaction, incremental, headwater and point load groups) on output responses of DO, TKN, TN and TP. To study whether CV_{in} values of input parameters affects the results of this part of the study, 3 sets of CV_{in} (i.e. base, minimum and maximum) of input parameters were considered. All parameters in one parameter group were sampled at a time, while the input parameters of other groups remain at their mean values (Table 5.2). This study was done for each of the two flow events and each parameter group. This method can identify which parameter group is the most sensitive to each of the output responses. The output CV obtained from MCS for each of the output responses was used to determine the relative deviation ratio (RDR = CV_{out} / CV_{in}) corresponding to each of the input parameters of each parameter group. Note that the input parameters which have no contributions to output responses were neglected. For example, there was no relationship between Organic-P decay rate to output response TN and therefore, RDR was not determined for this case. In some cases, there can be a correlation between input parameters. However, the correlation between input parameter was not considered due to lack of data to estimate the magnitude of their correlation.

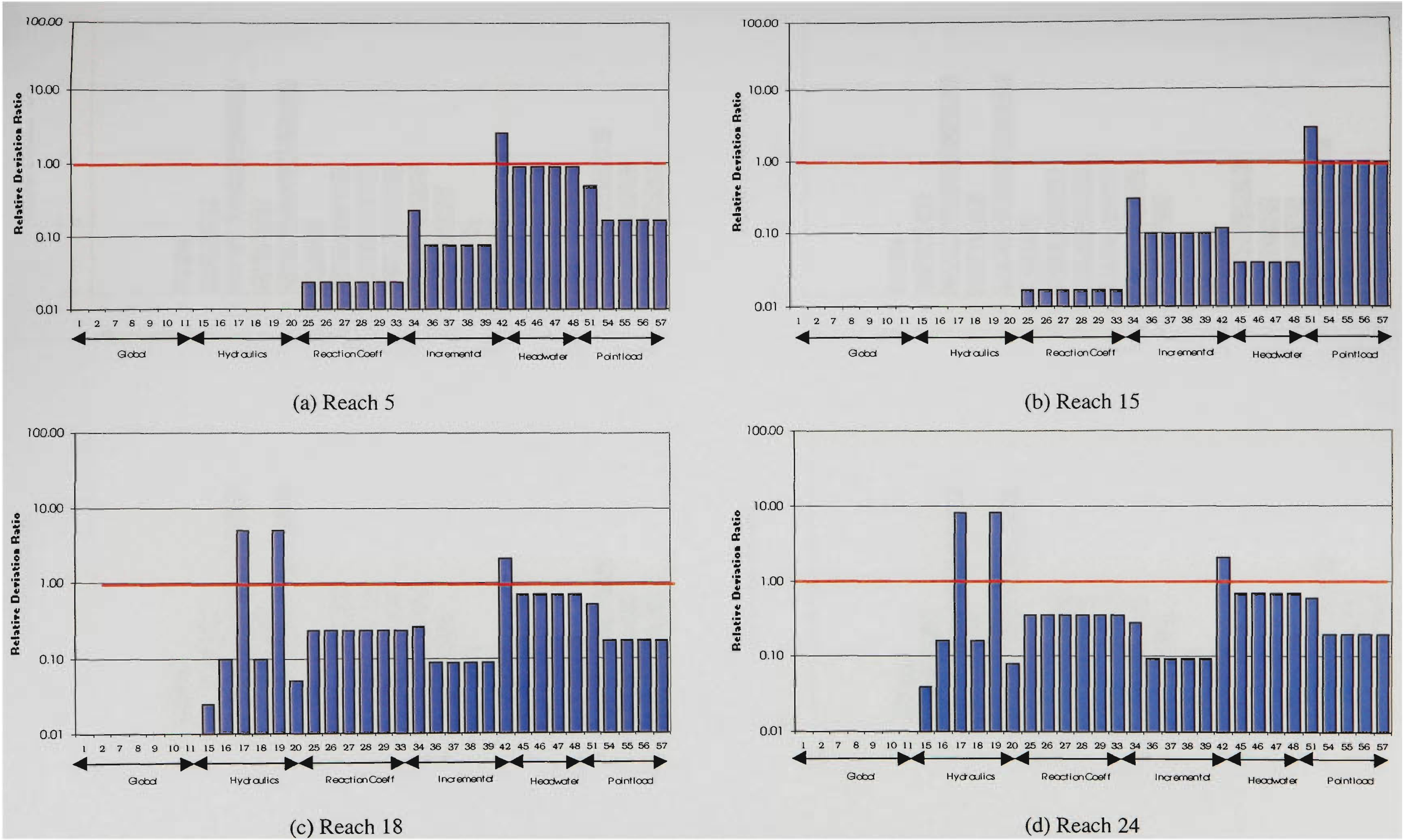
RDRs for each output response are shown in Figures 5.6-5.9 for the 'lower range' flow event and in Figures 5.10-5.13 for the 'higher range' flow event. In these figures, the base CV_{in} was considered. Plots for the other CV_{in} values used in this analysis (i.e. minimum and maximum CVs) are shown in Appendix B. These plots were analysed to study RDR at the four critical reaches, as discussed in Section 5.3. These plots indicate the effect of each of the input parameters on a particular output response. Since the exercise in this section was focused on parameter groups, the x-axis of each graph also shows the parameter group name. The input parameters of the parameter groups are



(c) Reach 18

(d) Reach 24

Figure 5.6 Relative Deviation Ratio of TKN for 'Lower Range' Flow Event (Base CV)



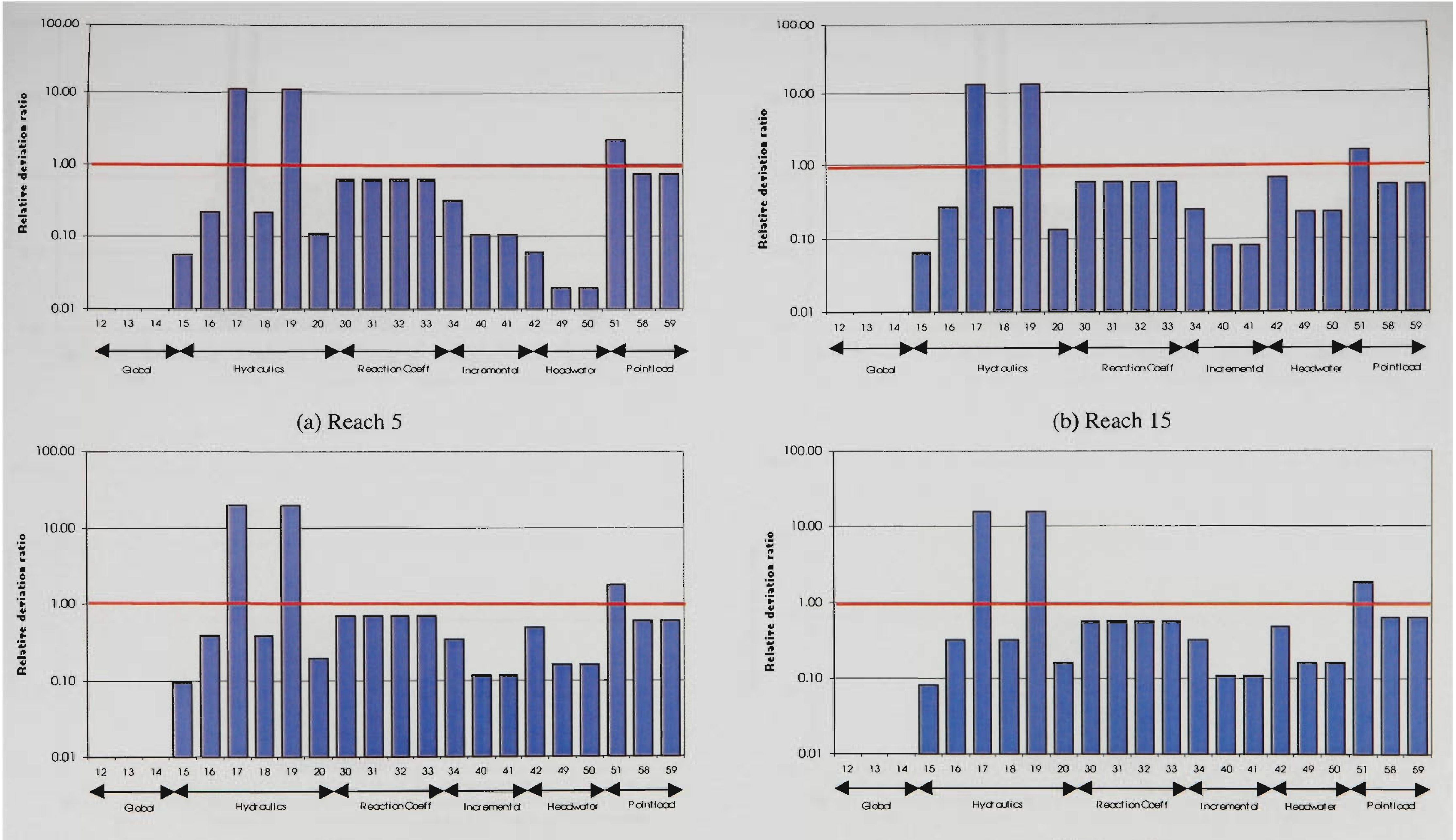
(a) Reach 5

(b) Reach 15

(c) Reach 18

(d) Reach 24

Figure 5.7 Relative Deviation Ratio of TN for 'Lower Range' Flow Event (Base CV)



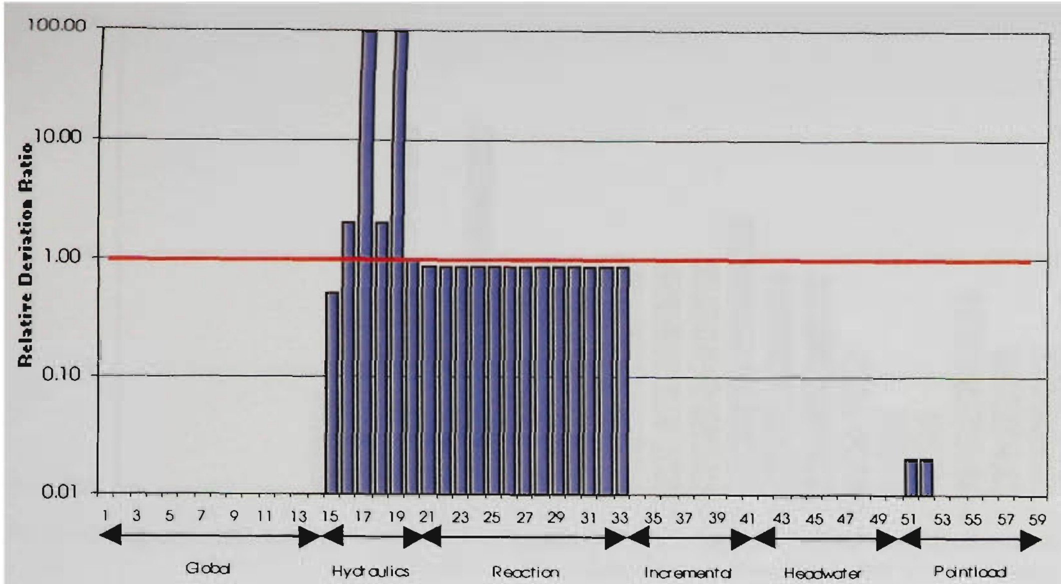
(a) Reach 5

(b) Reach 15

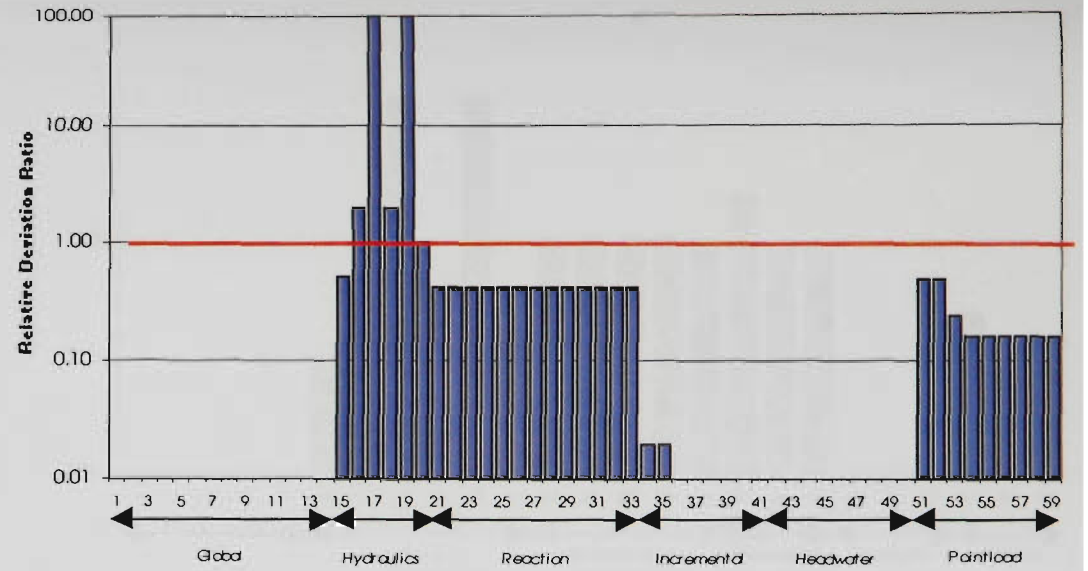
(c) Reach 18

(d) Reach 24

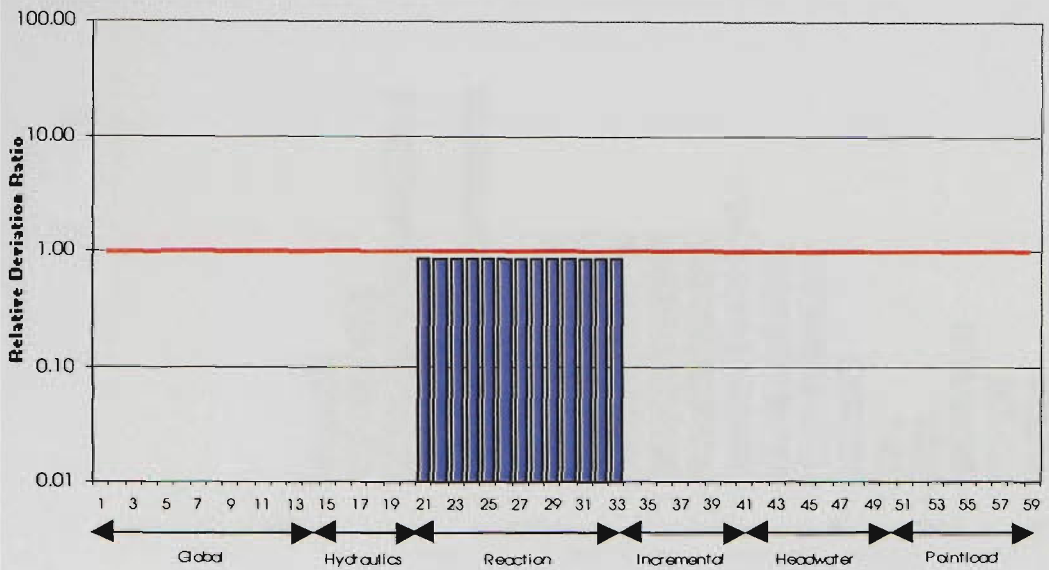
Figure 5.8 Relative Deviation Ratio of TP for 'Lower Range' Flow Event (Base CV)



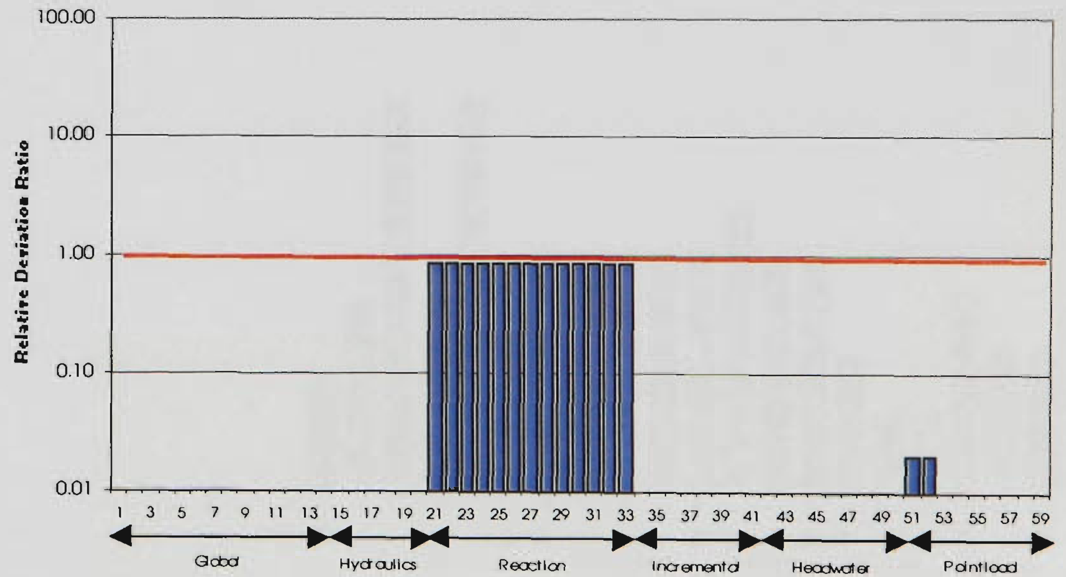
(a) Reach 5



(b) Reach 15

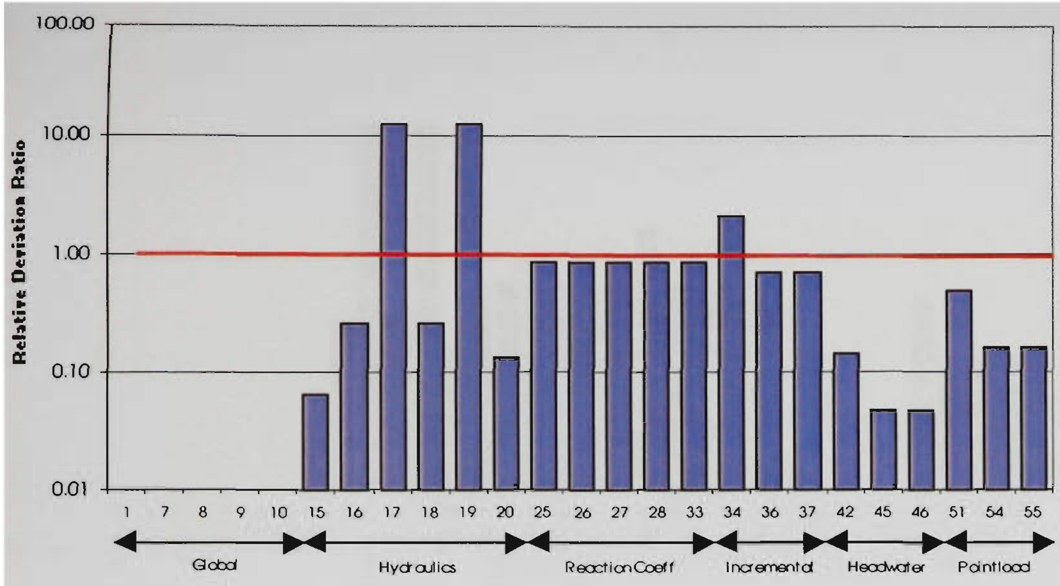


(c) Reach 18

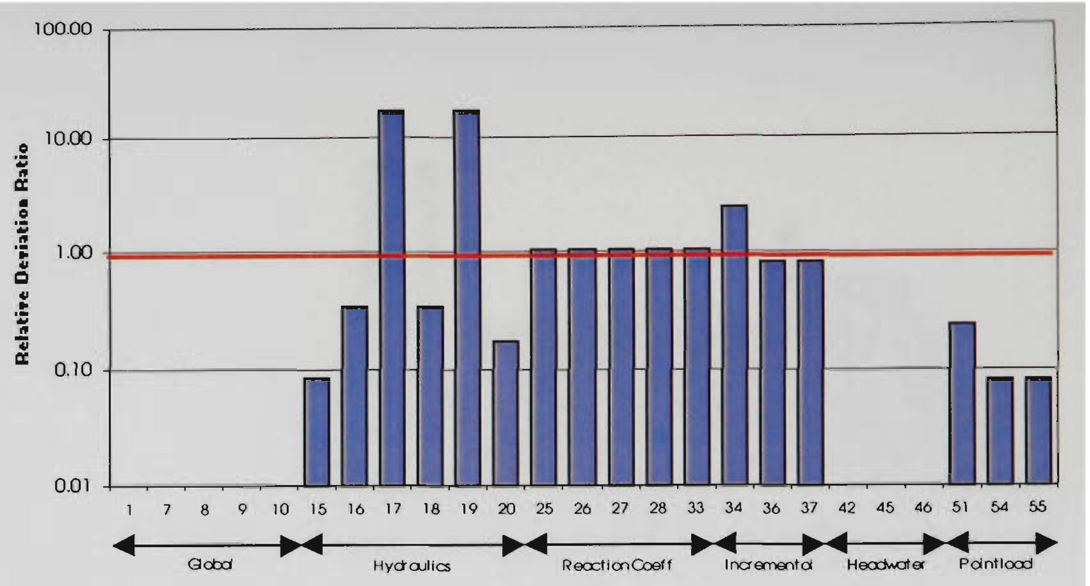


(d) Reach 24

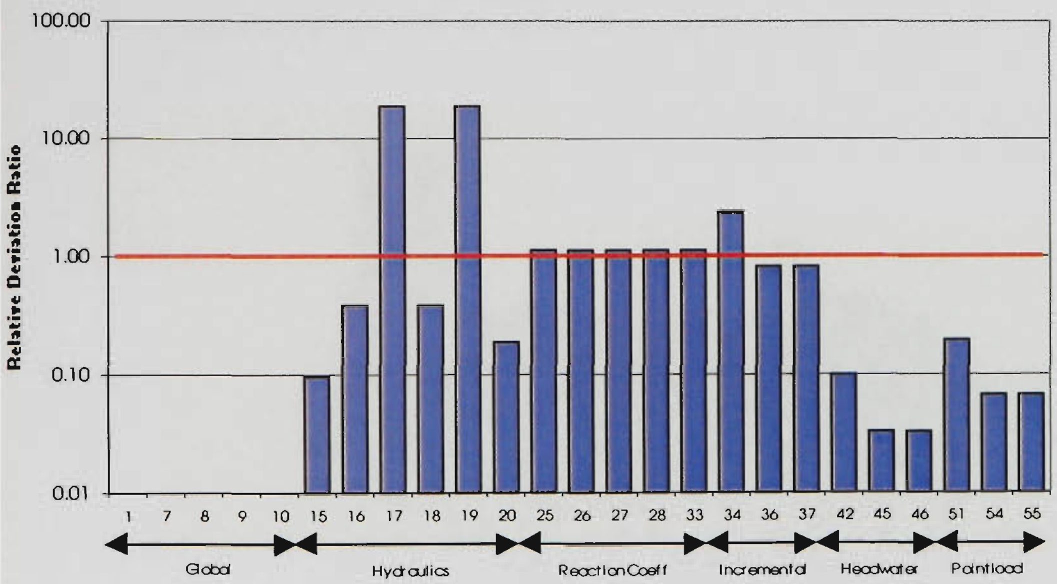
Figure 5.9 Relative Deviation Ratio of DO for 'Lower Range' Flow Event (Base CV)



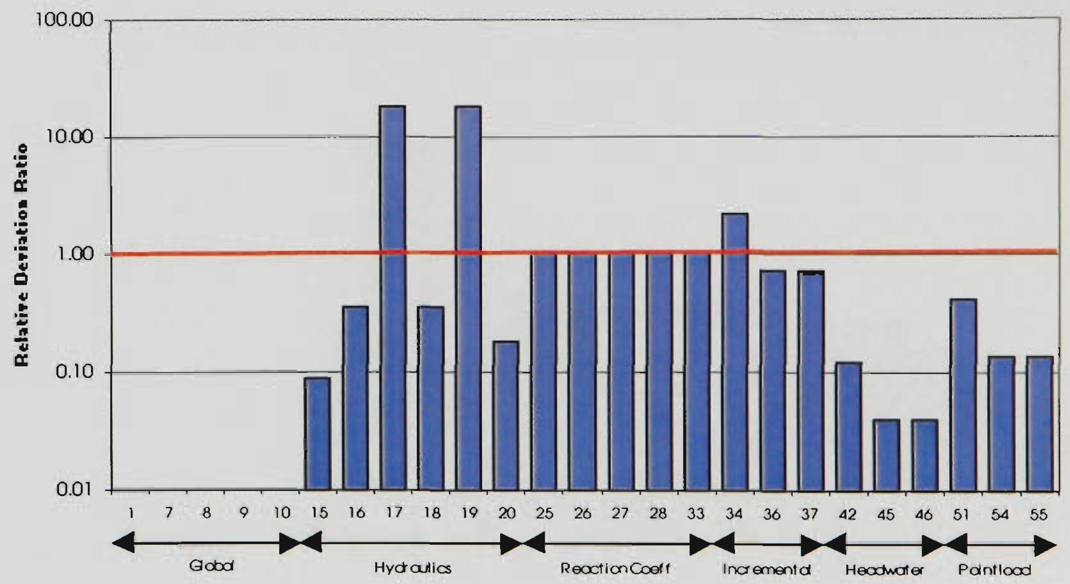
(a) Reach 5



(b) Reach 15

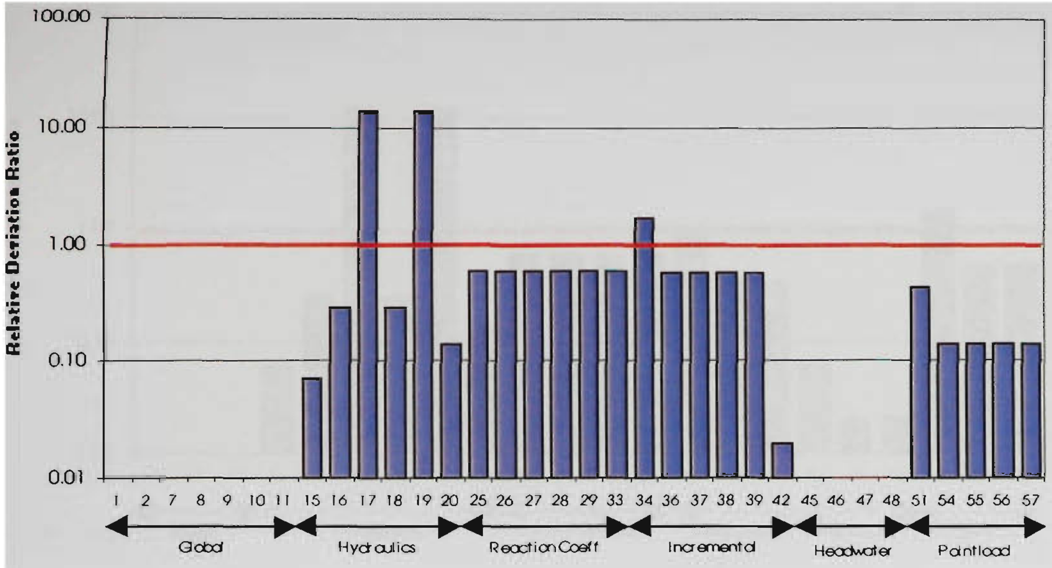


(c) Reach 18

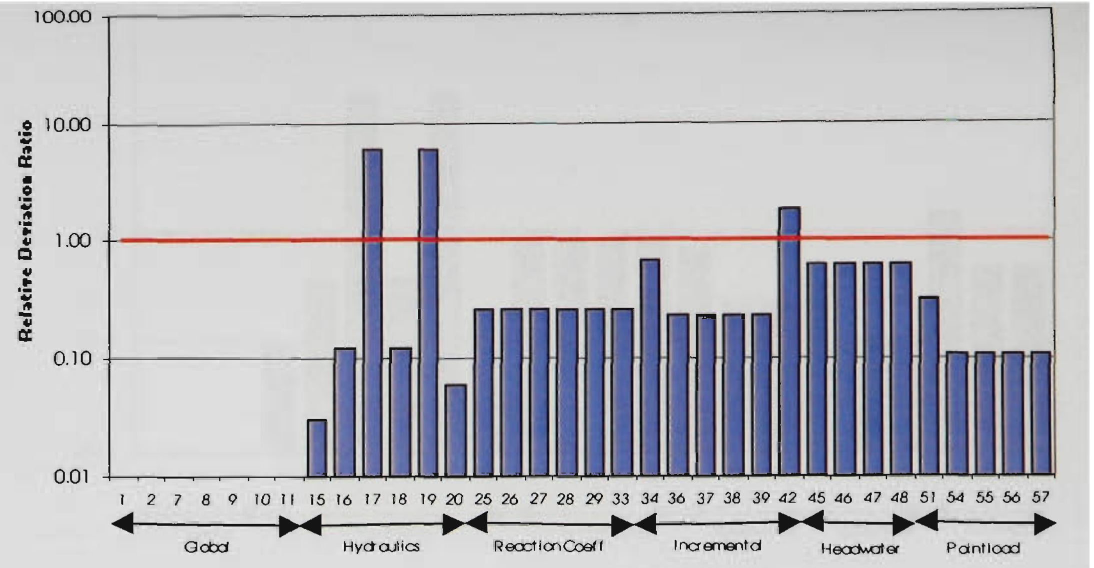


(d) Reach 24

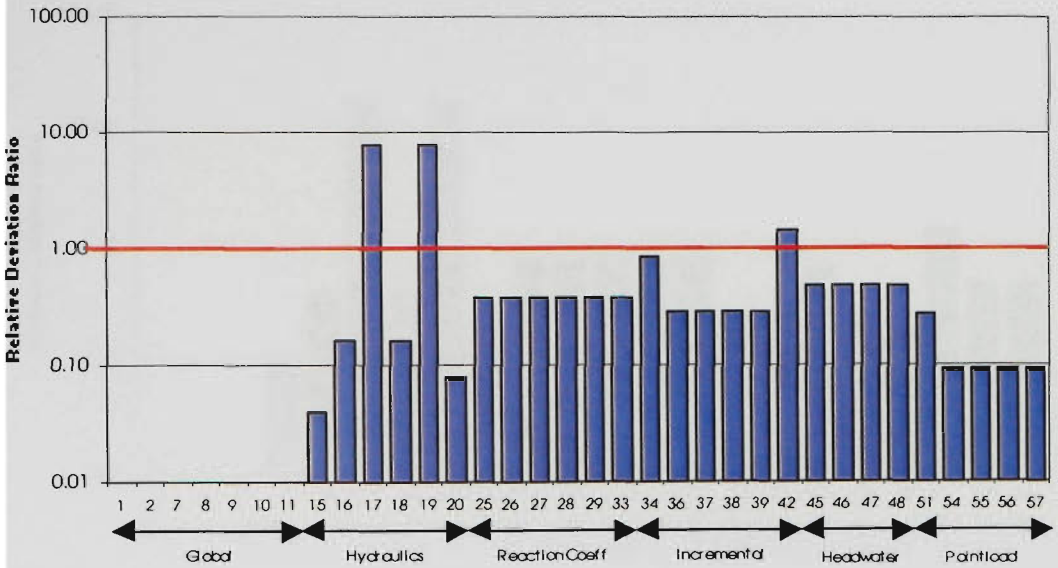
Figure 5.10 Relative Deviation Ratio of TKN for 'Higher Range' Flow Event (Base CV)



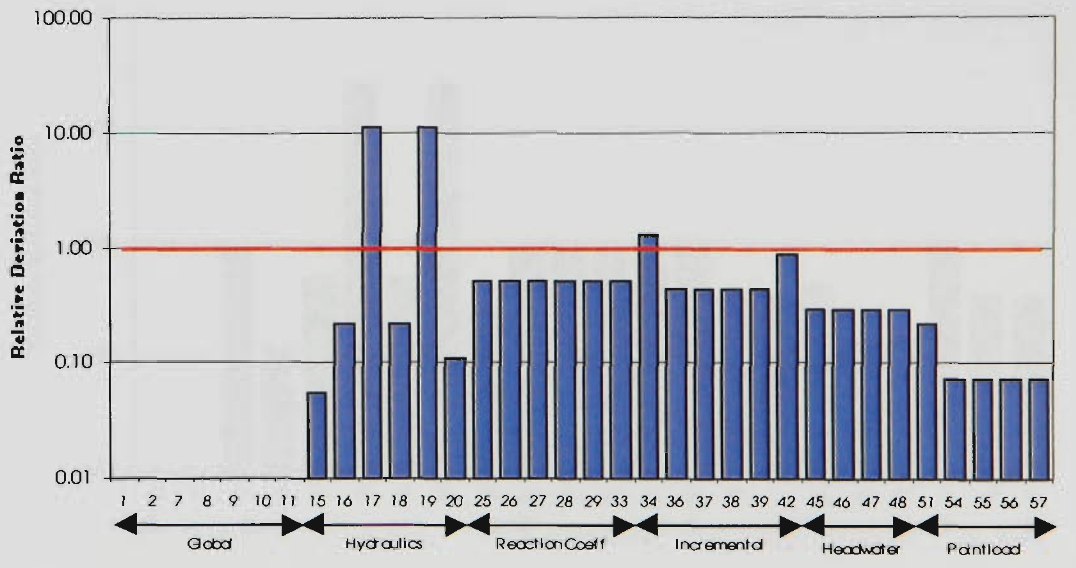
(a) Reach 5



(b) Reach 15

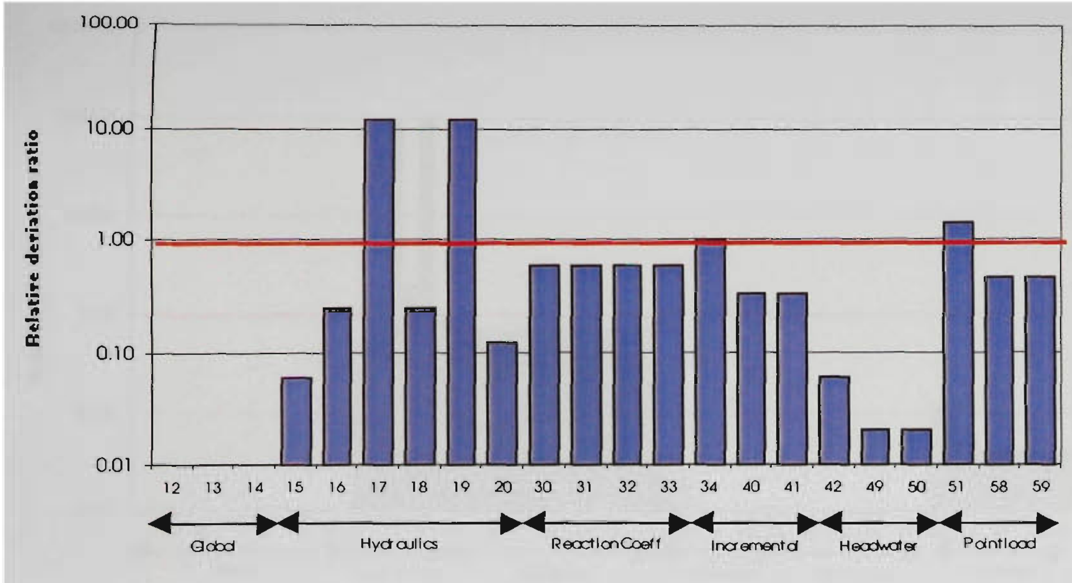


(c) Reach 18

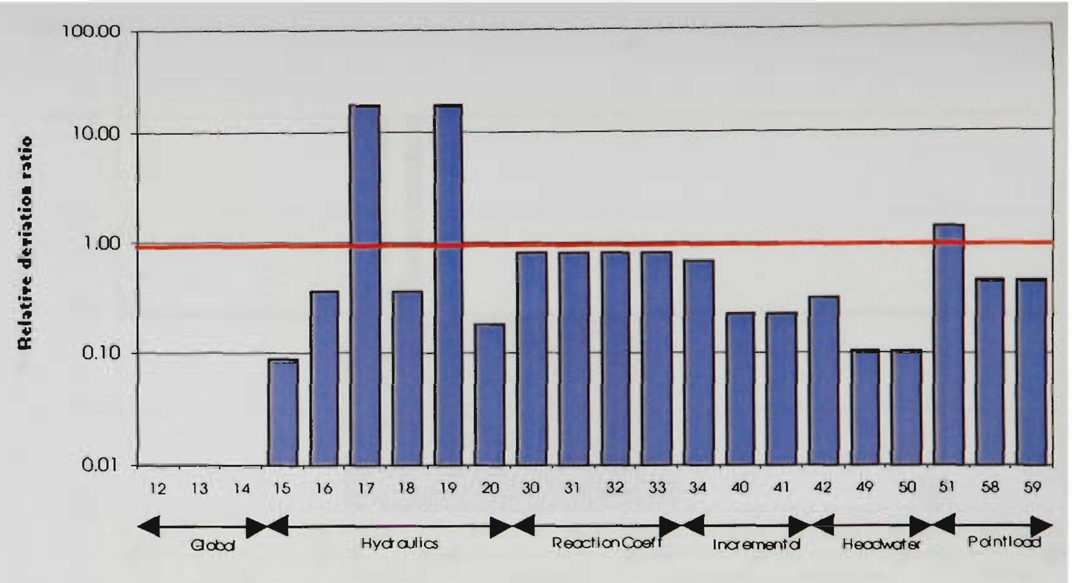


(d) Reach 24

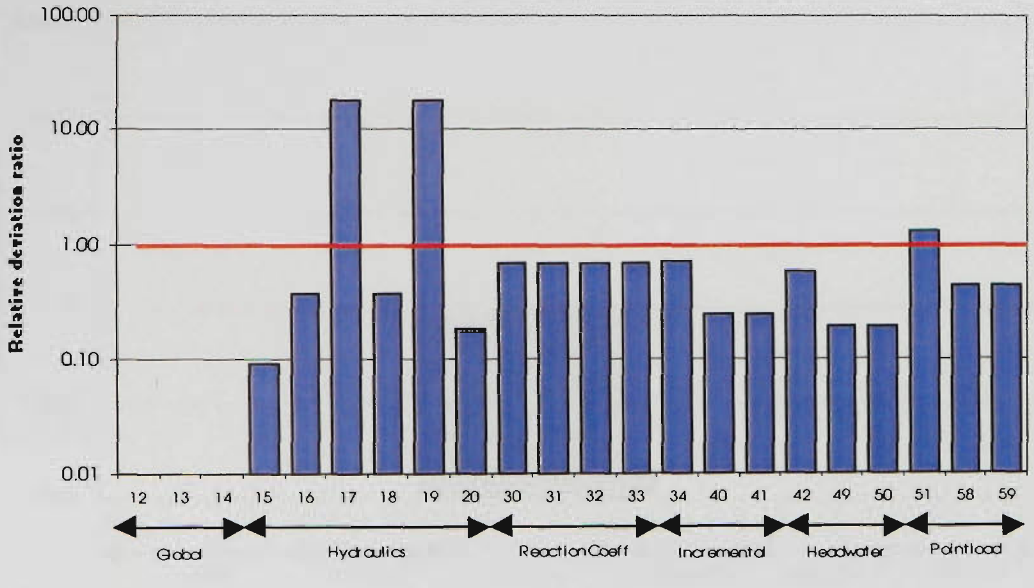
Figure 5.11 Relative Deviation Ratio of TN for 'Higher Range' Flow Event (Base CV)



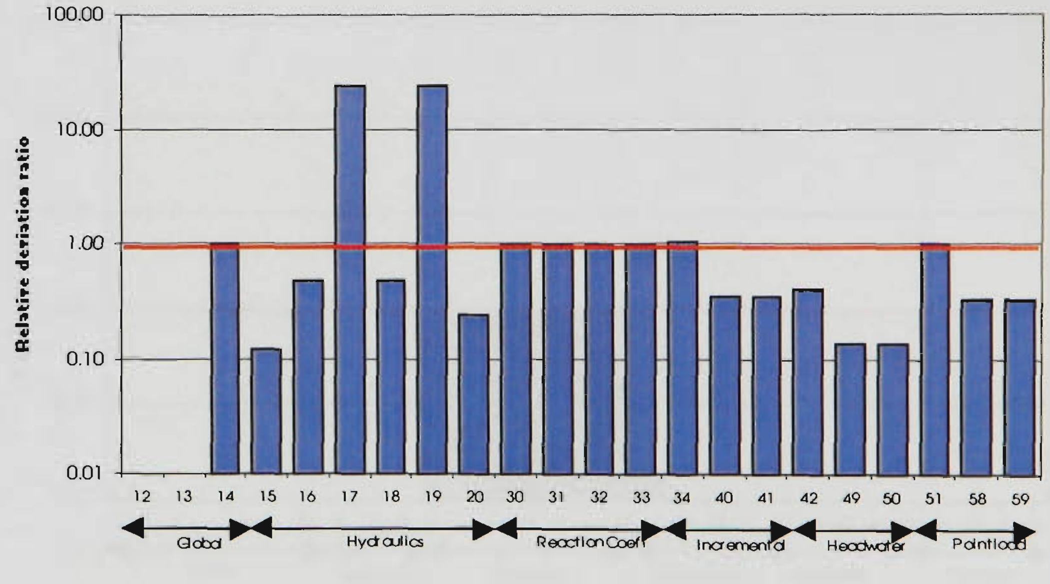
(a) Reach 5



(b) Reach 15

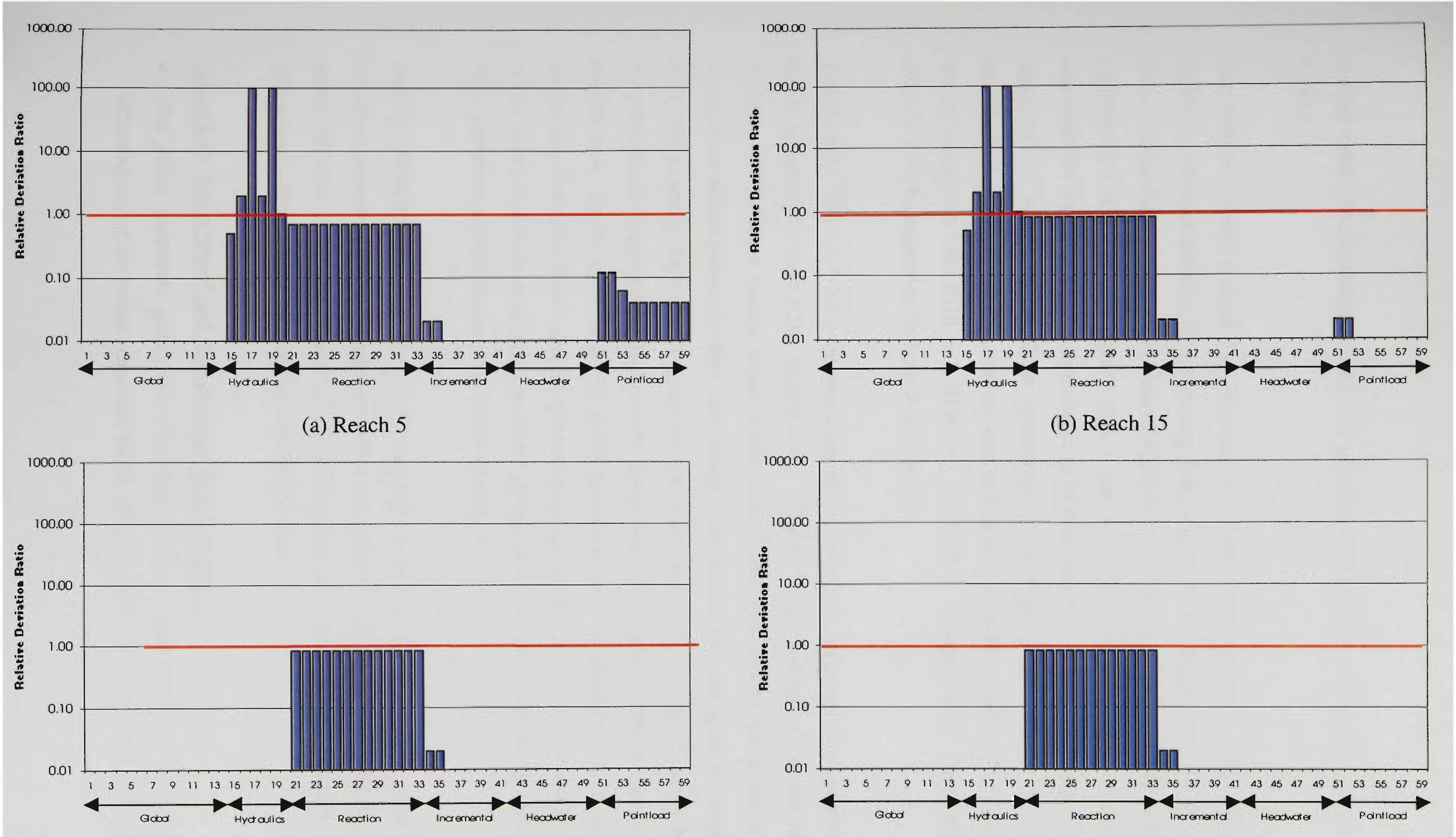


(c) Reach 18



(d) Reach 24

Figure 5.12 Relative Deviation Ratio of TP for 'Higher Range' Flow Event (Base CV)



(a) Reach 5

(b) Reach 15

(c) Reach 18

(d) Reach 24

Figure 5.13 Relative Deviation Ratio of DO for 'Higher Range' Flow Event (Base CV) number-coded for convenience, with their parameter names listed in the fifth column of Table 5.3.

The Hamby (1994b) method (as discussed in Section 5.5) was used to identify the sensitivity of the parameters, by comparing RDR with 1. From Figures 5.6-5.13, the red line indicates the sensitive cut-off line of RDR=1. When one or more parameter within a group has a RDR value of greater than 1, that whole group was treated as sensitive, otherwise the whole group was considered to be less sensitive. Tables 5.4-5.6 shows a summary of sensitivity of these parameters according to the Hamby (1994b) classification for different CV_{in} values used in this study. In these tables, G, E and L denote parameters with RDR > 1, RDR = 1 and RDR < 1 respectively. These tables also show RDR summaries for both flow events.

Results from Tables 5.4-5.6 were then analysed to identify the most sensitive parameter group according to their RDR classification. The parameter group having the most number of 'G's for a certain water quality constituent was interpreted as the most sensitive parameter group (for that water quality constituent) and was given the highest rank. For example, the hydraulic group has the most number of 'G's for DO under both flow conditions, which means the hydraulic group effect DO significantly, and given the highest rank. On the other hand, the global group has the most number of 'L's, which means that it was the least sensitive parameter group to DO and was given the lowest rank. These sensitivity ranking of parameter groups are shown in Tables 5.7-5.9 for the base, minimum and maximum CV_{in} values respectively.

Hamby (1994b) stated that the actual ranking of sensitivity does not deduce the sensitive parameter, but is the parameter that consistently appears in the ranking under many different conditions should be called the sensitive parameter. In general, the following findings were observed from Tables 5.7-5.9.

Under both 'lower range' and 'higher range' flow conditions

- The global parameter group was identified as an insensitive parameter group in affecting any of the output responses under all input CV_{in} values. The hydraulics

Table 5.4 Sensitivity of Parameter Groups for Base CV

	'Lower Range' Flow Event				'Higher Range' Flow Event			
Reach	R5	R15	R18	R24	R5	R15	R18	R24
TKN								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	G	G	G	L	G	G
Incremental	L	L	L	L	G	G	G	G
Headwater	L	L	L	L	L	L	L	L
Point load	G	G	G	G	L	L	L	L
TN								
Global	L	L	L	L	L	L	L	L
Hydraulics	L	L	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	G	L	L	L
Headwater	L	G	G	G	L	G	G	L
Point load	G	L	L	L	L	L	L	L
TP								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	E
Incremental	L	L	L	L	E	L	L	E
Headwater	L	L	L	L	L	L	L	L
Point load	G	G	G	G	G	G	G	E
DO								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	L	L	G	G	L	L
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	L	L	L	L
Headwater	L	L	L	L	L	L	L	L
Point load	L	L	L	L	L	L	L	L

Table 5.5 Sensitivity of Parameter Groups for Minimum CV

	'Lower Range' Flow Event				'Higher Range' Flow Event			
Reach	R5	R15	R18	R24	R5	R15	R18	R24
TKN								
Global	L	L	L	L	L	L	L	L
Hydraulics	E	G	G	G	G	G	G	G
Reaction	L	L	G	G	G	G	G	L
Incremental	L	L	L	L	G	G	G	G
Headwater	L	L	L	L	L	L	L	L
Point load	G	G	G	G	L	L	L	L
TN								
Global	L	L	L	L	L	L	L	L
Hydraulics	L	L	E	G	G	E	G	G
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	G	G	G	G
Headwater	L	G	G	G	L	L	L	L
Point load	G	G	G	G	G	E	L	L
TP								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	E
Incremental	L	L	L	L	G	G	G	G
Headwater	L	G	G	G	L	G	G	G
Point load	G	G	G	G	G	G	G	G
DO								
Global	L	L	L	L	L	L	L	L
Hydraulics	L	L	L	L	L	L	L	L
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	L	L	L	L
Headwater	L	L	L	L	L	L	L	L
Point load	E	L	L	L	L	L	L	L

Table 5.6 Sensitivity of Parameter Groups for Maximum CV

	'Lower Range' Flow Event				'Higher Range' Flow Event			
Reach	**R5**	**R15**	**R18**	**R24**	**R5**	**R15**	**R18**	**R24**
TKN								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	G	G	G	G	G	G
Incremental	L	L	L	L	G	G	G	G
Headwater	L	L	L	L	L	L	L	L
Point load	G	G	G	G	L	L	L	L
TN								
Global	L	L	L	L	L	L	L	L
Hydraulics	L	L	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	G	G	L	G
Headwater	L	G	G	G	L	G	G	L
Point load	G	L	L	L	L	L	L	L
TP								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	E
Incremental	L	L	L	L	E	L	L	E
Headwater	L	L	L	L	L	L	L	L
Point load	G	G	G	G	G	G	G	E
DO								
Global	L	L	L	L	L	L	L	L
Hydraulics	G	G	G	G	G	G	G	G
Reaction	L	L	L	L	L	L	L	L
Incremental	L	L	L	L	L	L	L	L
Headwater	L	L	L	L	L	L	L	L
Point load	L	L	L	L	L	L	L	L

Table 5.7 Sensitivity Ranking of Parameter Groups for Base CV

	Sensitivity Rank	'Lower Range' Flow Group	'Higher Range' Flow Group
TKN	1	Hydraulics	Hydraulics
	2	Point Loads	Incremental
	3	Reaction	Reaction
TN	1	Headwater	Hydraulics
	2	Hydraulics	Headwater
	3	Point Loads	Incremental
TP	1	Hydraulics	Hydraulics
	2	Point Loads	Point Loads
	3		Incremental
	4		Reaction
DO	1	Hydraulics	Hydraulics

Table 5.8 Sensitivity Ranking of Parameter Groups for Minimum CV

	Sensitivity Rank	'Lower Range' Flow Group	'Higher Range' Flow Group
TKN	1	Point Loads	Hydraulics
	2	Hydraulics	Incremental
	3	Reaction	Reaction
TN	1	Point Loads	Incremental
	2	Headwater	Hydraulics
	3	Hydraulics	Point Loads
TP	1	Hydraulics	Hydraulics
	2	Point Loads	Incremental
	3	Headwater	Point Loads
	4		Headwater
DO	1	Point loads	

Table 5.9 Sensitivity Ranking of Parameter Groups for Maximum CV

	Sensitivity Rank	'Lower Range' Flow Group	'Higher Range' Flow Group
TKN	1	Hydraulics	Hydraulics
	2	Point Loads	Incremental
	3	Reaction	Reaction
TN	1	Headwater	Hydraulics
	2	Hydraulics	Headwater
	3	Point Loads	Incremental
TP	1	Hydraulics	Hydraulics
	2	Point Loads	Point Loads
	3		Incremental
	4		Reactions
DO	1	Hydraulics	Hydraulics

parameter group plays the most significant role in affecting TKN, TN and TP under all input CV_{in} values. This implies that the hydraulic power functions should be estimated with high accuracy to avoid any errors in predicting water quality in the river. This can be done by having river cross-sections surveyed and stream gauging at a representative point of each reach. The reaction parameter group was sensitive to TKN and TP under all input CV_{in} values.

- The headwater parameter group was also identified as a sensitive parameter group to both TN and TP, but as input CV_{in} reduces, the sensitivity also reduces.

<u>Under 'lower range' flow conditions</u>

- In addition to the hydraulic parameter group, the point loads parameter group also plays a significant role in affecting TKN, TN and TP under all CV_{in} values. The point load parameter group was found as a sensitive group because the point loads were the major pollutant source entering the river under 'lower range' flow conditions.

Under 'higher range' flow conditions

- In addition to the hydraulic parameter group, the incremental parameter group plays a significant role in affecting TKN, TN and TP under all CV_{in} values. Incremental parameter group was found as sensitive because the incremental flows and concentrations were the major pollutant source under 'higher range' flow conditions.

After giving general comments about the sensitivity of parameter group with respect to flow conditions, further discussions relative to each water quality constituents are discussed below.

Total Kjeldahl Nitrogen - TKN

The hydraulics parameter group was found to be the most sensitive parameter group for TKN under both 'lower range' and 'higher range' flow conditions. The major difference between two flow events was that the point load parameter group appeared on the sensitivity rank under 'lower range' flows, whereas the incremental parameter group appeared under 'higher range' flows. This was to be expected since point source is the major pollutant contributor during 'lower range' flows, which significantly affect the water quality condition. On the other hand, during 'higher range' flow conditions, incremental flows and concentrations dominate the role of pollutant source. This indicates the importance of estimation of non-point source flows and concentration when modelling 'higher range' flow conditions in Yarra River. The reaction parameter group was also sensitive under both flow conditions, which meant the estimation of decay rates for both Org-N_d and $NH_{3\text{-}d}$ need to be done with accuracy. These rates effect the concentration levels of TKN, and in turn effect TN due to the cascade process in the nitrogen cycle.

Total Nitrogen - TN

The headwater parameter group was shown to be the most sensitive parameter group, which affects TN under 'lower range' flow conditions. Even under 'higher range' flows, the headwater parameter group seemed to be fairly sensitive. This sensitivity is in addition to the sensitivity of the hydraulic parameter group. As can be seen in Tables 5.4-5.6, the effect of the headwater parameter group was predominant for R15 and R18 under both flow conditions (except for the minimum CV_{in} values where 'higher range' flow event did not show any sensitivity). In addition, reach R24 was also shown to be sensitive to the headwater parameter group under 'lower range' flows.

Total Phosphorus -TP

The sensitivity ranking under both flow conditions was the same for TP. Under both flow conditions, the point source parameter group was shown to be the most significant source, after the hydraulics group. This suggests that data collection for point source flow and concentration needs to be done with extra effort to avoid any errors in modelling. The incremental flow group was also identified as sensitive to TP under 'higher range' flows.

Dissolved Oxygen - DO

As can be seen from Tables 5.7-5.9, DO has the least sensitivity in terms of sensitive input parameter groups compared to the other water quality constituents. Only the hydraulic parameter group was shown to be sensitive for both 'lower range' and 'higher range' flows when CV_{in} of input parameter is high. This implies:

- When the hydraulic parameters are determined accurately (or with low uncertainty), then sensitivity is reduced.
- Flow (determined through hydraulic parameters) could affect the DO concentration in the river due to the rate of oxygen input from the atmosphere via reaeration rate. The reaeration rate increases with flow velocity and vice versa.

Although the reaeration rate (K2) was thought to be as one of the major rates controlling the rate of oxygen input in the river, the result revealed that K2 (in reaction parameter group) was not sensitive to DO in the Yarra River. One possibility for this was that the DO concentration in Yarra River was at a high level for the two flow events considered and the variation in K2 was not affecting DO response significantly because of its (already) high values.

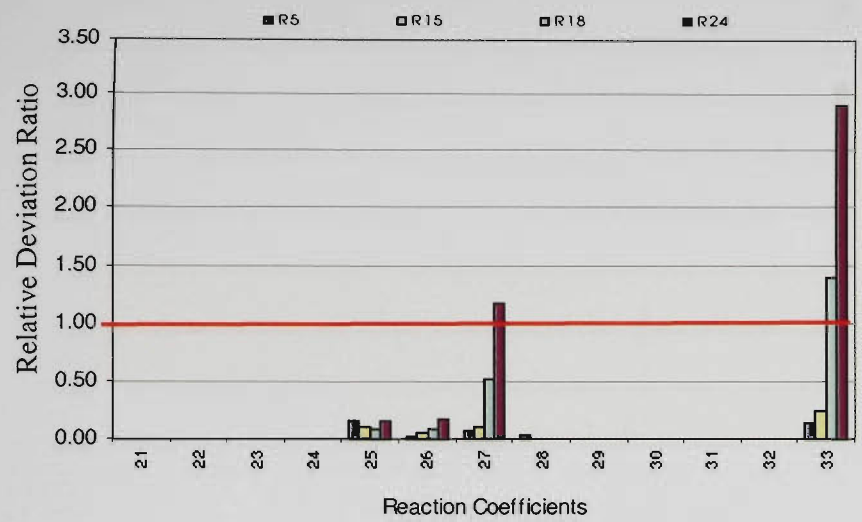
5.6.3 Sensitivity of parameters within reaction group

As stated in Section 5.2, the reaction parameter group was further analysed to identify the most sensitive parameters to the output responses. These sensitive parameters can then be given higher priority in calibrating YRWQM.

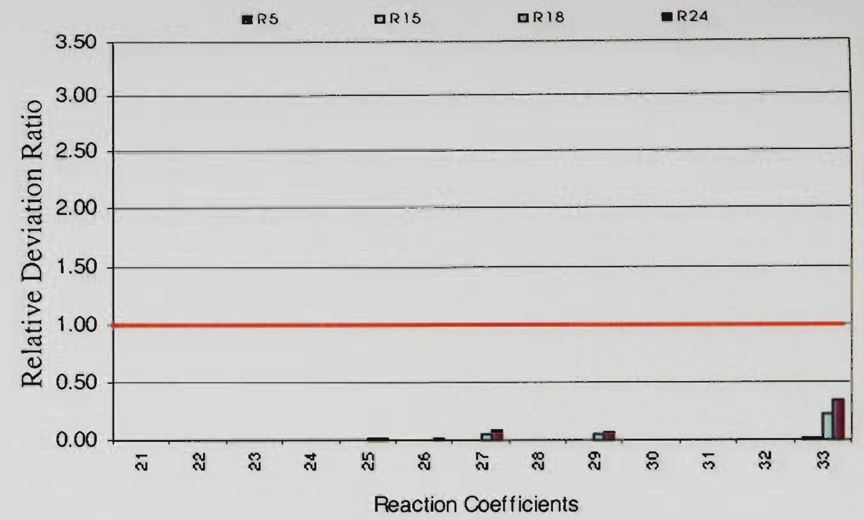
As stated in Section 5.2, the method used for this analysis was to sample each decay parameter (within the reaction parameter group) at one time, while holding other decay rates and the parameters in other groups at their mean values. The two flow events and the three input CV scenarios were considered as in Section 5.6.2. The sensitivity was also studied through RDR. Figures 5.14-5.16 show the plots of RDR for each individual decay parameter for the 'lower range' flow event with respect to three input parameter CV_{in} values. These figures show all 4 study reaches (i.e. R5, R15, R18 and R24). Similar plots are shown for the 'higher range' flow event in Figures 5.17-5.19. The number shown in Figures 5.14 to 5.19 are defined in Table 5.10.

In general, the initial temperature was found to be the most sensitive decay parameter to TKN, TN, TP and DO, regardless of the flow conditions and input parameter CVs. This was expected as all decay rates are affected by temperature. Generally, the decay parameter sensitivity increases towards downstream in most cases and this could possibly be due to the increase in pollution towards the downstream, which increases the decay rates of organic matters.

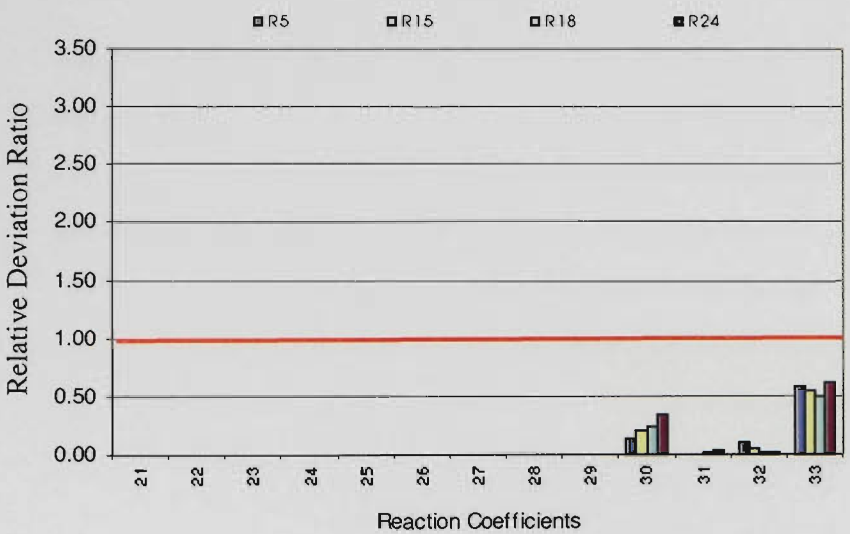
The RDR of Figures 5.14-5.19 were then simultaneously analysed to identify the sensitive decay rates across all 3 input parameter CVs for both flows. The results of this



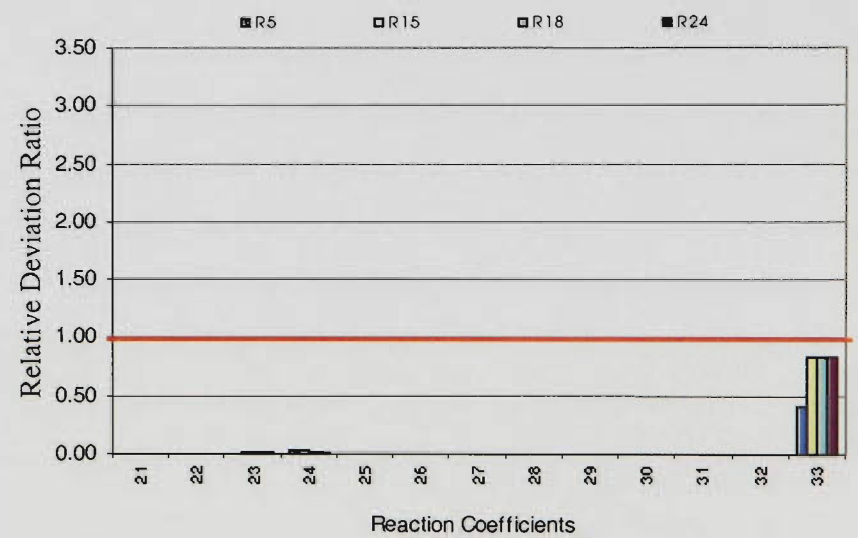
(a) TKN



(b) TN



(c) TP



(d) DO

Figure 5.14 Relative Deviation Ratio for 'Lower Range' Flow Event (Base CV)

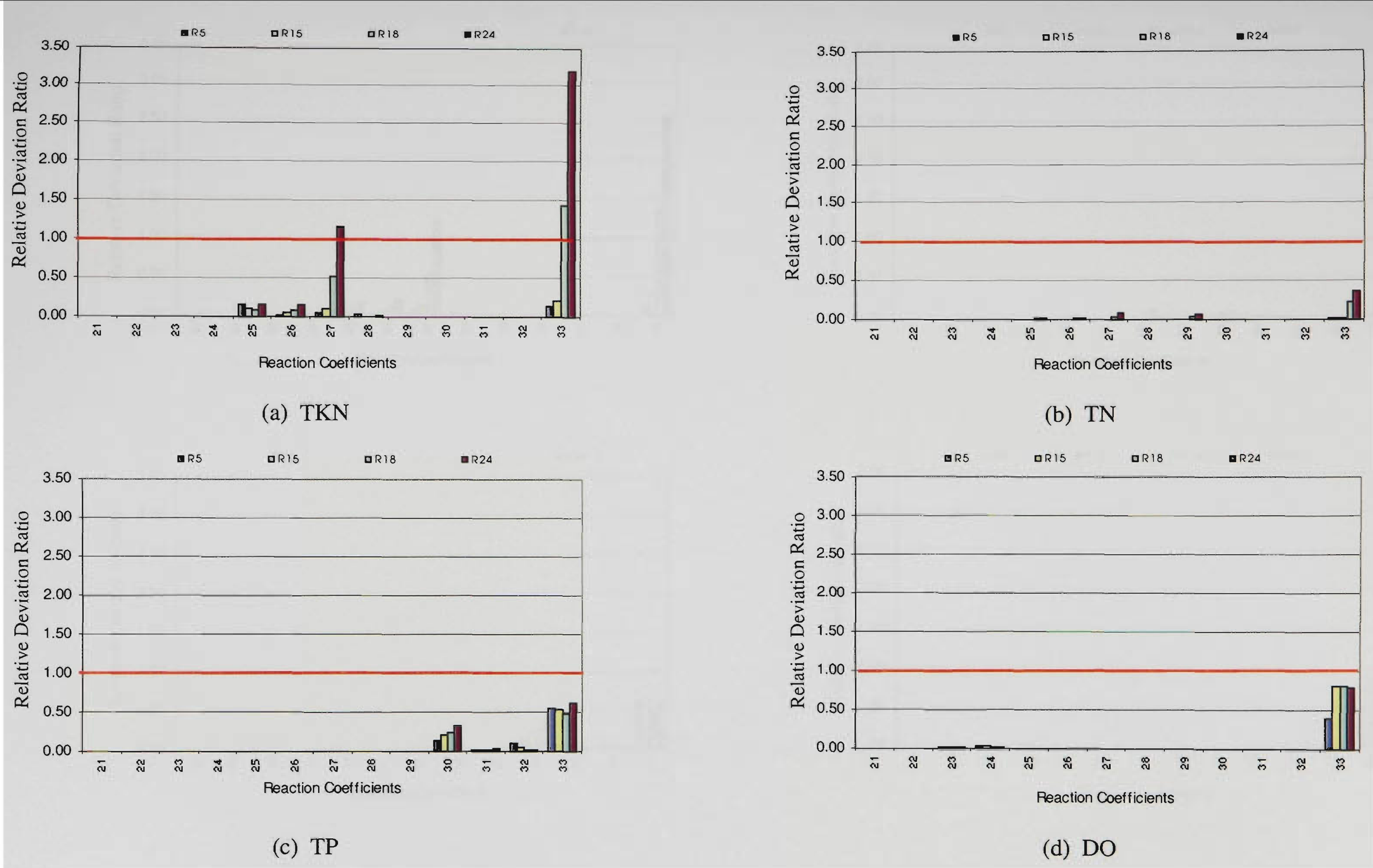
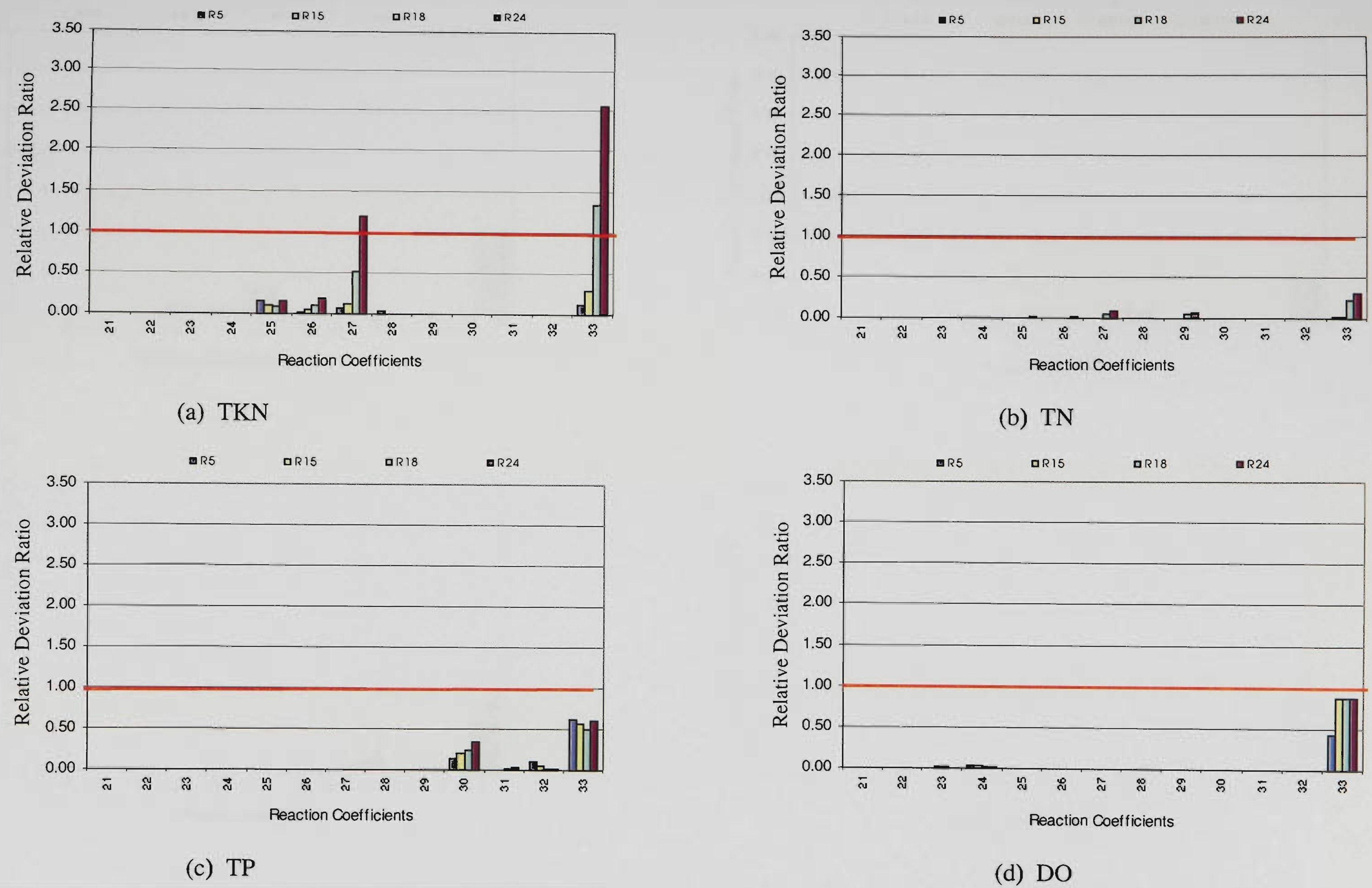


Figure 5.15 Relative Deviation Ratio for 'Lower Range' Flow Event (Minimum CV)



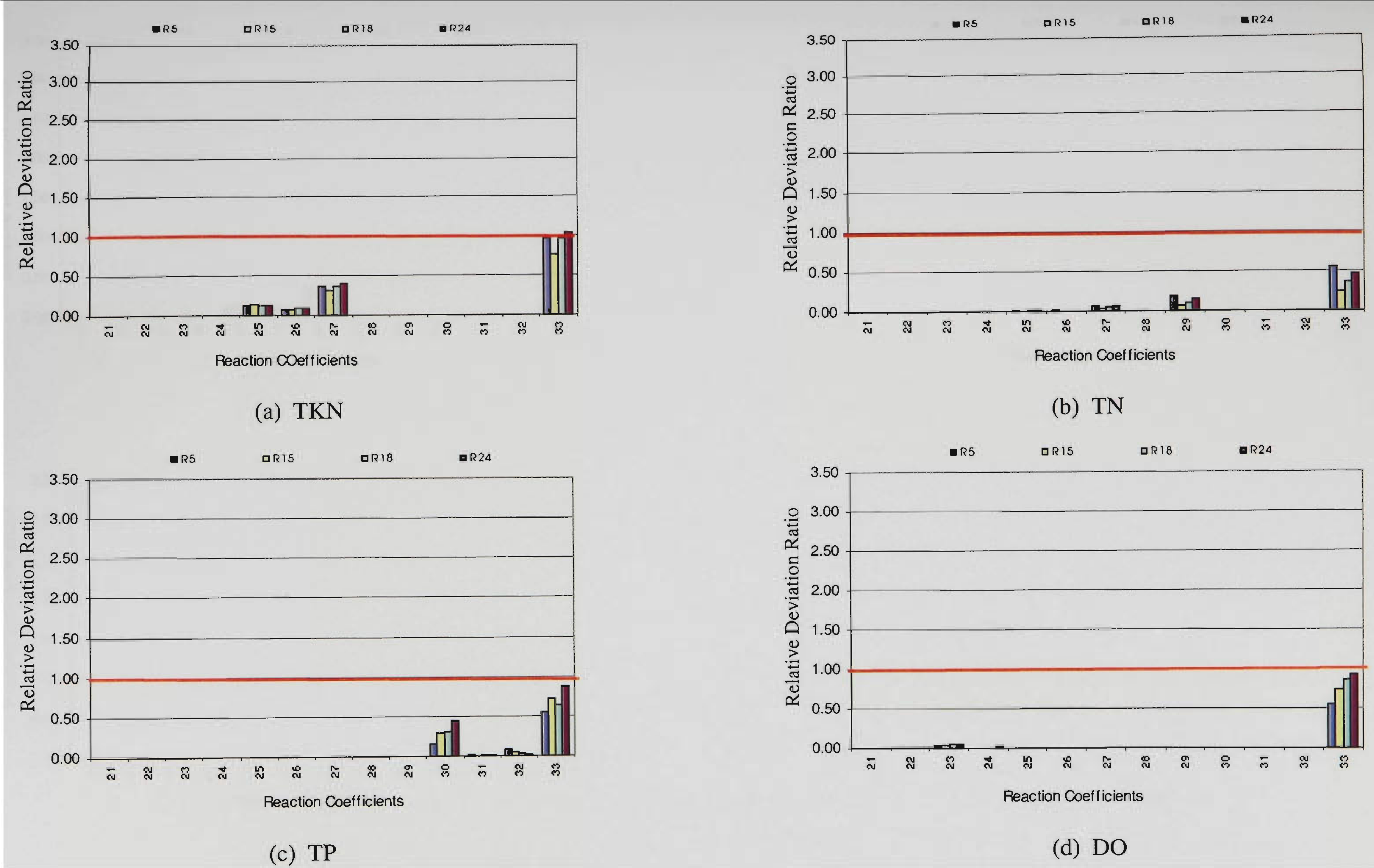
(a) TKN

(b) TN

(c) TP

(d) DO

Figure 5.16 Relative Deviation Ratio for 'Lower Range' Flow Event (Maximum CV)



(a) TKN

(b) TN

(c) TP

(d) DO

Figure 5.17 Relative Deviation Ratio for 'Higher Range' Flow Event (Base CV)

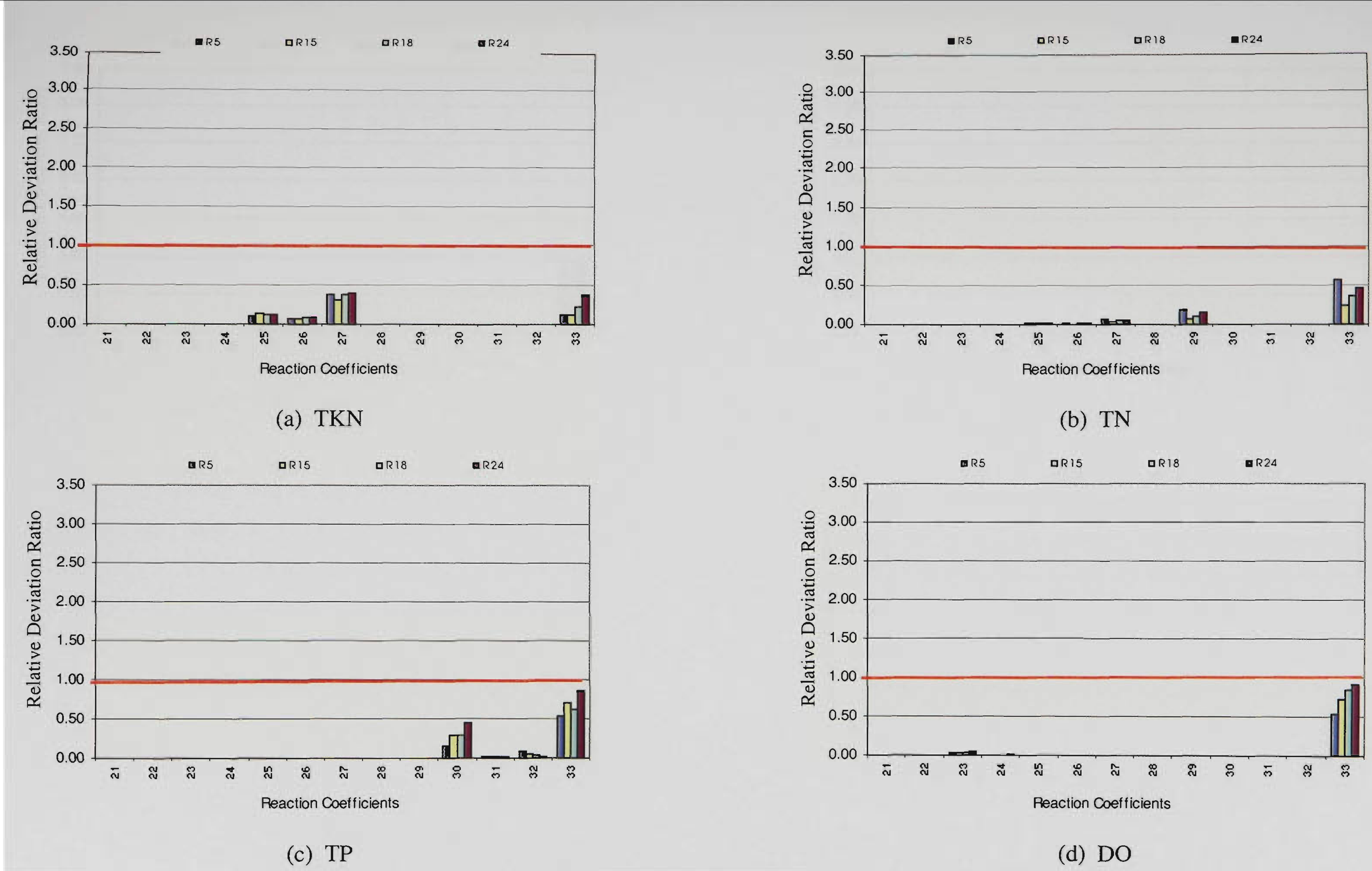
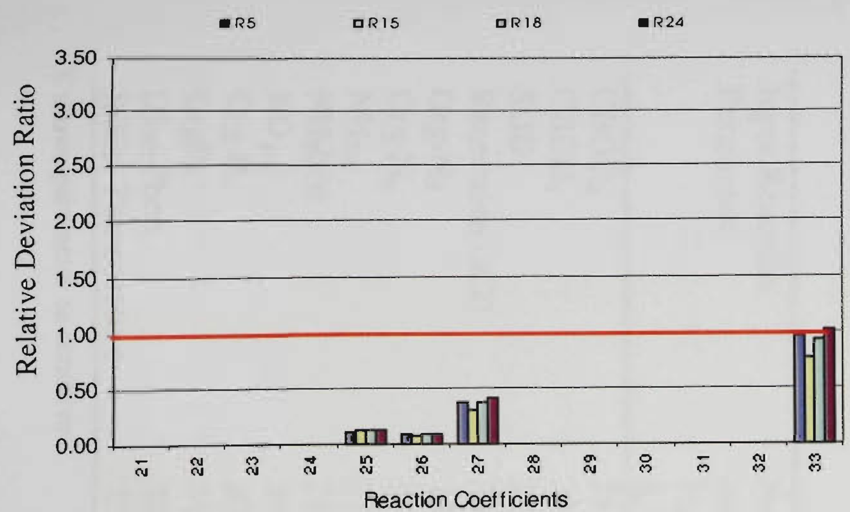
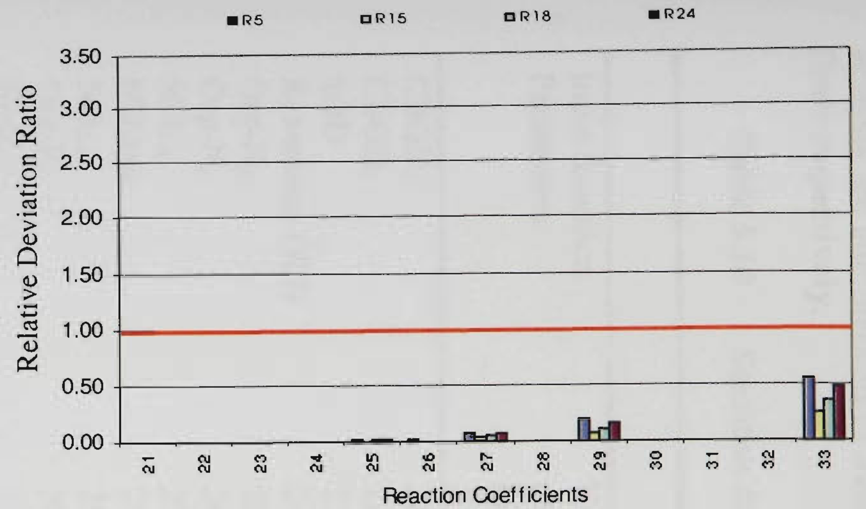


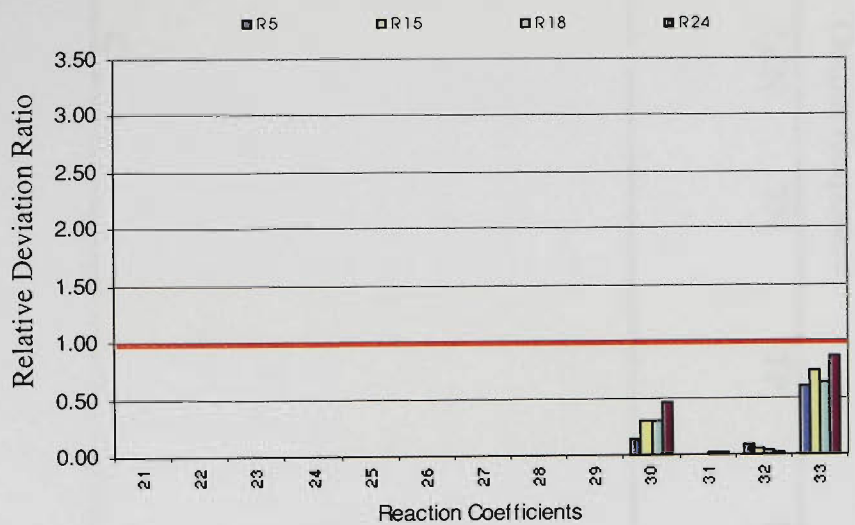
Figure 5.18 Relative Deviation Ratio for 'Higher Range' Event Flow (Minimum CV)



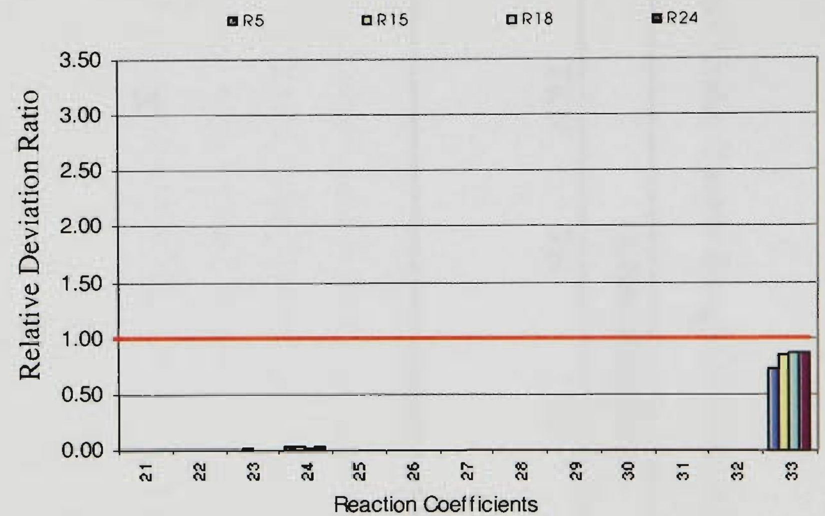
(a) TKN



(b) TN



(c) TP



(d) DO

Figure 5.19 Relative Deviation Ratio for 'Higher Range' Flow Event (Maximum CV) analysis are summarised in Tables 5.10 and 5.11 for 'lower range' and 'higher range' flows respectively.

Table 5.10 Sensitive Decay Rates Under 'Lower Range' Flow Condition

		Output Responses			
Input Reaction Parameters	Numbers used in Figures 5.14-5.19	TKN	TN	TP	DO
$CBOD_d$	21				
$CBOD_s$	22				
SOD	23				
Reaeration (K2)	24				
$Org\text{-}N_d$	25				
$Org\text{-}N_s$	26				
$NH_{3\text{-}d}$	27	X			
NH_3ben	28				
$NO_{2\text{-}d}$	29				
$Org\text{-}P_d$	30				
$OrgP_s$	31				
Diss-Pben	32				
Initial Temperature	33	X			

X shows parameter as sensitive

Table 5.11 Sensitive Decay Rates Under 'Higher Range' Flow Condition

		Output Responses			
Input Reaction Parameters	Numbers used in Figures 5.14-5.19	TKN	TN	TP	DO
$CBOD_d$	21				
$CBOD_s$	22				
SOD	23				
Reaeration (K2)	24				
$Org\text{-}N_d$	25				
$Org\text{-}N_s$	26				
$NH_{3\text{-}d}$	27				
NH_3ben	28				
$NO_{2\text{-}d}$	29				
$Org\text{-}P_d$	30				
$OrgP_s$	31				
Diss-Pben	32				
Initial Temperature	33	X			

X shows parameter as sensitive

As can be seen from Table 5.10, only $NH_{3\text{-}d}$ decay and initial temperature are identified as sensitive to TKN for 'lower range' flows. The $NH_{3\text{-}d}$ decay rate affects TKN only at the most downstream reach of (i.e. R24). This result is generally consistent with that of Section 5.6.2 where it was shown that reaction parameter group was sensitive only to TKN. Both initial temperature and $NH_{3\text{-}d}$ decay rate remain as sensitive parameters irrespective of input CV.

Under 'higher range' flows, only the initial temperature was sensitive to TKN, as seen from Table 5.11. Although, the initial temperature did show as a sensitive parameter to TKN, its RDR was less in comparison to 'lower range' flows, which shows that the temperature effect plays a lesser role under 'higher range' flows. For 'higher range' flows, dilution is generally the dominant process in controlling the river water quality, as shown in Section 3.7

In summary, decay rates are generally not sensitive in effecting output responses (i.e. TKN, TN, TP and DO). Although initial temperature was identified as one of the most sensitive parameters within the group, it is not a decay parameter and it is not one of the parameters that needs to be calibrated. Initial temperature is mainly use for adjustments of decay rates. This shows that the temperature measurement should be done with high accuracy to avoid the errors in affecting the adjustments of decay rates.

5.7 Summary and Conclusions

A pre-calibration uncertainty/sensitivity analysis of model parameters enhances the efficiency of the model calibration. This is because the sensitive parameters identified through the above analysis would be given more attention during calibration compared to less sensitive parameters. An analysis of previous research on river water quality modelling showed that no pre-calibration uncertainty/sensitivity analyses of model parameters were done on river water quality models. Since the results of uncertainty/sensitivity analysis of model parameters of river water quality models are case dependent, the results of previous studies of uncertainty/sensitivity analysis (if any) cannot be transferred for calibration of model parameters of the Yarra River Water

Quality Model (YRWQM). Therefore, it was necessary to conduct uncertainty/sensitivity analysis of model parameters of YRWQM prior to model calibration.

The major objectives considered in this study were to identify the most sensitive parameter groups and to find the most sensitive decay rates within the reaction parameter group. Two low flow events, one 'lower range' and the other 'higher range' were used in this study. The mean (or representative) values for input parameters of all parameter groups except global and reaction groups were determined from the flow events. The default values in QUAL2E were used for mean (or representative) values of input parameters of the global parameter group, while the mean decay rates were determined from Yarra River water quality data and previously published decay rates. Three input parameter coefficients of variations (CVs) were considered covering a reasonable range. Two probability density function (PDFs) distributions for input parameters are available in QUAL2E, namely normal and lognormal distributions, and both of them were considered in this study. Two thousand simulation runs were selected in this study.

The following conclusions were drawn from the study.

- The output distributions of total nitrogen (TN), total phosphorus (TP) and dissolved oxygen (DO) did not show any significant differences due to input parameters sampled from normal and lognormal PDFs, for both flow events.

- In general, it was found that the hydraulic parameter group was the most sensitive parameter group in affecting majority of output responses (i.e. total kjeldahl nitrogen (TKN), TN, TP and DO) under both flow events. In addition to the hydraulic parameter group, the point load group was found to be sensitive for 'lower range' flows, while the incremental group was sensitive to 'higher range' flows. To minimise the errors associated with the output responses the power functions, which are used to model hydraulics should be derived with precision. This can be done by having river cross-sections surveyed at a representative point of each reach. .

- It was found that in general the initial temperature was shown to be the most sensitive parameter to TKN, TN, TP and DO under both flow conditions. In addition to initial temperature, it was found that NH_{3-d} decay rate was sensitive to TKN under 'lower range' flows. Initial temperature was the only parameter found to be sensitive under 'higher range' flow conditions.

CHAPTER 6

CALIBRATION AND VERIFICATION OF YARRA RIVER WATER QUALITY MODEL

6.1 Introduction

Management of river water quality has become increasingly important due to its decline caused by human activities, and this decline in river water quality can be managed through implementation of effective strategies. Water quality models can be used as tools to simulate and assess the cause and effect relationships of river water quality, and then to implement appropriate management strategies to improve river water quality.

In order to use river water quality models effectively, it is necessary to estimate the model parameters. Some of the model parameters can be physically measured, while the remaining model parameters have to be estimated through model calibration. The model calibration is generally done through an iterative process by comparing model predictions with observations. Once the model parameters are obtained through calibration, it is necessary to verify these model parameters by comparing the predictions of the model with observations under different data sets, which were not used in calibration. The calibration and verification of the Yarra River Water Quality Model (YRWQM) is considered in this chapter. The focus of the calibration is on the estimation of decay rates of water quality constituents.

There are several methods available to calibrate mathematical simulation models ranging from trial and error methods to optimisation methods. These methods were reviewed in Section 2.5. A genetic algorithm (GA) optimisation technique was used for calibration of YRWQM, since it has proven to provide a global optimum solution in complex search spaces. As stated in Section 2.6, GA is a stochastic optimisation technique that is based on the concept of natural selection and genetics, which has been applied successfully in many applications. However, there was only one application cited in the literature related to parameter optimisation of water quality models, which was by Mulligan and Brown (1998). To gain the most efficient use of GA (i.e. to reach convergence to the 'optimum' solution quickly), it is necessary to find the best GA operator set. This was also studied in this chapter, since only few studies were conducted in the past on this aspect and the results were inconclusive.

Once the decay rates are found through calibration and validated with different data sets (which were not used in calibration), a post-calibration sensitivity analysis was conducted on the 'optimised' decay rates. This will provide confidence in applying YRWQM for planning and management of Yarra River water quality management strategies.

This chapter first describes the events used for both calibration and verification of model parameters of YRWQM. The investigation of best GA operators in achieving the 'optimum' parameter set was then presented, followed by calibration and verification of YRWQM. Finally, the post-calibration sensitivity analysis of 'optimised' decay rates is presented.

6.2 Flow Events Used for Calibration and Verification

As stated in Section 3.6, the 'lower range' flow events were used in calibration and verification of YRWQM model parameters. This is because lateral (or incremental) flows were negligible for 'lower range' flow events and therefore they can be ignored in calibration and verification of model parameters. This makes the calibration of decay rates simpler, yet gives the correct values of these parameters, since these parameters are not a function of incremental flows, but a function of pollutant characteristics (or composition). Furthermore, the 'lower range' flow condition is often the most critical water quality condition in the river. From the six 'lower range' flow events presented in Section 3.6, three events were selected for calibration and the remaining 3 events were used for verification. These events are shown in Table 6.1.

Table 6.1 Flow Events Used for Calibration and Verification

Calibration	Verification
18/3/92	21/1/92
18/2/92	11/6/96
3/11/95	2/4/97

6.3 Use of Genetic Algorithm for YRWQM Calibration

As stated in Section 6.1, GA is used to estimate water quality decay rates of YRWQM. The details of GA were discussed in Section 2.6. Many public domain GA software have been developed for multi-purpose usage and are available free of charge through the Internet web site http://www.cs.purdue.edu/coast/archive/clife/Welcome.html. In this study, a public domain GA software known as GENESIS (Grefenstette, 1995) was used to optimise water quality decay rates of YRWQM.

GENESIS was selected for this study, since it has been used successfully for different applications (Whitley, 1989; Liong *et al.*, 1995 and Mulligan and Brown, 1998) in the past. YRWQM and GENESIS were linked through their input and output files for each calibration event. The linked YRWQM-GENESIS is described in detail in Section 6.4. This linkage is shown in Figure 6.1 and is discussed below.

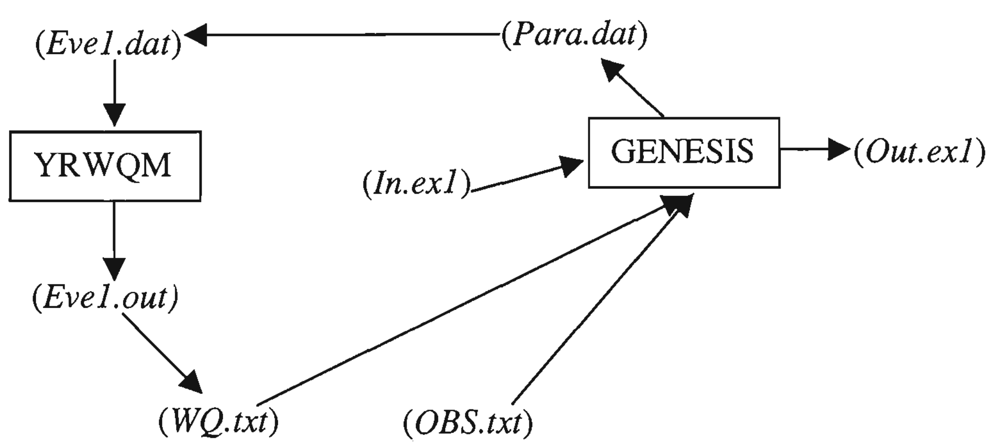


Figure 6.1 Linkage of YRWQM and GENESIS via Input and Output Files

A computer program was developed by the candidate to link the operations of GENESIS and YRWQM to produce 'optimum' decay rates considering a single flow event, within a complete GA run consisting of several generations. For each flow event, before running YRWQM-GENESIS, it is necessary to create two data files by the user outside YRWQM-GENESIS. The first data file is the QUAL2E input file for the six parameter groups (Section 4.4) which has to be created via QUAL2E for the flow event. In Figure 6.1, it is shown as *Eve1.dat.* The second file is *OBS.txt* which contains observed water quality data at the six EPAWQ stations for the flow event.

After preparing these two data files, the other five files shown in Figure 6.1 are automatically created when YRWQM-GENESIS run is initiated. However, these file names are specified by the user. First, the user is prompted with input information by GENESIS on the number of generations, population size, crossover rate and mutation rate, and these information are stored in *In.ex1*. GENESIS then generates the decay rates for one simulation (or one generation) and stores them in *Para.dat.* These decay rates then replaces the reaction parameter group parameters in *Eve1.dat.* YRWQM is then run to produce a standard QUAL2E output file, which is shown as *Eve1.out* in Figure 6.1. The modelled water quality at the six EPAWQ sites were then extracted from *Eve1.out* and stored in *WQ.txt.* GENESIS then uses both *WQ.txt* and *OBS.txt*, and computes the objective function for this parameter set. This process is continued until the generated parameter sets equal the population size for that generation, and the next generation starts. This process continues until the last generation is completed. GENESIS has the option of producing the objective functions of parameter sets for all generations or only for the final generation. These results which include all parameter sets (within one generation) and its corresponding objective functions values are stored in *Out.ex1*. The user selects the 'optimum' decay rates sets from this file.

6.3.1 Procedures used in calibrating decay rates in YRWQM

Four water quality output responses were used in calibration and these are total kjeldahl nitrogen (TKN), total nitrogen (TN), total phosphorus (TP) and dissolved oxygen (DO). These four output responses require estimation of eleven water quality decay rates, which are shown in Table 6.2. This table also shows the influence and relationships of these decay rates on water quality output responses. As discussed in Section 2.3, there is some interaction between these output responses (i.e. TKN affects TN, and both TN and TP affects DO). These interactions are shown in Figure 6.2, and these interactions were considered systematically in the calibration procedure.

Table 6.2 Water Quality Decay Rates Considered in Calibration

Decay rates	Symbols Used in Text	Influence on Output Responses
Org-N decay	Org-N_d	TKN
Org-N settling	Org-N_s	TKN
NH_3 decay	$NH_{3\text{-}d}$	TKN
NH_3 benthos	NH_3ben	TKN
NO_2 decay	$NO_{2\text{-}d}$	TN
Org-P decay	Org-P_d	TP
Org-P settling	Org-P_s	TP
Diss-P benthos	Diss-Pben	TP
CBOD decay	$CBOD_d$	DO
CBOD settling	$CBOD_s$	DO
SOD (sediment oxygen demand)	SOD	DO

The procedure for estimating decay rates was done in a systematic way as shown in Figure 6.2. First, the parameters of the water quality constituents which are not affected by other water quality constituents are estimated. Then, these parameters are kept constant, and the parameters of other water quality constituents are estimated as previously. This procedure has been stated by McCutheon (1989), Wesolowski (1994) and USEPA (1997b). The first set of parameters 'optimised' was Org-N_d, Org-N_s, $NH_{3\text{-}d}$ and NH_3ben, considering TKN. Then, these parameters were kept constant and the second parameter set, $NO_{2\text{-}d}$ was 'optimised' considering TN. The third set of parameter includes Org-P_d, Org-P_s and Diss-Pben, and were 'optimised' using the output response of TP. Optimisation of decay rates for phosphorus can be done in parallel with TKN and/or TN, since TKN and TN are not influenced by TP and vice versa. The last set of parameters of $CBOD_d$, $CBOD_s$ and SOD was 'optimised' using DO keeping all other parameters (already 'optimised') at their 'optimised' values.

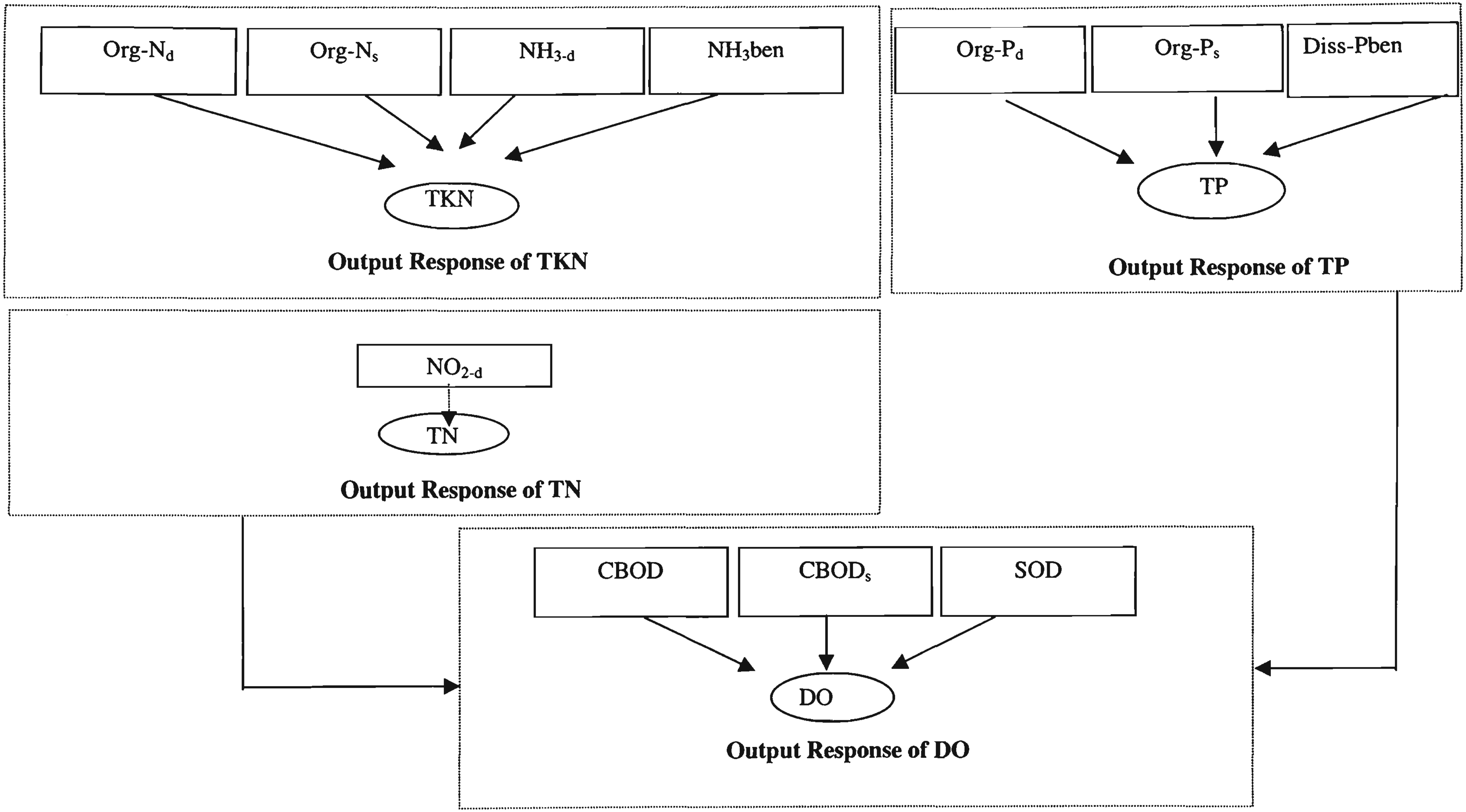


Figure 6.2 Systematic Process Used in Calibration of YRWQM

Due to same wastewater composition discharged from all STPs, the water quality decay rates were considered constant for all 29 Reaches. This is because the rate of reaction of water quality constituents is a function of the wastewater composition.

6.4 GA Capabilities of YRWQM-GENESIS

As stated in Section 6.3, YRWQM was linked with GENESIS to optimise the water quality decay rates of YRWQM. In general, all GA software (including GENESIS) undergo the following procedures within one optimisation run:

- Parameter representation
- Generation of initial population
- Application of selection operator
- Application of recombination (i.e. mutation and crossover) operators

However, different options are available in different GA software for the user to choose the type of coding scheme for parameter representation, initial population generation, the selection operator and the recombination operators. The most commonly used options of GA operators were discussed in Section 2.6.1. The available methods in GENESIS as applicable to YRWQM model are discussed in this section. Before introducing these methods, an overview of the steps involved in one YRWQM-GENESIS optimisation run is discussed. Figure 6.3 shows the schematic of a YRWQM-GENESIS run, which shows the generations by the outer dotted block. The inner dotted block shows the parameter sets within one generation.

The initial stage of the run is to transform the range of decay parameter values (i.e. search space) into some form of coding representation. The next stage is to select an initial population (i.e. first generation), which can be done randomly or heuristically in GENESIS. YRWQM was run with each parameter set of the first generation and the objective function is computed corresponding to each parameter set.

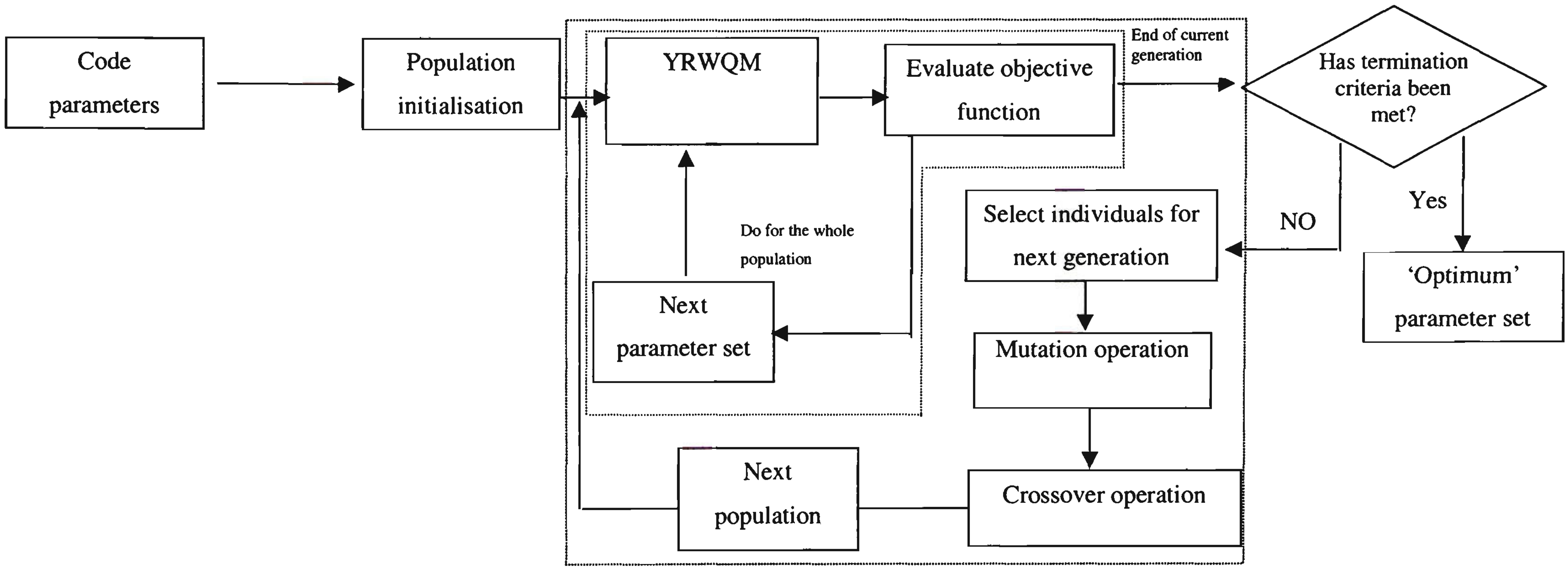


Figure 6.3 Schematic of YRWQM-GENESIS Run

The next step is to select parameters for the second and subsequent generations. These selected parameters undergo transformation through mutation and crossover operations to create better parameter sets for these generations. These parameter sets then become the inputs into YRWQM within this generation and computations are carried out similar to the first generation. These generations are continued until the termination criteria set by the user are met.

The methods available in GENESIS for parameter representation, generation of initial population, selection operator, recombination operations and other related issues are discussed below.

(a) Parameter representation

The parameter coding encodes the range of parameter values into a form that is understood by GA. In GENESIS, only binary and gray representations are available. Due to problem of 'Hamming Cliffs' in binary representation, the gray coding method is selected for use in this study. These terms i.e. binary coding, gray coding and 'Hamming cliffs' are explained in Section 2.6.1.1.

One appropriate binary string length is required to be specified in GENESIS which covers all decay rates. This binary string length depends on the parameter range (or search space) of decay rates, as can be seen from Equation 2.1. In Section 4.4.3.1, the search space for each decay rate of YRWQM was determined. For example, the Org-N_d decay rate range was between 0.006-0.42, as shown in Table 6.3. To determine the required binary string length for Org-N_d, Equation 2.1 of Section 2.6.1.1 was solved. Assuming required precision of 2 decimal places, Equation 2.1 becomes,

$$2^s - 1 \geq (0.7\text{-}0.0042)\text{x } 10^2$$

where s is the required binary string length.

From above inequality, the smallest binary string length required for Org-N_d is 6. Table 6.3 shows the eleven decay rates, their search spaces and the required binary string lengths to achieve 2-decimal precision. As can be seen from Table 6.3, the minimum binary string length required to satisfy all parameters with 2-decimal precision is 9. A binary string length of 10 was used in this study. The use of 10 as the binary string length obviously enhances the accuracy of all decay parameters even beyond 2 decimals.

Table 6.3 Determination of Binary String Length for Use in YRWQM-GENESIS

Decay rates	Search space (day^{-1})	String length for 2 decimal precision
Org-N_d	0.006-0.42	6
Org-N_s	0.001-2.630	9
$NH_{3\text{-}d}$	0.001-0.72	7
NH_3ben	0.001-1.8	8
$NO_{2\text{-}d}$	0.001-0.7	7
Org-P_d	0.001-1	7
Org-P_s	0-0.14	4
Diss-Pben	0.001-1.7	8
$CBOD_d$	0.0042-3.5	9
$CBOD_s$	0.001-1.53	8
SOD	0-2	8

(b) Population initialisation

The population initialsation is the selection of first population (or the first generation) which consists of *N* parameter sets selected from the pre-defined search space. Random and heuristic methods are available in GENESIS to initiate the first population. These methods are explained in Section 2.6.1.2. The random method was used in this study, because no prior knowledge of the likely 'optimum' parameter set. In generating parameters randomly, the 'seed' which initiates the random process can play an important role, in converging to the 'optimum' parameter set. This issue was investigated in Section 6.5.1.2 (a).

(c) Evaluation of objective functions

The evaluation of objective function assesses the strength of parameter sets, which is used to obtain the 'optimum' parameter set. During each generation, each parameter set within the population becomes the input to YRWQM, which was run to produce output water quality responses. The difference in the modelled and the observed water quality is then assessed using an objective function, which can be specified in GENESIS.

Several objective functions are available in the literature to assess modelled and observed responses. However, the most commonly used objective functions in water resource studies were based on simple least squares (Mohan, 1997) and weighted least squares (Little and Williams, 1992 and Mulligan and Brown, 1998). The difference between these two methods is that the weighted least squares method require different weights to be attached to each data point (i.e. in this case, water quality measurement points) in the objective function, whereas, in the simple least square the weights are assumed to be equal. In this study, the objective function based on simple least squares was used, since no information was available on the weights. The simple least squares objective functions is given in Equation 6.1. This equation considers the minimisation of squared difference between observed and modelled water quality concentrations at the six EPAWQ stations. This squared difference in Equation 6.1 is known as the fitness in GA. The lower the value, the fitter is the parameter set.

$$\text{Min} \sum_{i=6} (\text{OBS}_i - \text{MOD}_i)^2 \qquad 6.1$$

where OBS_i is the observed water quality concentration at water quality station i

MOD_i is the modelled water quality concentration at water quality station i

(d) Selection and sampling

The selection and sampling processes determine which parameter sets should proceed into the next generation to produce new parameter sets via mutation and crossover processes. Two selection methods, namely proportionate selection and linear ranking are available in GENESIS to determine the number of parent copies of each parameter set of the previous population that should go into the next generation. The linear ranking selection method was used in this study because it can remove exaggerated difference in the fitness value and high selection pressure during early stages of the process. Furthermore, it specifies the maximum and minimum number of parent copies so that there is a limit of parent copies allocated to parameter sets with high fitness and low fitness values.

Stochastic Universal Sampling (SUS) is the only sampling method available in GENESIS, used to convert the real number of parent copies into an integer number of parent copies accounting for accuracy and precision.

The technical terms used in selection and sampling (i.e. selection pressure and parent copies) were explained in Section 2.6.1.3.

(e) Mutation and Crossover Operations

The mutation and crossover operations generate offsprings (i.e. new parameter sets) by changing and modifying the values of the parent parameter sets which have gone to new generation through selection and sampling processes. Since gray coding is used in this study, the mutation process is undertaken by changing the parameter values by flipping '0' to '1' and vice versa. This mutation process is also the same for binary coding. The crossover operator is applied after mutation in GENESIS. Two point crossover is the only method available in GENESIS. Both mutation and crossover operations are controlled by their rates, which specify how often these actions are to be undertaken.

(f) Miscellaneous operations

Apart from the essential GA operations (i.e. parameter representation, population initiation, selection and sampling, crossover and mutation operators) available in GENESIS, several enhancement options such as generation gap and elitist strategy are also available in GENESIS. The former enhancement option was used in this study in selection and sampling process, while the latter option was used throughout one generation.

The generation gap is defined as the number of parents remain or replaced after each generation. After selection and sampling, the user needs to decide on the number of parent that will survive during the new generation without undergoing mutation and crossover processes. GENESIS has the choice of modelling generation gap either by replacing the complete population by offsprings, (which is known as generational replacement) or replacing a fraction of the parents (selected by the user) by offsprings at random, (which is known as steady state replacement). Results from the previous studies (e.g. Peck and Dhawan, 1995) show comparable results were obtained from both methods. Therefore, in this study, the whole population was replaced totally by their offsprings.

The elitist strategy is designed to preserve and prevent the loss of the best parent from genetic operations of selection, mutation and crossover (Davis, 1991). According to this strategy, the best parent within the previous population will always go to the next generation. Michalewicz (1996) also recommended that elitism should always be applied. This strategy is available in GENESIS and was used in this study to preserve the best parameter set.

6.5 Importance of GA Operators on Model Parameter Optimisation

GA operators are essential components of GA, which include parameter representation, parameter initialisation, selection, mutation and crossover. These operators are responsible for the efficiency in achieving the 'optimum' parameter set. Therefore, it is necessary to study the effect of the GA operators on the 'optimum' solution and the efficiency of achieving the 'optimum' solution.

As discussed in Section 2.6, limited guidelines are available for using GA operators in various modelling applications. Although the significance of GA operators has been studied to a certain extent in rainfall and runoff applications, no such studies have been conducted in river water quality modelling. It was also recommended by Mulligan (1995) that an 'optimum' GA operator set should be found so that it can be used for general purpose water quality modelling, which could be applied to any river configuration.

Numerical experiments were conducted in this study to investigate the significance of GA operators in achieving an 'optimum' parameter set for river water quality models. An initial hypothesis was made that the sensitivity of model parameters had an effect on the best GA operators that should be used in a particular application. Two river networks were considered to study this hypothesis.

- A hypothetical river system with known insensitive and sensitive parameter sets.
- YRWQM river network.

This hypothesis was tested using GA operators obtained from literature subject to capabilities of GENESIS (Section 6.4); this operator set is called 'LIT' set. This 'LIT' set values are taken from rainfall/runoff modelling study of Franchini (1996) and a water quality modelling by Mulligan and Brown (1998), and are shown in Table 6.4. The first column of Table 6.4 is the number of parameter sets in a generation. The second column represents the number of generations in the GA run. Multiplying the population size by the number of generations yields the total number of simulations, which are the number of QUAL2E runs in this case. The last two column represents the crossover and mutation rates respectively.

Table 6.4 'LIT' GA Operator Set

Population size	Number of Generations	Simulations	Crossover Rate	Mutation Rate
125[a]	40[a]	5000	0.6[b]	0.03[b]

[a]Franchini (1996); [b]Mulligan and Brown (1998)

Depending on the outcome of using the 'LIT' set in achieving convergence to the 'optimum' model parameter set, the second stage of the investigation was conducted for both insensitive and sensitive models, provided that the model did not converge within 2% (arbitrarily selected) in all parameters. The insensitive model is defined as a model where the Monte Carlo simulation (MCS) uncertainty and sensitivity analysis yields insensitive parameters for output water quality responses, having relative deviation ratio (RDR) of less than 1 (Section 5.5), and vice versa. If the solution did converge to the 'optimum' parameter set, no second stage was considered.

The second stage of the investigation involves a systematic optimisation of GA operators; the optimised GA operator set is called 'OPTI' set. The operators considered in the optimisation were mutation and crossover rates, because the other operators (i.e. parameter representation, population initialisation and selection) were restricted by the capabilities of GENESIS. However, in addition to mutation and crossover rates, the population size in a GA run was studied first to investigate its effect on the 'optimum' solution.

Prior to the investigation of the hypothesis testing as discussed above, two additional issues were studied which can effect both river networks. These issues were:

- the number of parameter sets that should be considered in the final generation to define the 'optimum' parameter set
- the effect of seed

There were inconclusive findings on the above two issues. Therefore, both of these issues were investigated with both river networks to provide a conclusive result, which can be used in other water quality modelling applications or at least in YRWQM.

6.5.1 Hypothetical river system

6.5.1.1 Development of insensitive and sensitive river models

A small river system consisting of 6 reaches, as shown in Figure 6.4, was considered for the hypothetical river study. This network was modified from an example given in Chapra (1998). An QUAL2E model was developed for this hypothetical river network. Each reach was then sub-divided within QUAL2E into a number of computational elements of 1-km length, where each element was considered as completely mixed. The furthest upstream reach was defined as headwater, and the boundaries of reaches were shown with black dots, as indicated in Figure 6.4. The tributary inflow and the discharge of effluent from STP were indicated with arrows.

Data for all six input parameter groups (i.e. global, hydraulics, reaction, incremental, headwater and point load) required to develop the QUAL2E model were the same for both insensitive and sensitive model, unless stated otherwise and were obtained from Chapra (1998). The incremental flow was considered negligible in this exercise. The *Manning's* method instead of power functions was used to model hydraulics of river reaches. The respective BOD_d concentrations discharged from STP and tributary were considered as 200 mg/L and 5 mg/L respectively for the insensitive model (Chapra, 1998), while 400 mg/L and 100 mg/L was used for the sensitive model. The decay rate group was considered in this exercise as the parameters to be optimised, since the calibration of the decay rates is the main focus of this chapter. The output response of

DO was considered as in Chapra (1998) The decay rates which effect DO in this exercise were BOD_d, BOD_s, SOD and reaeration only. The decay rates were known from Chapra (1998) and are shown in Table 6.5. This table also shows the parameter search space used in GA optimisation of model parameters. The reaeration rate was determined within QUAL2E as a function of depth and velocity using the O'Connor and Dobbins method (Brown and Barnwell, 1987), as in Chapra (1998).

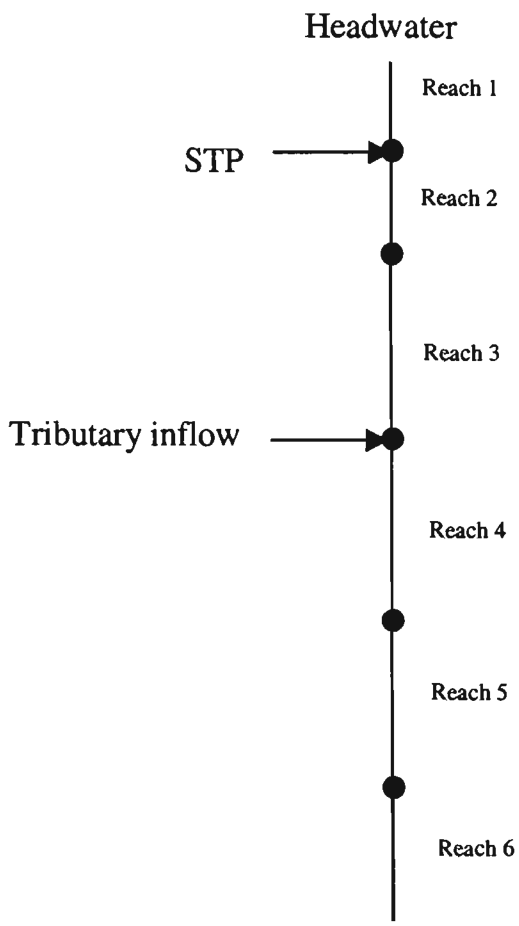


Figure 6.4 Hypothetical River System

Table 6.5 Known Decay Rates and Parameter Search Space Used in GA Runs

Decay Rates	Known Parameter Value (per day)	Parameter Search Space
BOD_d	0.50	0.0042-0.7
BOD_s	0.25	0.03-1.53
SOD	5.00	0.05-7

(a) Insensitive model

In order to create an insensitive model (as defined in Section 6.5) for this hypothetical network, the decay rates given in Chapra (1998) were first considered. These decay rates were 0.50, 0.25 and 5.0 for BOD_d, BOD_s and SOD respectively, as also shown in Table 6.5. A MCS uncertainty and sensitivity analysis of model parameters was conducted to investigate the sensitivity of these parameters on DO, as in Section 5.6.2. The methodology adopted was as follows. By using known input data of Chapra (1998) and decay rates (Table 6.5) which are assumed to be error free, a QUAL2E run was conducted first to obtain the water quality concentration for DO for the six reaches. This output DO concentration obtained was then considered as the observed concentration for use in GA calibration. The decay rates (i.e. BOD_d, BODs and SOD) in GA calibration were then treated as unknown.

All input parameters in six parameter groups were perturbed simultaneously in determining the overall sensitivity of the input parameters to DO output water quality. Three reaches were selected for the output analysis and they were Reaches 1, 2 and 4. Reaches 2 and 4 were selected because they experience high influence from inflows and concentration from STP and tributary. Reach 1 was an additional reach, arbitrarily selected. Three values of coefficient of variation (CV) for input parameters (i.e. 0.1, 0.175 and 0.25) were used in MCS and found that RDR was less than 1 for all three reaches. Therefore, the model with the decay rates of Chapra (1998) can be considered as an insensitive model and used in this study to study the effect of GA operators on optimum parameter set.

(b) Sensitive model

The input data including the decay rates for this model was same as in the insensitive model except for that point source BOD concentrations as stated earlier. The methodology used in the insensitive model was also applied to this model. The MCS analysis showed that the RDR values for DO responses for Reaches 1, 2 and 4 were all greater than 1. Therefore, this model can be considered as a sensitive model and used in this study to investigate the effect of GA operators on optimum parameter set.

6.5.1.2 Initial investigation of number of parameter sets and the effect of seed

(a) Optimum parameter set from final generation

The hypothetical river network insensitive model was initially considered in this investigation. In a typical GA optimisation run, there will be several solutions (in this case parameter sets) in the final generation that can be considered equally good as the solution with the 'best' objective function. All these solutions differ only by a small amount in their objective functions, yet there could be some difference in their actual values (i.e. in the parameter values). Therefore, it is not appropriate to select the solution with 'best' objective function value. However, to date, no study has clearly specified the number of parameter sets that should be considered from the final generation to derive the 'optimised' parameter set. Mulligan and Brown (1998) adopted the mean of the best 50 parameter sets (in terms of the objective functions) from the last generation for decay rates in their river water quality model. Franchini and Galeati (1997) determined the mean value of the best 20 parameter sets (in terms of the objective functions) in their rainfall runoff model. Wang (1991), Liong *et al.* (1995), Mohan and Loucks (1995), and Meier and Barkdoll (2000) selected the best parameter set based on the minimum objective function from the last generation as the 'optimum' parameter set. No justification was given in any of the above studies for adopting of these numbers. Therefore, a study was conducted in this part of the study to investigate the number of parameters that should be considered from the final generation to obtain the 'optimum' parameter set.

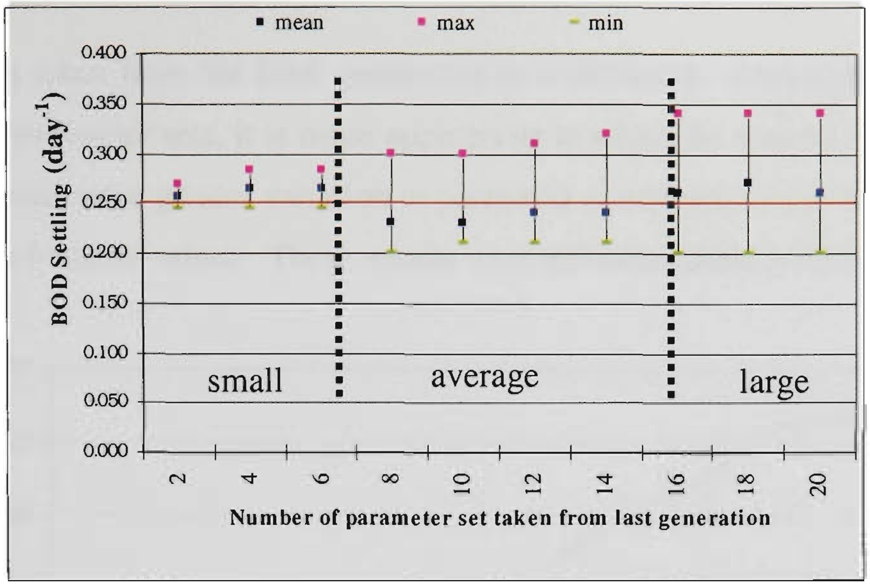
The population size was hypothesised as a factor in determining the number of parameter sets to be considered from the final generation. Therefore, three other population sizes (in addition to the 'LIT' set of population size of 125) as shown in Table 6.6, were investigated. The number of simulations, crossover rate and mutation rate was kept constant in these four runs. Because of the constant number of simulations in four runs, the increase of the population size decreased the number of generations in the runs. The insensitive model of the hypothetical river network was initially considered for this part of the study and the GA optimisation was carried out for each of the four runs. In addition, the sensitive hypothetical model as well as the YRWQM river network was also considered in later simulations.

Table 6.6 Runs Used to Determine the Optimum Number of Parameter Sets Taken From the Last Generation

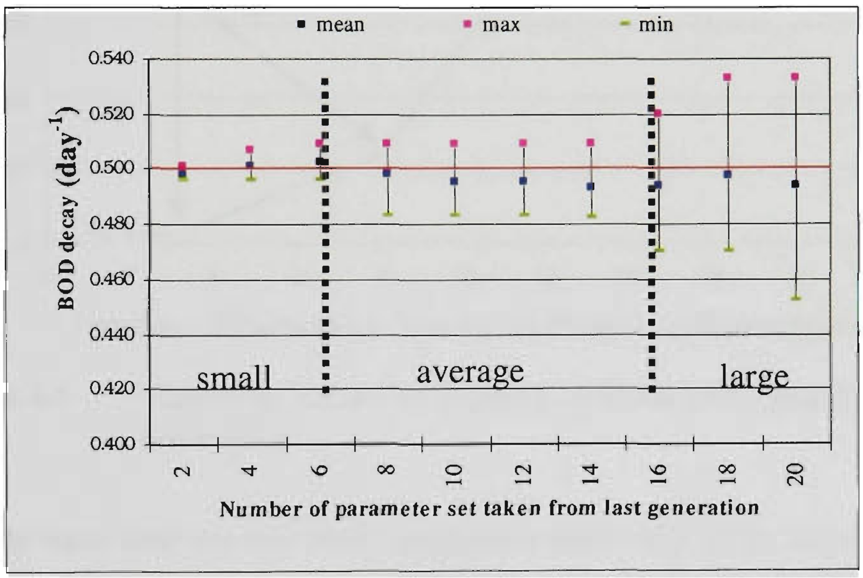
Run	Population Size	Generations	Simulations	Crossover Rate	Mutation Rate
1	125	40	5000	0.6	0.03
2	250	20	5000	0.6	0.03
3	500	10	5000	0.6	0.03
4	1000	5	5000	0.6	0.03

The results of this experiment were analysed to study the variation in the parameter range. Figure 6.5 shows the plot of decay rate ranges versus the number of parameter sets taken from the last generation for the population size of 125 of the insensitive hypothetical model. Each plot shows the minimum, maximum and mean parameter value calculated up to the top best 20 parameter sets of the last generation, in increments of 2. As shown in Figure 6.5, the results clearly indicate 3 distinct segments showing small, average and large ranges. In general, the variation in the parameter range becomes larger as the number of parameter sets considered increases. Although the 'small' segment has the lowest objective functions and the smallest range for the parameters, this segment was not considered any further since it contained only 2 to 4 parameter sets which is considered to be insufficient to derive the best parameter set. The average segment as indicated in Figure 6.5 showed a reasonable parameter range that allows more parameter combinations, yet remained in the low band of objective functions and therefore, it was considered to derive the best parameter set.

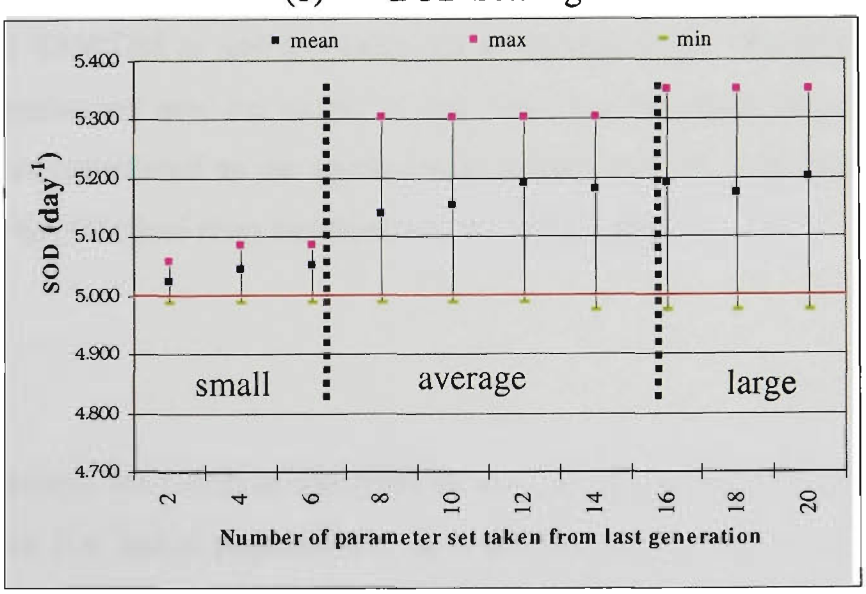
Data in Figure 6.5 were plotted to show the objective functions corresponding to number of parameter sets taken from the last generation of each of the four runs. Figure 6.6 shows the rate of change of the objective function with respect to number of parameter sets taken from the last generation, for population size of 125. This plot indicates the slope increases in the objective function value as the number of parameters taken from the last generation increased. Therefore, it is reasonable to keep the number



(a) BOD Decay



(b) BOD Settling



(b) SOD

Figure 6.5 Parameter Range Vs Number of Parameter Sets Taken From Final Generation of parameters taken from the final generation to a minimum. Although the first slope changes at 5 parameter sets, it is more appropriate to adopt the second slope change (at 8 parameter sets) since greater variation in parameter combinations can be obtained with low objective function values. These results were quite consistent with Figure 6.5.

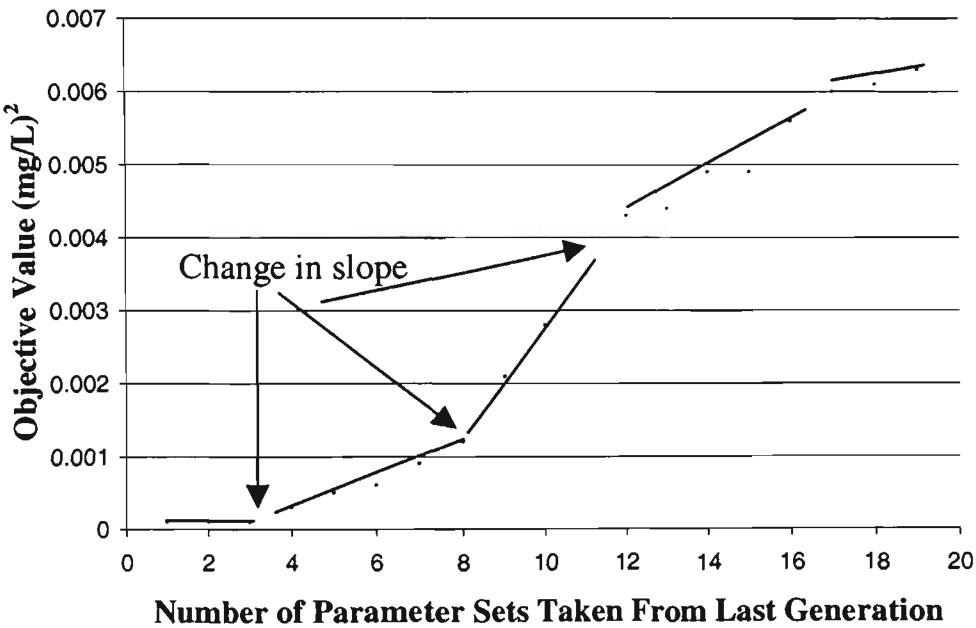


Figure 6.6 Objective Value Vs Number of Parameter Sets Taken from Last Generation

Similar results were seen for the other population sizes with slope cut off point within 10 parameter sets. This result was also consistent with the sensitive hypothetical model and also with YRWQM as will be described in Section 6.5.2. Therefore, the mean of the best 10 parameter sets (in terms of the objective function) taken from the final generation was considered as the optimum parameter set and was used in subsequent studies in the hypothetical river network and YRWQM studies.

(b) Seed

Seed is a parameter set which is specified by the user to generate the population for the first generation (i.e. initial population). It is the first parameter set of the population, which is combined with random numbers to generate the remaining parameter sets of the population. Different seeds generate different parameter sets for the population, and therefore the seed may play a role in the 'optimum' parameter set obtained from the final generation of GA optimisation.

Several studies have explicitly and implicitly considered the effect of seed in parameter optimisation applications (Wang, 1991; Franchini, 1996 and Mohan, 1997). Wang (1991), in his conceptual rainfall and runoff model compared the best parameter sets from 10 runs, which were initiated with different seeds. Of the 10 runs, he found that 8 runs had given the same optimum parameter set with the same objective function value. The objective function values for the other 2 runs were marginally higher, but the parameter set was of the same order as in the other 8 runs.

Franchini (1996) considered the effect of seed in obtaining the optimum parameter set in his conceptual rainfall and runoff model. Similar to Wang (1991), Franchini also analysed the optimum parameter set with 10 runs of different seeds and considered the best parameter set as the 'optimum' parameter set. He found that the objective function value corresponding to the 'optimum' parameter set of the 10 runs were almost the same, but with a significant difference in the actual value parameter values. He claimed that this was attributed to the errors in the data and the imperfect structure of the model which caused some of the parameters insensitive.

Mohan (1997) used 20 different seeds to initiate the GA runs in his nonlinear Muskingum flood routing model, because he acknowledged that parameters are initiated randomly in GA. He adopted the commonly used simple least squares of the actual and routed outflows as the objective function. The best parameter set of 20 runs were considered as the 'optimum' parameter set, but he did not comment on the results generated from the 20 seeds.

Based on the above studies, the findings on the effect of seed in obtaining the 'optimum' parameter set was inconclusive. Therefore, it was necessary to study the effect of seed in achieving the 'optimum' parameter set. The methodology used is described below and is valid for both hypothetical and the YRWQM model. The discussion on YRWQM is given in Section 6.5.2.

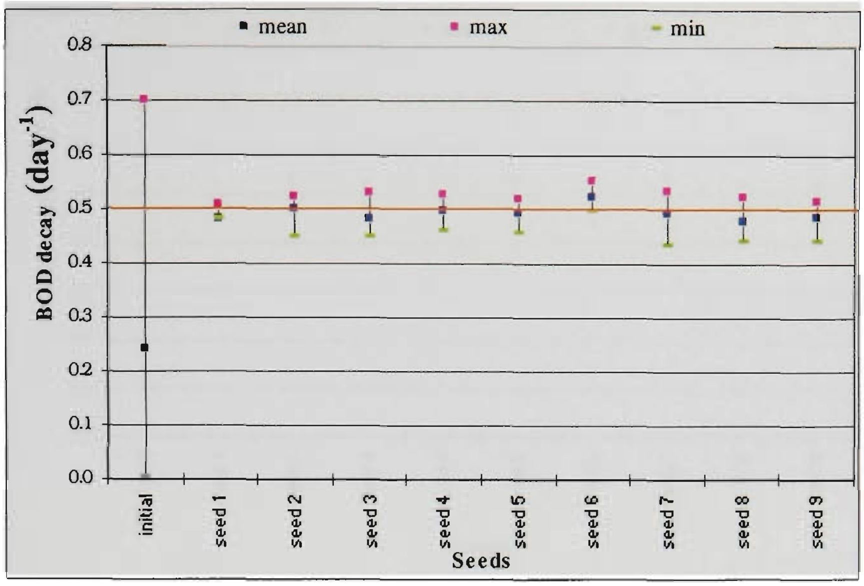
Nine seeds were considered for GA optimisation of the hypothetical river network insensitive model. For each seed, several GA runs were used with different parameter search spaces. The parameter search space used in the first GA run was shown in Table 6.5 (which is also repeated in Table 6.7) for hypothetical models. The parameter search spaces used in subsequent GA runs were obtained from the envelope of the minimum and maximum parameter ranges (of 125 parameter sets) of the previous GA runs with respect to nine seeds. Note that this envelope was obtained from the whole population of the last generation and not the best 10 parameter sets as found in Section 6.5.1.2. This process was continued until the search space reduced to a narrow range. In this case, it can be said that the seed did not play a role in determining the 'optimum' parameters. However, there can be cases where the search space may not reduce to a narrow range and the parameter range is then quite different with respect to different seeds, which says that the seed plays a role in determining the 'optimum' parameters.

For the hypothetical river network insensitive model, only 3 GA runs were required to obtain a narrow range for the parameter search space. Figure 6.7 shows the mean and the range for BOD_d, BOD_s and SOD decay rates obtained from the best 10 parameter sets corresponding to each of the 9 seeds. The initial parameter search space is also indicated on each plot to demonstrate the convergence of the GA solution. This figure also shows the actual parameter values (i.e. red line on each of the plots). In general, the results indicated that the parameter sets generated with 9 different seeds produce different parameter ranges and means.

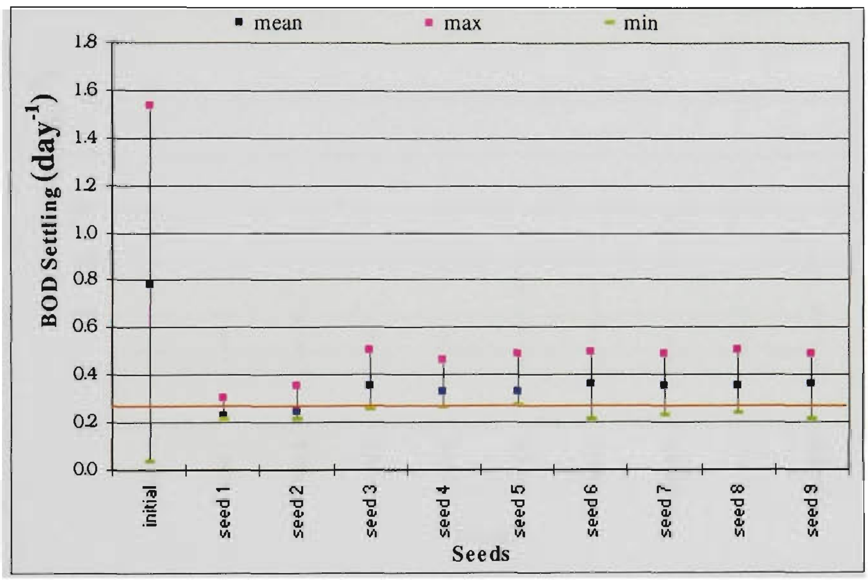
Table 6.7 Search Space Used for Parameters in Different GA Runs for Hypothetical River Network Model

	GA Run 1	GA Run 2	GA Run 3
BOD_d	0.0042-0.7	0.101-0.676	0.433-0.563
BOD_s	0.03-1.53	0.139-1.247	0.108-0.555
SOD	0.05-7.00	3.929-6.755	4.418-6.552

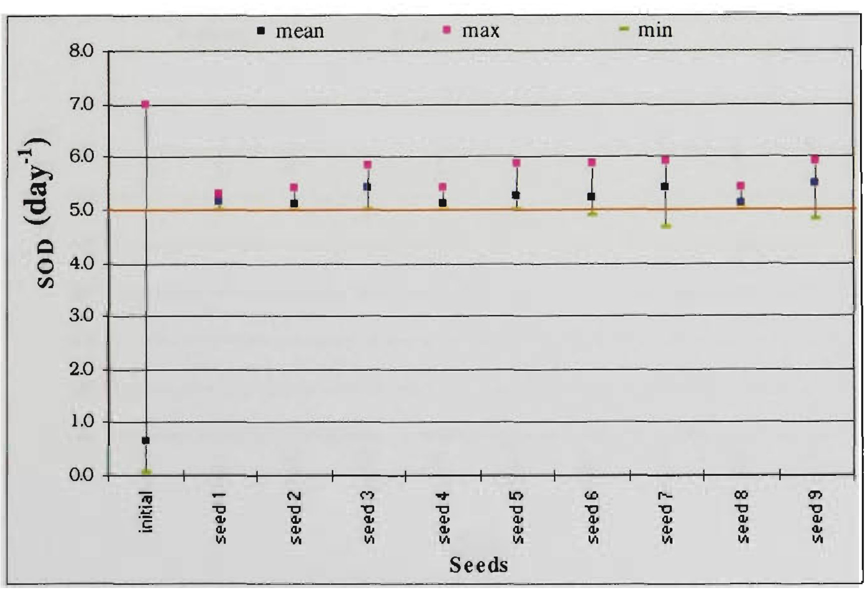
Similar information was plotted on Figures 6.8 and 6.9, based on the results of the GA runs using the reduced search space obtained from the previous GA run. Note that the reduced range obtained from previous run has replaced the initial range shown on each



(a) BOD Decay

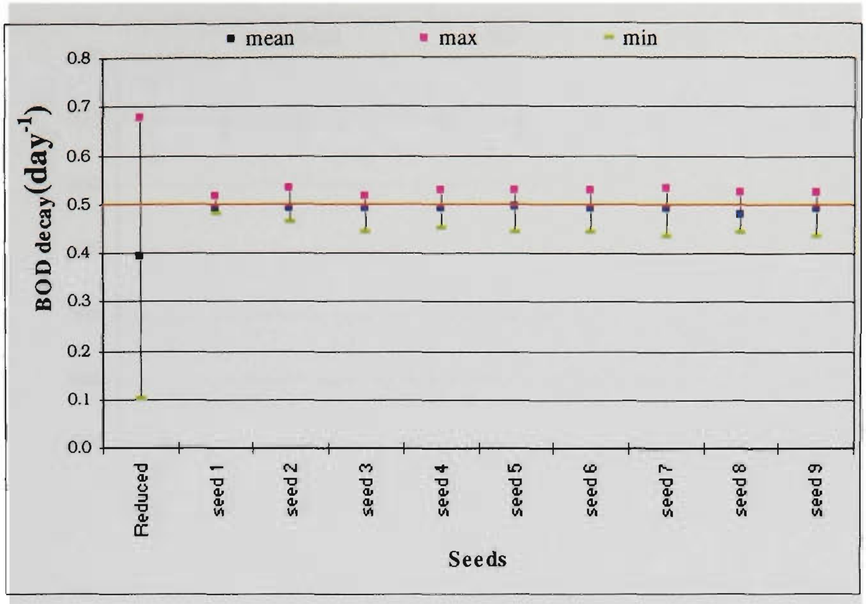


(b) BOD Settling

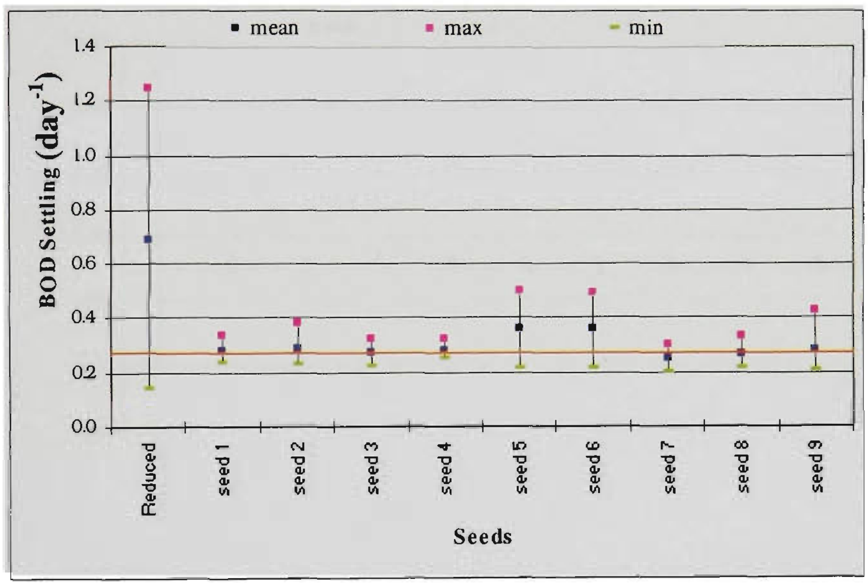


(c) SOD

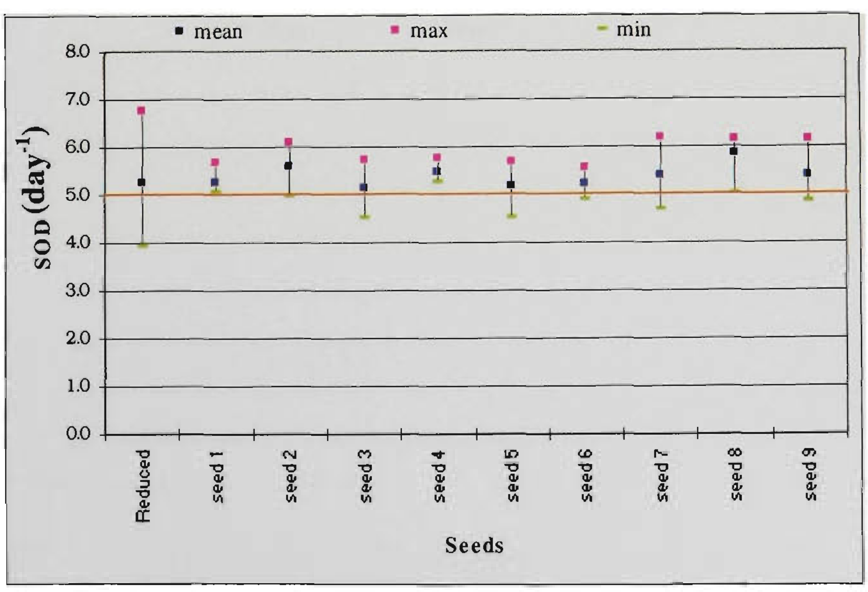
Figure 6.7 Effect of Seed – GA Run 1 (Statistics Based on Best 10 Values)



(a) BOD Decay

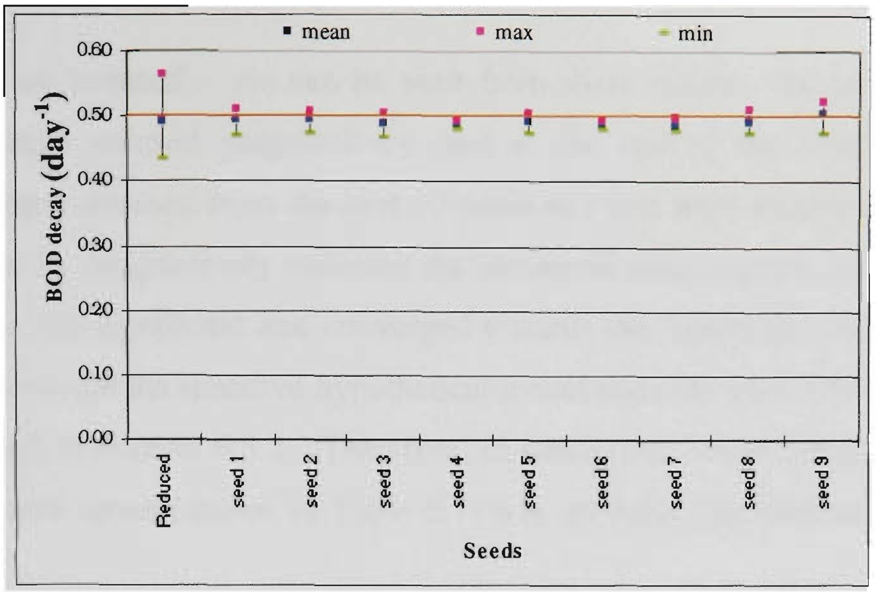


(b) BOD Settling

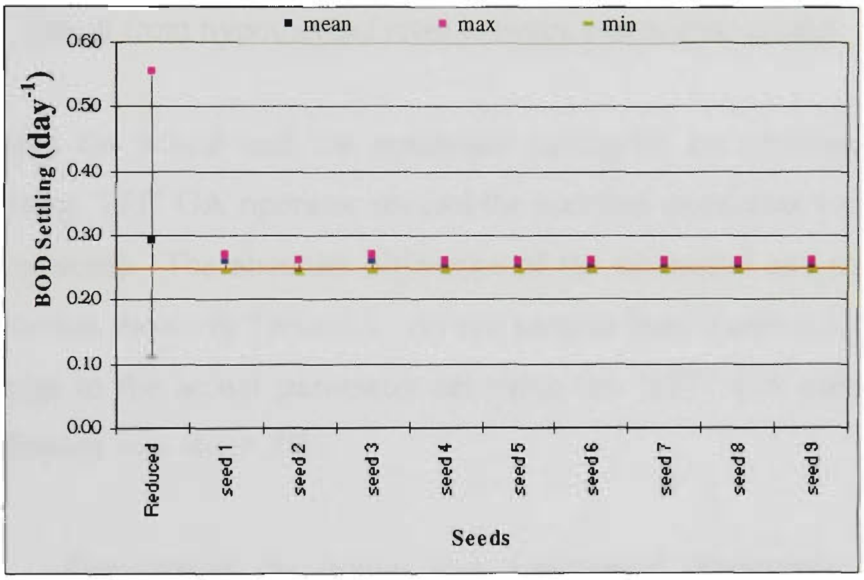


(c) SOD

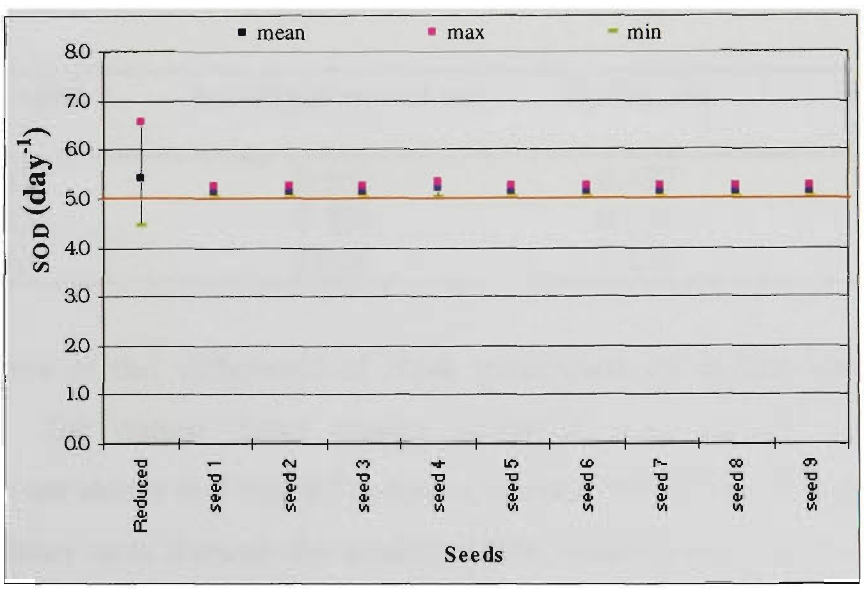
Figure 6.8 Effect of Seed – GA Run 2 (Statistics Based on Best 10 Values)



(a) BOD Decay



(b) BOD Settling



(c) SOD

Figure 6.9 Effect of Seed – GA Run 3 (Statistics Based on Best 10 Values)

plot labelled as 'reduced'. As can be seen from these figures, the parameter search space have been reduced progressively, and at the end of the third GA run, the parameter ranges obtained from the best 10 parameter sets were almost the same. This suggested that by progressively reducing the parameter search space, the problem with seed becomes less significant and converged towards the 'optimum' solution. Similar result was shown for the sensitive hypothetical model and also with YRWQM, which is to be described in Section 6.5.2. Therefore, in subsequent investigations, the reduced parameter search space (shown in Table 6.7) was used for hypothetical river network models.

6.5.1.3 Result from hypothetical river network insensitive model

Table 6.8 shows the actual and the optimised parameter set obtained through GA optimisation using 'LIT' GA operator set and the reduced parameter range (to account for the effect of seed). The absolute difference of the optimised and the actual to the parameter set is also shown in Table 6.8. As can be seen from Table 6.8, the decay rates did not converge to the actual parameter set using the 'LIT" GA operator set. The maximum difference was about 8%.

Table 6.8 Comparison on Actual and 'Optimised' Parameter Sets Obtained From 'LIT' GA Operator Set for Hypothetical River Network Insensitive Model

Decay rates	Actual parameter set	'Optimised'	% absolute difference
BOD_d	0.500	0.497	0.6
BOD_s	0.250	0.231	7.6
SOD	5.000	5.153	3.0

The significance of this difference of input parameters on output water quality was investigated. The output water quality modelled with 'actual' and 'optimised' parameter sets are shown in Table 6.9. As can be seen from Table 6.9, the difference of up to 8% in decay rates showed the modelled DO concentration within an accuracy of 0.24%. This is mainly due to the insensitivity of the model parameters in this insensitive model.

Table 6.9 Comparison of Actual and Modelled DO Concentrations (Insensitive Model)

Reach	DO (mg/L) Using Actual Parameter Set	DO (mg/L) Using 'Optimised' Parameter Set	% absolute difference
1	7.27	7.26	0.14
2	4.17	4.16	0.24
3	3.97	3.97	0.00
4	4.60	4.60	0.00
5	4.85	4.84	0.21
6	5.17	5.16	0.19

Since the convergence of the decay rates was not achieved in the insensitive model, a systematic optimisation of GA operators was conducted to investigate whether the GA operators play a role in achieving convergence to the actual parameter values in an insensitive model. As found from Section 6.5.1.2(b), the effect of seed can be considered negligible by using the reduced range, therefore, the reduced range shown in the last column in Table 6.7 was used throughout this investigation.

This part of the study can be considered as an academic study. This is because even if the 'optimised' GA operators produces the 'optimum' model parameter set close to the actual parameters, the output responses of DO would not change by a large margin, since already DO had been produced within 0.25% of the actual values with the 'optimised' model parameter obtained from the 'LIT' GA operators. Nevertheless, a systematic optimisation of GA operators was considered on population size, mutation rate and crossover rate, as described below.

(a) Population Size

The first GA operator to be optimised is the size of population used in one generation. The population consists of N number of parameter sets. The product of population size and the number of generations gives the number of simulations (in this case QUAL2E simulations) in a GA run. The balance between the total number of simulations and the population size (or the number of generations) can be an important factor in the convergence of the solution and ultimately the efficiency of GA (Grefenstette, 1986).

When the population size is small (i.e. 20-40 parameter sets), the variation in parameters within the population is small and then there is a danger that the 'optimum' can trap in a sub-optimal solution. On the other hand, when the population size is large (i.e. greater than about 90 parameter sets), there are more parameter combinations within one population size however, it becomes inefficient in the convergence because more generations are required (Grefenstette, 1986). Franchini (1996) compared the objective function in reaching convergence for 125, 250, 500 and 1000 population sizes, each with a total number of 5000 simulations. He found that the population size of 125 converged within 5000 simulations, while the large population size of 1000 did not converge. Furthermore, he found that with large population size of 1000, the number of simulations had to be increased to 20000 to reach convergence.

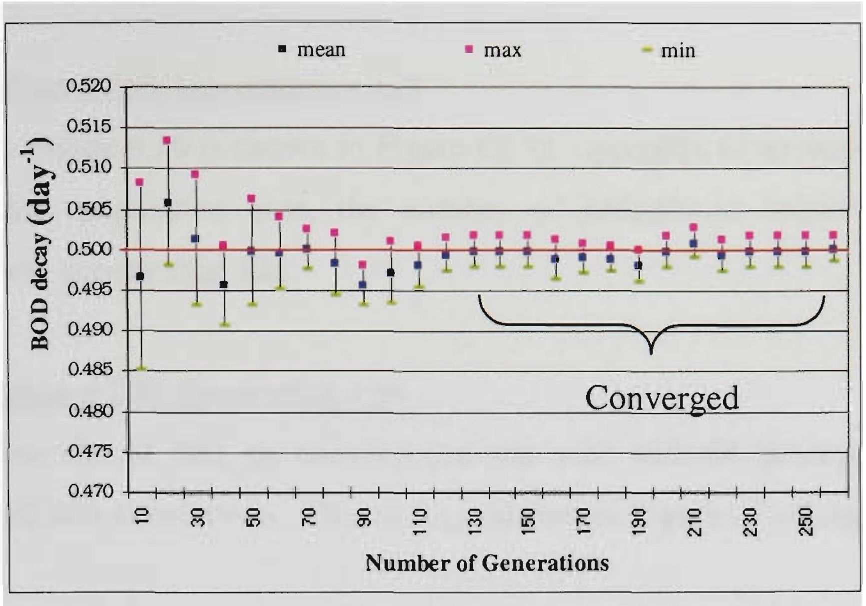
In this section, a similar study to Franchini (1996) was conducted to find an optimum population size. Four different population sizes of 125 (i.e. 'LIT' set), 250, 500 and 1000 were investigated, as shown in Table 6.10. The number of simulations (or parameter sets) used for all these population sizes were kept constant at 32,000, which was the maximum limit in GENESIS. The crossover and mutation rates were kept at 0.6 and 0.03 respectively as in Mulligan (1995). The results for different combinations of population sizes and generations are described in parts (i) to (v) below.

Table 6.10 Population Sizes Used to Find Optimum Balance

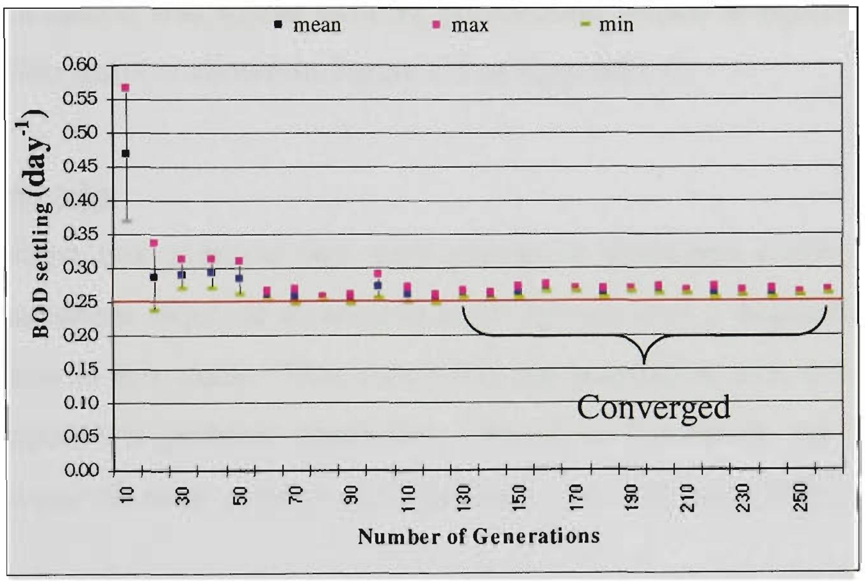
Run	Population size	Generations	Simulations	Crossover Rate	Mutation Rate
1	125	256	32000	0.6	0.03
2	250	128	32000	0.6	0.03
3	500	64	32000	0.6	0.03
4	1000	32	32000	0.6	0.03

(i) Population = 125, Generations = 256

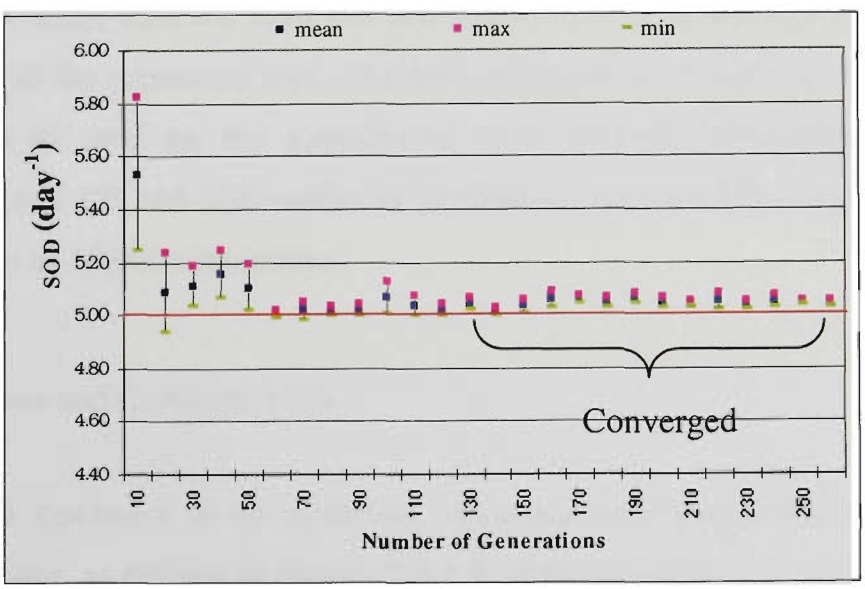
Figure 6.10 shows the mean and the range of decay rates after each generation for population size of 125. With this population size, the number of generations required to achieve convergence is around 120, which represents 15000 simulations.



(a) BOD Decay



(b) BOD Settling



(c) SOD

Figure 6.10 Convergence with Parameter Population Size of 125

(ii) Population = 250, Generation = 128

Similar plot to Figure 6.10 is shown in Figure C1 of Appendix C for population size of 250. With this population size, the number of generations required to achieve convergence was greater than 128.

(iii) Population = 500, Generation = 64

With population size of 500, no convergence was seen with 64 generations, which is equivalent of 32,000 simulations. This result is shown in Figure C2 of Appendix C.

(iv) Population = 1000, Generation = 32

Again no convergence was found with 32 generations, which is equivalent of 32,000 simulations. This result is shown in Figure C3 of Appendix C.

(v) Final remarks

The findings from this exercise was very similar to Franchini (1996), although the number of generations required to achieve convergence with a population size of 125 was much greater in this study. This shows that the population size and the number of simulations required is problem dependent. Based on the result, the most optimum population size and the total number of simulations were 125 and 15,000 respectively.

In subsequent GA operator optimisation (in selecting best GA operators), the best 10 parameter sets taken from the final GA generation were considered to derive the mean and the range of the parameter sets. The reduced parameter range was used to account for the effect of seed for the hypothetical river network insensitive model. The population size of 125 and 120 number of generations were used in these studies, which was equivalent to 15,000 simulations.

(b) Mutation and Crossover rates

The final GA operators to be optimised were mutation and crossover rates. The mutation operator, as defined in Section 2.6.1.4, adds variability (i.e. changes the value) to the selected parent model parameter set by randomly selecting and altering the values of the selected model parameters. The crossover operator exchanges model parameter values from two selected model parameter sets as defined in Section 2.6.1.5. These two operators effectively create offsprings for the next population. Selection of optimum values for both of these rates is very important in GA optimisation for efficient convergence to the 'optimum' parameter set. The criteria in controlling the mutation and crossover rates are different. The mutation rate needs to be controlled to avoid high rates, because by flipping of bits so frequently within the new generation, the population becomes random, which is similar to the initial population. On the other hand, a low rate should be avoided in crossover to prevent any stagnation (or convergence to a local optimum) in the search (Grefenstette, 1986). These two rates simultaneously determine the rate of convergence of the GA optimisation. Goldberg and Deb (1991) pointed out that a good combination of mutation and crossover rates is essential to effectively use GA for parameter optimisation.

Since the mutation and crossover rates simultaneously determine the rate of convergence, they should be studied together. However, initially these rates were optimised independently to narrow down the optimum range for these rates. This section is divided into three parts. In part (i), the feasible range of optimum mutation rate was determined, assuming a reasonable value for crossover rate. The optimum feasible range of crossover rate was then studied with a reasonable value for the mutation rate and discussed in part (ii). Finally, the combinations of mutation and crossover rates within the ranges obtained in parts (i) and (ii) were studied to determine the optimum mutation and crossover rates, and discussed in part (iii).

(i) Mutation

The effect of mutation rate on the convergence of various water resource optimisation studies have been investigated Mohan (1997), Mulligan and Brown (1998) and Wardlaw and Sharif (1999). In addition, Grefenstette (1986) studied the effect of mutation rate on the performance of five numerical test functions (e.g. Rosenbrock's saddle) which covered discontinuous, multidimensional and noisy functions. He explored eight values of mutation rates obtained from the range of 0.0 to 1.0 with nonlinear increase. His general finding was that any mutation rate greater than 0.1 did not converge to the optimum solution.

Mohan (1997) conducted an investigation on the effect of mutation rate in his Muskingum flood routing model in optimising model parameters. He found that the most optimum mutation rate was 0.001 in reaching the 'optimum' parameter set.

Mulligan and Brown (1998) compared three different mutation schemes in estimating water quality decay rates with GA. These schemes were traditional mutation rate (i.e. constant rate from start to finish of the GA run), sliding mutation rate and a spiked mutation rate. The sliding mutation scheme was used by initially having the mutation rate kept at 0.03 until 5000 simulations (which is one-third of the total number of simulations and 200 generations), and then increasing to 0.09 for the remaining simulations. The spiked mutation rate was done by initially setting the mutation rate set at 0.03 for first 6250 simulations (i.e. 250 generations), and then increasing it to 0.25 for one generation and finally decreasing back to 0.03 for the remaining generations. They found that the sliding mutation resulted in rapid changes in parameters which resulted in an unstable solution. While the spiked mutation scheme did increase diversity into the parameter sets, it did not increase the convergence rate compared to the traditional mutation scheme. As a general finding, Mulligan and Brown (1998) concluded that any mutation scheme other than the traditional did not add any improvement to the convergence rate.

Wardlaw and Sharif (1999) studied the optimum reservoir system operation in their four-reservoir problem. They varied the mutation rate from 0.002 - 0.208 (with a constant crossover rate of 0.7) and found that the mutation rate of 0.03 was robust in achieving the optimum value. Nevertheless, they reported that any value between 0.01 - 0.04 produced reasonable results.

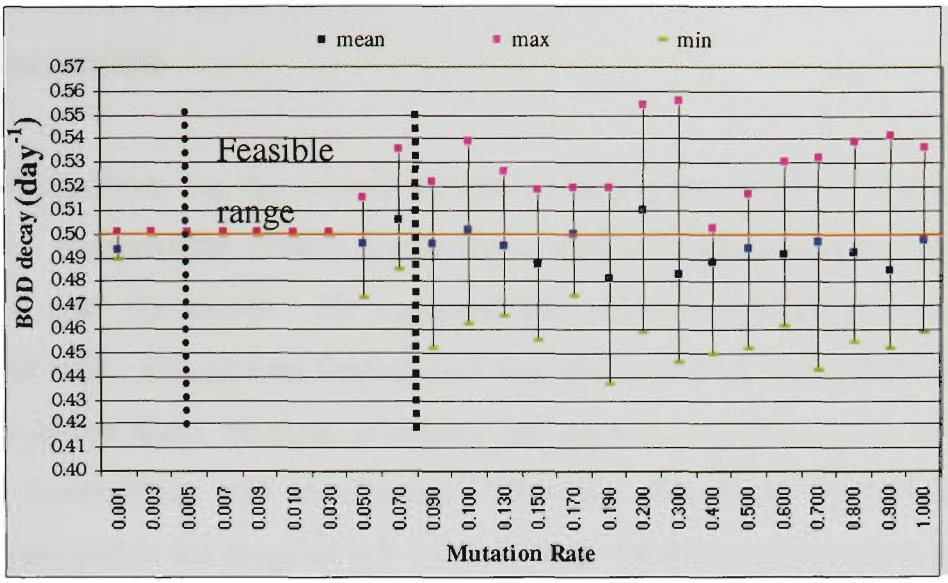
Several other researchers in their water resource studies used a single mutation rate. For example, Wang (1991) used a mutation rate of 0.01 in his rainfall and runoff model for parameter estimation, while Liong *et al.* (1995) used the default value of GENESIS (which is 0.001) to optimise eight SWMM (Huber *et al.,* 1982) model parameters, in predicting the peak flow.

Based on the above studies, it is difficult to borrow suitable values for the mutation rate for the study described in this thesis. Therefore, an experiment was conducted by varying mutation rate from 0.0 to 1.0 (inclusive), as in Grefenstette (1986). Twenty-four different rates within four sub-ranges were selected to cover typical mutation rates considered in the past. They are listed below,

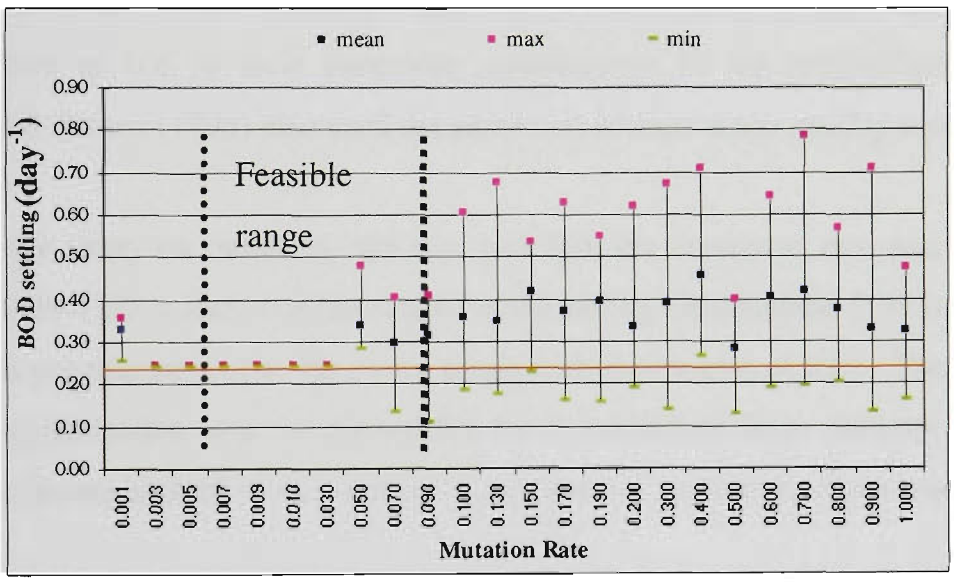
- Sub-range 0.001-0.01 (0.001, 0.003, 0.005, 0.007, 0.009).
- Sub-range 0.01-0.1 (0.01, 0.03, 0.05, 0.07, 0.09).
- Sub-range 0.1-0.2 (0.1, 0.13, 0.15, 0.17, 0.19).
- Sub-range 0.2-1 (0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1.0).

The model parameter optimisation for the hypothetical river network insensitive model was conducted with these 24 mutation rates (taking one at a time) with the crossover rate kept at 0.6 (i.e. 'LIT' set value). This produced 24 GA optimisation runs. Figure 6.11 shows the 'optimum' parameters with respect to the best 10 parameter sets obtained for each of the 24 mutation rates for BOD_d, BOD_s and SOD respectively. The results in this figure are presented in terms of the mean and the range (i.e. maximum and minimum) of the best 10 parameter sets of each GA run. As can be seen clearly, an optimum mutation rate within 0.003-0.03 shows convergence for all three model parameters (i.e. BOD_d, BOD_s and SOD) with a crossover rate of 0.6. Any mutation rate greater than 0.03 and less than 0.003 was clearly shown to be infeasible in converging to the optimum parameter value in the hypothetical river network insensitive model. The range 0.003-0.03 was used in part (iii) of this study.

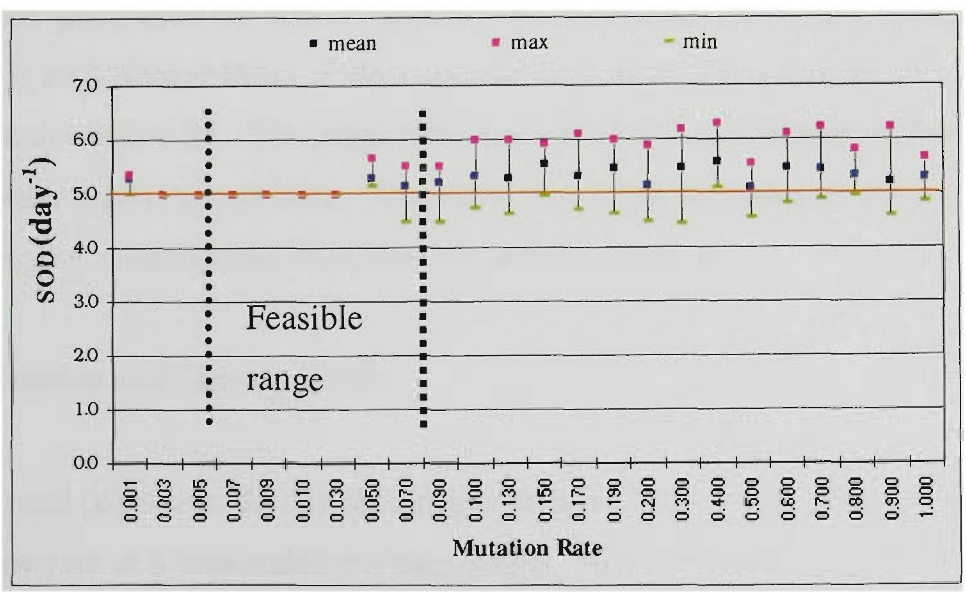
To proceed with the next experiment on the crossover rate, a constant mutation rate of 0.03 was considered. This mutation rate has been successfully used in Mulligan and Brown (1998) and Wardlaw and Sharif (1999), and was also found within the feasible range of this study.



(a) BOD Decay



(b) BOD Settling



(d) SOD

Figure 6.11 Convergence with Different Mutation Rates

(ii) Crossover rate

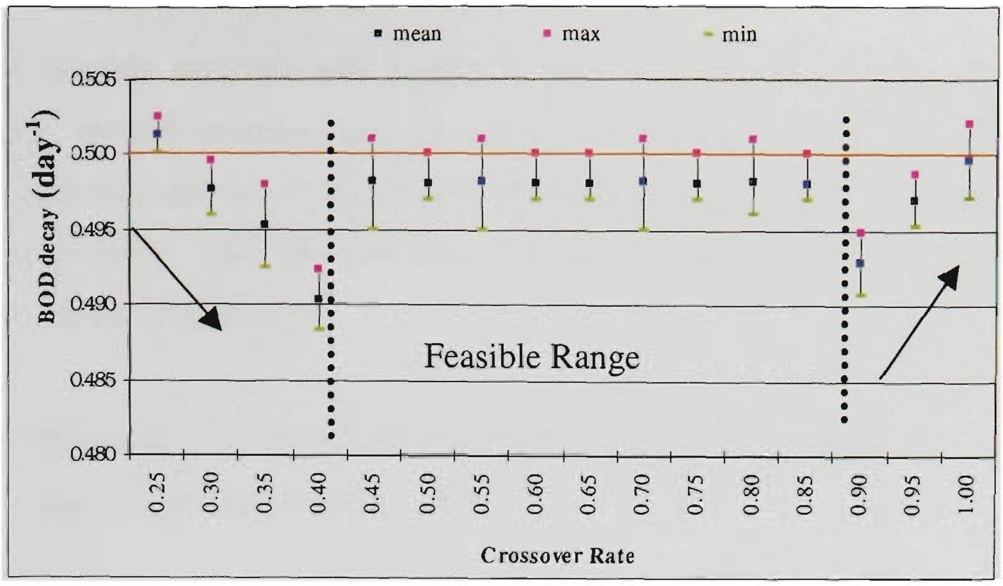
Limited investigations on the crossover rate in GA were found in water resources applications. Grefenstette (1986) conducted a similar investigation to mutation rate discussed in part (i) above. He explored the crossover range from 0.25-1.0 in increments of 0.05. His general finding was that, the crossover rate should be higher in smaller population sizes, because crossover can play an important role in preventing premature convergence, and vice versa. Wardlaw and Sharif (1999) studied the crossover rates within the range of 0.5 - 0.95 in their reservoir optimisation study. They found that the crossover rate of 0.7 reached the optimum solution, while with any rate after 0.7, the fitness values decrease dramatically. Liong *et al.* (1995) used a constant crossover rate of 0.6 in their parameter optimisation of the rainfall/runoff model. Mulligan and Brown (1998) also used the same rate in their water quality model.

Similar to the study on mutation rate (i.e. part (i)), the crossover rate was allowed to vary from 0.25-1.00, which was the range considered by Grefenstette (1986). A total of 16 runs was considered within the above range with increments of 0.05. The GA model parameter optimisation was conducted for the hypothetical river network insensitive model considering each of these crossover rates and the constant mutation rate of 0.03.

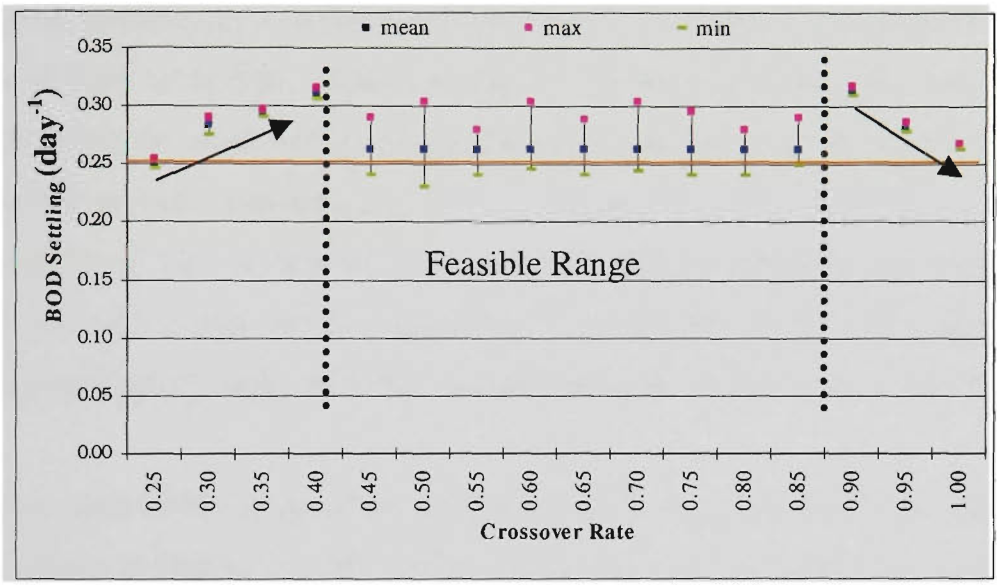
Figure 6.12 shows the mean and the range of the best 10 parameter sets obtained from the final GA generation for each crossover rate for BOD_d, BOD_s and SOD. It appears that there is a consistent trend of non-convergence when the crossover rate is less than 0.4 and greater than 0.85. The range between 0.45-0.85 was considered feasible across all three water quality parameters. Therefore, this range was used in the next section in determining the mutation and crossover rate simultaneously.

(iii) Mutation and Crossover rate

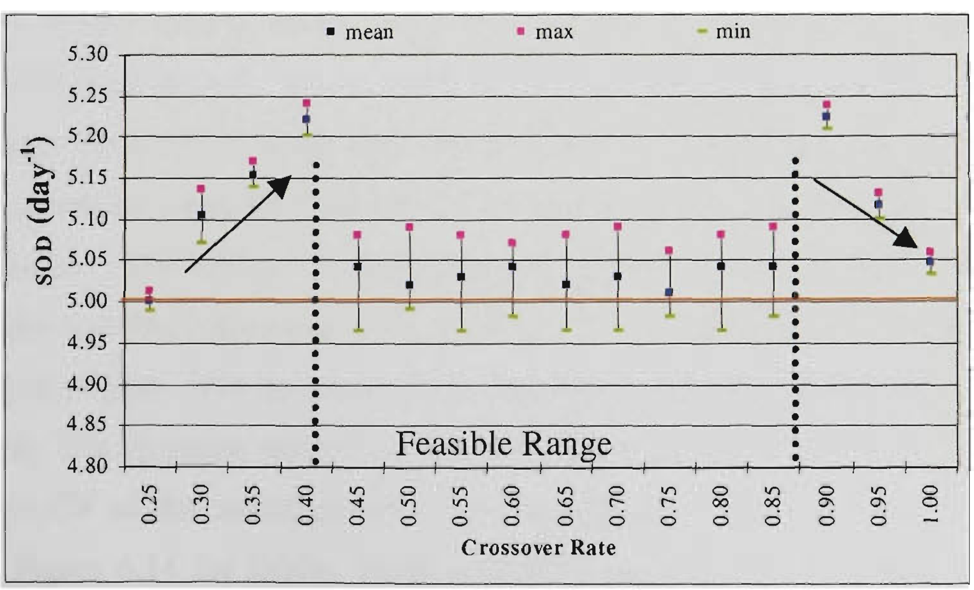
In parts (i) and (ii) above, the feasible mutation and crossover rate ranges were found by keeping one rate at a reasonable constant value. However, the crossover and mutation rates simultaneously determine the rate of convergence to the optimum parameter set.



(a) BOD Decay



(c) BOD Settling



(c) SOD

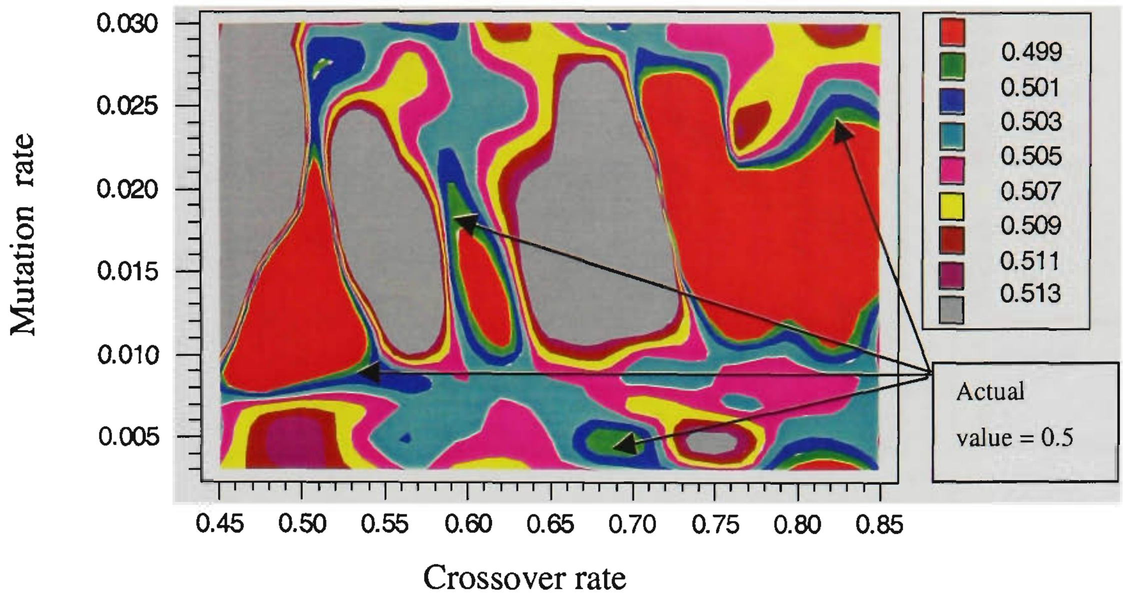
Figure 6.12 Convergence with Different Crossover Rate

Using the feasible mutation and crossover rate ranges found in parts (i) and (ii) respectively, several mutation and crossover combinations were investigated. The crossover rate was allowed to vary from the feasible range of 0.45-0.85 with constant increments of 0.05. The mutation rate was varied from 0.003-0.030 with varying increments as specified below.

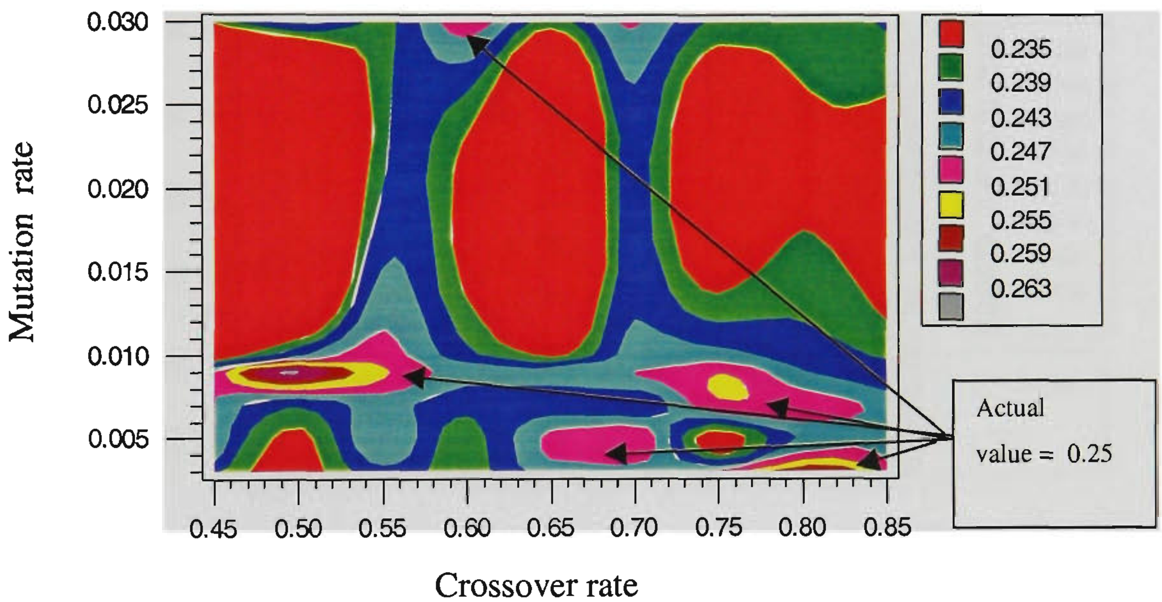
- Sub-range 0.001-0.009 (0.001, 0.003, 0.005, 0.007, 0.009)
- Sub-range 0.01-0.03 (0.01, 0.03)

In total, 63 runs were simulated. The results of the analysis was presented using contour plots of mean and coefficient of variation (CV) of the model parameters. These plots can be used to find the regions where the optimum mutation and crossover rates lie, by observing the mean value close to the actual parameter value and the lowest CV. Similar to the previous experiments, the best 10 parameter sets obtained from the final GA generation of each of the 63 runs were analysed to produce the mean and CV. Although 10 values may not be sufficient for a standard statistical analysis, it was considered satisfactory here, since the variation of these 10 values were fairly minimal.

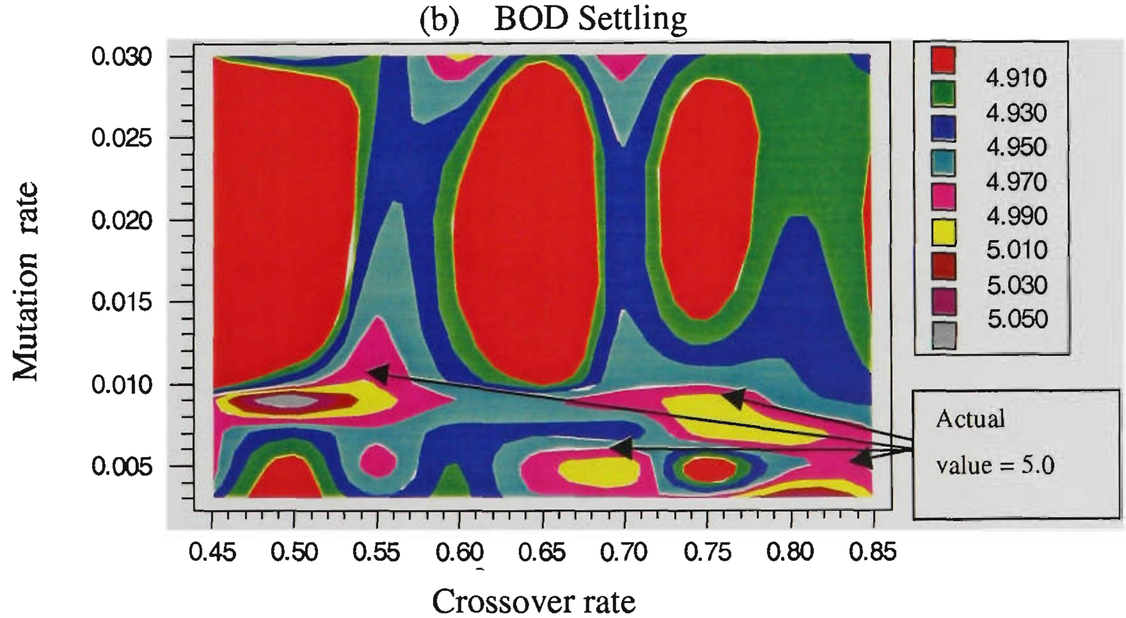
The contour plots of the mean of the parameters with respect to mutation and crossover rates are shown in Figure 6.13 for BOD_d, BOD_s and SOD respectively. In finding the optimum mutation rate and crossover rate, the mean of all three water quality parameters should closely match with their respective actual values for the same mutation and crossover rate. In each plot, a colour coding scheme was used to identify the optimum region for mutation and crossover rates. For example, in Figure 6.13 (a), the actual parameter value for BOD_d is 0.5 (which should also be the optimum value for this parameter). The colour of green representing the value of 0.501, which is the region where optimum value lies. The optimum value for BOD_s is 0.25 and lies in the colour region of pink. The optimum value for SOD is 5.0, and lies in the colour region of also pink. The optimum regions are marked on each plot in Figure 6.13. The contour plots of the CV of the model parameters with respect to crossover and mutation are shown in Figure 6.14 for BOD_d, BOD_s and SOD respectively. The regions with the



(a) BOD Decay

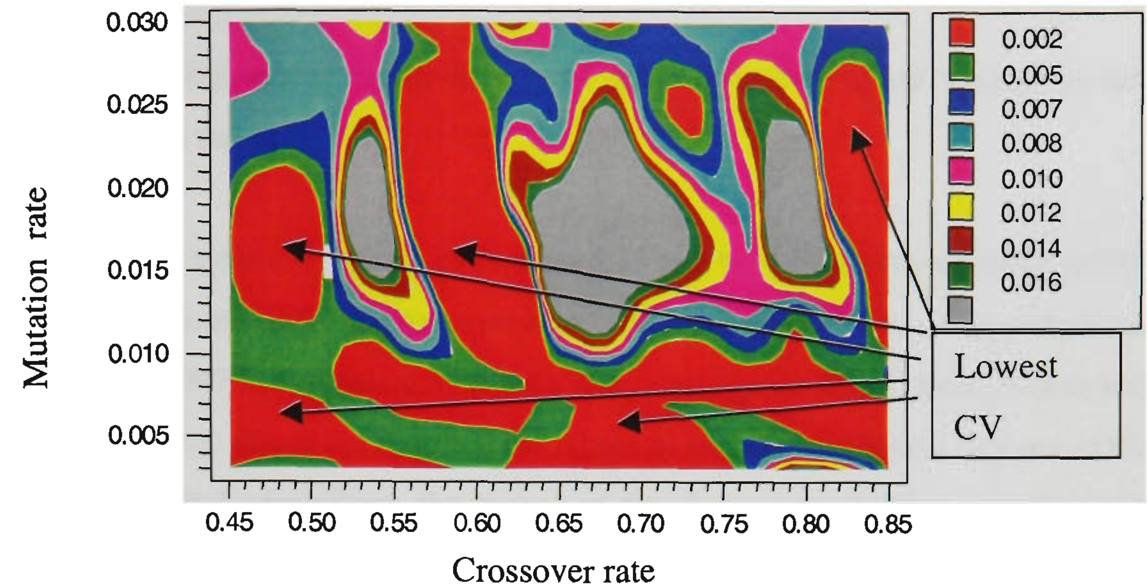


(b) BOD Settling

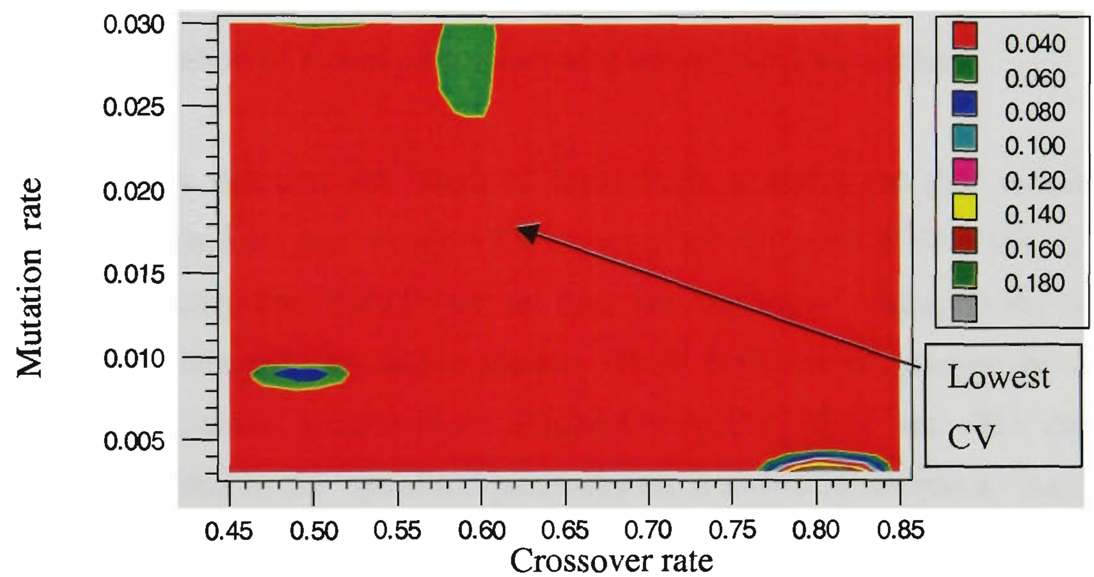


(c) SOD

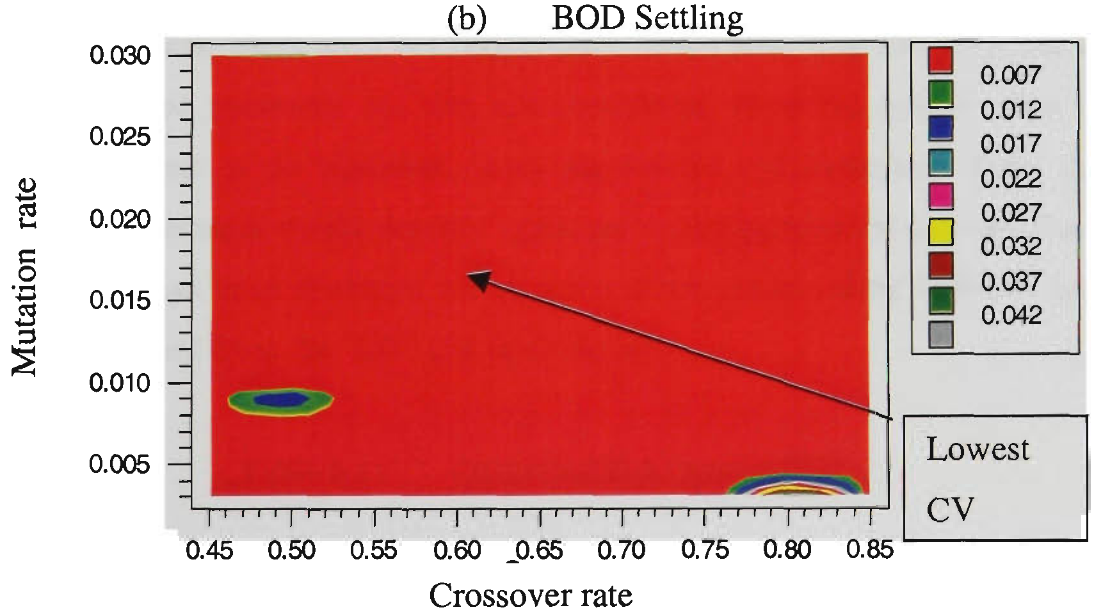
Figure 6.13 Contour Plot of Mean Parameters with Mutation and Crossover Rates



(a) BOD Decay



(b) BOD Settling



(c) SOD

Figure 6.14 Contour Plot of CV with Mutation and Crossover Rates lowest CV (colour region red) for all three water quality parameters are marked on each plot as the optimum region.

By considering the optimum regions in Figures 6.13 and 6.14 for the three water quality decay rates, it was found that the crossover rate between 0.66-0.72 and the mutation rate between 0.0035–0.007 simultaneously satisfy the convergence of all three parameters. Since single optimum values are required for mutation and crossover rates for use in GA optimisation of model parameters, the means of the above ranges were recommended for the hypothetical river network insensitive model, which were mutation rate of 0.005 and crossover rate of 0.69.

(c) GA optimisation of model parameters with optimised GA operators

The optimised GA operator set obtained from Parts a and b of this section are summarised in Table 6.11, and is called 'OPTI' set in this chapter. A final GA run was then conducted using the 'OPTI' set to find the 'optimum' decay rates of the hypothetical river network insensitive model. Table 6.12 shows comparison of the actual and the 'optimised' parameter set obtained using the 'OPTI' set. The absolute difference of the actual to the 'optimum' parameter set is shown in column 4. As can be seen from this table, the model parameters have converged within 1% of the actual values. This is in contrast to the results obtained from 'LIT' GA operator set (Table 6.8), where the parameters were converged only within 8%. The DO response due to the new optimised parameter set was not considered, since the optimisation of GA operators was done as an academic study (as pointed out earlier) to show that the optimised GA operators would produce optimum model parameters through GA. The output response had been predicted within 0.5% of the actual values with the optimum parameters obtained from the 'LIT' GA operator set.

Table 6.11 Optimised GA Operator Set

Population size	Number of Generations	Number of Simulations	Crossover rate	Mutation Rate
125	120	15000	0.69	0.005

Table 6.12 Comparison of the Actual and 'Optimised' Parameter Set Obtained From Optimised GA Operator Set

Decay rates (day^{-1})	Actual parameter set (day^{-1})	'Optimised' (day^{-1})	% absolute difference (Optimised to Actual)
BOD_d	0.500	0.500	0.0
BOD_s	0.250	0.248	0.8
SOD	5.000	5.010	0.2

6.5.1.4 Result from hypothetical river network sensitive model

An approach similar to the one used for the insensitive model was used in the sensitive model. First the 'LIT' GA operator set was used to optimise the model parameters of the hypothetical river network sensitive model. The comparison of the model parameter set and the actual parameter set is shown in Table 6.13.

Table 6.13 Comparison on Actual and 'Optimised' Parameter Set Obtained From 'LIT' GA Operator Set for Hypothetical River Network Sensitive Model

Decay rates (day^{-1})	Actual parameter set (day^{-1})	'Optimised' (day^{-1})	% absolute difference ('Optimised' to Actual)
BOD_d	0.50	0.499	0.20
BOD_s	0.25	0.255	1.96
SOD	5.00	5.060	1.19

As can be seen from Table 6.13, GA with the 'LIT' GA operator set converged to the 'optimum' parameter set in the sensitive model much closer than in the insensitive model. The difference between the actual and the 'optimised' parameter set was less than 2% in the sensitive model, as compared to 8% in the insensitive model.

A QUAL2E run was made for the sensitive model with the 'optimised' decay rate set of Table 6.13. The modelled DO concentration was then compared with the actual DO and the comparison is shown in Table 6.14. The modelled DO using the 'optimised' parameter set was within 1% of the actual values. Since the model parameters were converged according to the pre-set criterion (i.e. 2%) as stated earlier in Section 6.5

with the sensitive model using 'LIT' GA operator set, no further optimisation of GA operators was considered for the sensitive model.

Table 6.14 Comparison of Actual and Modelled DO Concentrations (Sensitive Model)

Reach	DO (mg/L) Using Actual Parameter Set	DO (mg/L) Using Optimised' Parameter Set	% absolute difference
1	7.27	7.26	0.1
2	2.16	2.15	0.5
3	1.92	1.91	0.5
4	0.97	0.97	0.0
5	1.33	1.32	0.8
6	2.34	2.34	0.0

6.5.1.5 Summary of findings from hypothetical river network insensitive and sensitive models

From the analysis from Sections 6.5.1.2 to 6.5.1.4, the following findings were obtained.

- The best 10 parameter sets obtained from the final GA generation can be used to derive the optimum parameter set. This result was found from both hypothetical river network insensitive and sensitive models, and later to be verified with YRWQM in Section 6.5.2.
- Seed did not play a role on the optimum parameter set, as the search space progressively reduced in the way that was done in this study. Three GA runs were able to remove the effect of seed in both hypothetical river network insensitive and sensitive models, and later to be verified with YRWQM in Section 6.5.2.
- The hypothetical river network model with insensitive parameters did not converge to the actual parameter set with the GA operator set obtained from literature.
- The optimised decay parameters obtained from GA with literature GA operator set predicted DO within 0.25% in the insensitive model, although there were upto 8% difference between optimised and actual decay parameter values. The close prediction of DO was due to parameter insensitivity on DO.

- An optimised GA operator set was able to converge decay parameters in the insensitive model within 1%.
- The literature GA operator set was able to converge the decay parameters in the hypothetical river network sensitive model within 2% of the actual decay parameter values.

6.5.2 YRWQM river network

A similar approach to the hypothetical river network model (Section 6.5.1) was conducted on YRWQM to verify the findings obtained from the hypothetical river network models in relation to the use of GA operators First, the number of parameter sets taken that should be considered in the final GA generation to derive the optimum parameter set and the effect of seed were investigated using YRWQM. YRWQM was found to be an insensitive model based on the uncertainty/sensitivity analysis in Chapter 5. Therefore, it was interesting to find out whether the findings obtained from the hypothetical river network insensitive model were the same for YRWQM. The procedures carried out were as follows. The 'LIT' GA operator set was first used with YRWQM. Then, the 'OPTI' set obtained from the hypothetical river network insensitive model was used, without carrying out an optimisation of GA operators. The reason for this was unlike in the hypothetical river network model, the parameters of YRWQM were not known with certainty and therefore an optimisation could not be effectively conducted. At the same time, since YRWQM was an insensitive model, it was not necessary to do an optimisation of GA operators based on the findings of the hypothetical river network insensitive model. However, it was interesting to see the difference between 'optimised' model parameter sets obtained from 'LIT' and 'OPTI' GA operator sets. These GA operator sets are summarised in Table 6.15.

Table 6.15 'LIT' and 'OPTI' GA Operators Sets Used for YRWQM

GA Operator Set	Population size	Number of generations	Number of Simulations	Crossover rate	Mutation rate
'LIT' Set	125	40	5000	0.60	0.030
'OPTI' Set	125	190	15000	0.69	0.005

Two flow events, namely, 18/3/92 and 18/2/92 were arbitrarily selected from the flow events described in Section 6.2 to undertake this experiment. The water quality responses of TKN, TN, TP and DO were considered in model parameter estimation. The relevant decay rates and their ranges (i.e. search space used for GA calibration) were used in this experiment are shown in Table 6.16. As can be seen from Table 6.16, 3 GA runs (with their progressively reduced search spaces) were used in this experiment to eliminate the effect of seed. The parameter ranges for the first GA run was determined in Section 4.4.3. In YRWQM, both nitrogen and phosphorus decay rates were also considered in addition to CBOD and SOD, unlike in the hypothetical river network models. Therefore, this investigation requires to be conducted in a sequential manner as discussed in Section 6.3.1, where decay rates affecting TKN were estimated first, then TN and TP (in parallel), and finally DO. The objective function used in the YRWQM - GENESIS was the same (i.e. simple least squares) as for the hypothetical river network.

Table 6.16 Progressive Reduced Search Space (day^{-1}) for YRWQM

Decay rates	GA run 1 (per day)	GA run 2 (per day)	GA run 3 (per day)
Org-N_d	0.006 - 0.420	0.006 - 0.418	0.23 - 0.400
Org-N_s	0.001 - 2.630	0.001 - 0.315	0.001 - 0.315
NH_{3-d}	0.001 - 0.720	0.001 - 0.099	0.001 - 0.099
NH_3ben	0.001 - 1.800	0.022 - 1.782	0.7 - 1.782
NO_{2-d}	0.001 - 0.700	0.004 - 0.7	0.25 - 0.49
Org-P_d	0.001 - 1.000	0.001 - 0.783	0.09 - 0.57
Org-P_s	0.000 - 0.140	0.014 - 0.140	0.08 - 0.14
Diss-Pben	0.001 - 1.700	0.014 - 1.645	0.11 - 1.1
$CBOD_d$	0.0042 - 3.500	0.004 - 2.741	0.004 - 0.5
$CBOD_s$	0.001 - 1.530	0.122 - 1.530	0.21 - 0.67
SOD	0.000-2.000	0-0.057	0-0.033

(a) Number of parameter sets taken from the final GA generation and the effect of seed

The study on the number of parameter sets taken from the final GA generation showed that similar results to those found with the hypothetical river network models were obtained from YRWQM. The results have indicated that the slope of the graph (similar to Figure 6.6) between the objective function and the number of parameters taken from the final generation increased. Furthermore, the slope cut-off point was around 10 parameter sets as in the hypothetical river network insensitive model. Therefore, the mean of the 10 best parameter sets was adopted as the optimum parameter set for YRWQM.

Similar to hypothetical river network models, the effect of seed was also found to be negligible as the parameter search space was reduced successively in 3 GA runs. The reduced ranges used for GA runs 2 and 3 were obtained from the minimum and maximum parameter values (of the 125 parameter values) in the last generation of previous GA run with respect to GA optimisation of model parameters using 'LIT' and 'OPTI' GA operator sets and two flow events. For subsequent studies described in this section, and calibration described in Section 6.6, the reduced parameter search space obtained from GA run 3 was used, and only one GA optimisation run was considered.

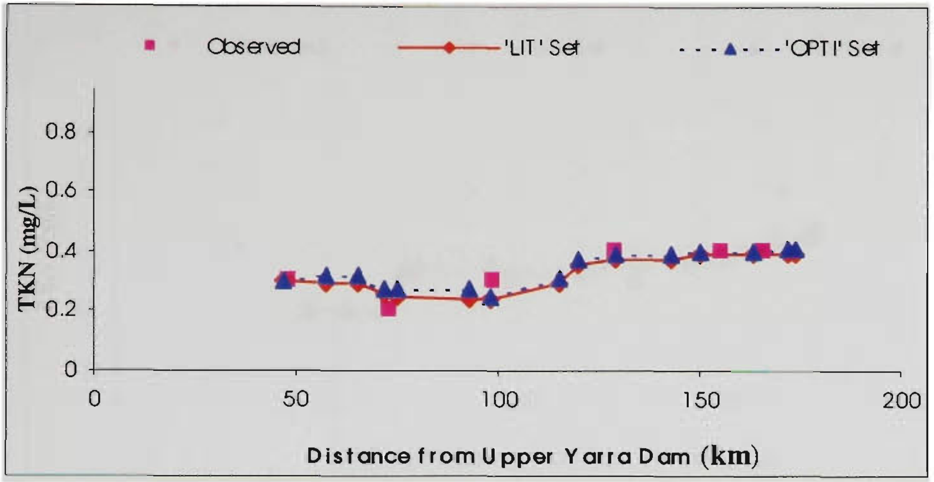
(b) Effect of GA operators on YRWQM

The GA model parameter estimation was conducted for the two cases of GA operator sets using the two flow events and the reduced parameter search space. For each GA model parameter optimisation run, the mean decay rates were determined from the best 10 parameter sets of the last generation. They are presented in Table 6.17. Decay rates 'optimised' through both sets of GA operators are shown in Table 6.17. In general, the objective function values determined from the 'OPTI' set was slightly lower than these obtained from the 'LIT' set for both flow events, although there was some difference in the decay rates values. This was to be expected in an insensitive model, as also found with hypothetical river network insensitive model.

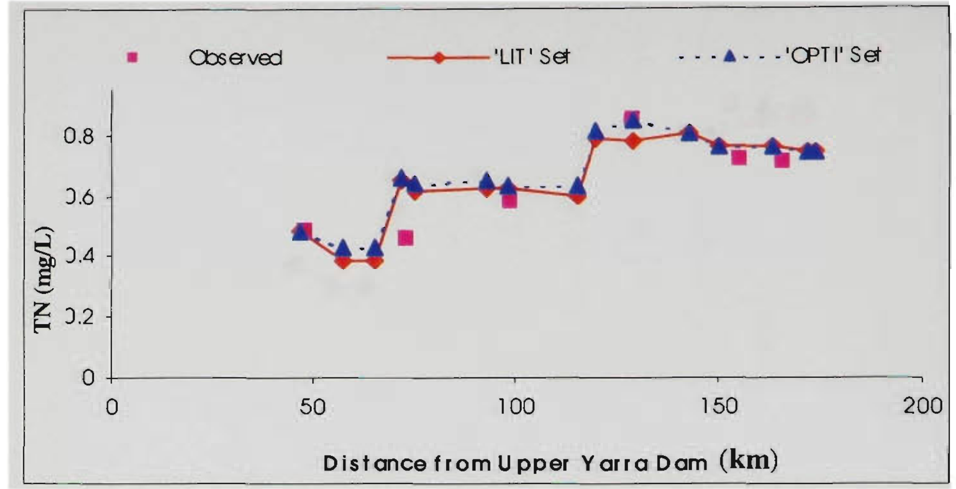
These two GA 'optimised' decay rates sets were then used in YRWQM and the output water quality were compared with the observations at the 6 EPAWQ measurement points. Figures 6.15 and 6.16 show comparisons of the water quality responses of TKN, TN, TP and DO respectively for the events 18/2/92 and 18/3/92. Each plot shows the observed and modelled water quality concentration from Upper Yarra to Lower Yarra. Both decay rate sets were able to match the observations with reasonable accuracy, and no statistical difference was found between these water quality responses at 95% significant level. This is mainly because of the insensitivity of model parameters to output responses, as also found with the hypothetical river network insensitive model. Considering negligible differences in the output water quality from the 'LIT' and 'OPTI' GA operator sets, the 'LIT' set was selected for the YRWQM parameter estimation in Section 6.6. This study on YRWQM effectively verified the findings of the hypothetical river network models.

Table 6.17 Comparison on 'Optimised' Decay Rates with Different GA Operator Sets

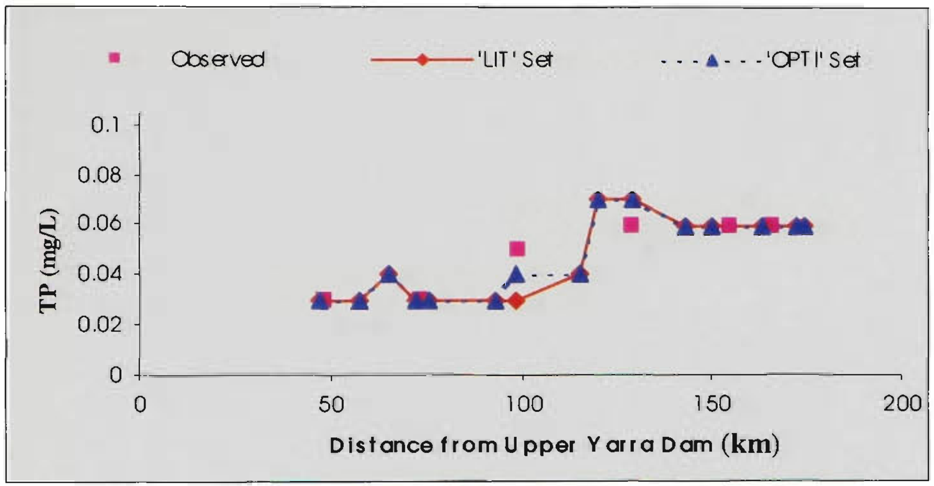
	18/2/92			18/3/92		
Runs	'LIT'	'OPTI'	% difference	'LIT'	'OPTI'	% difference
Org-N_d (day^{-1})	0.211	0.32	51.6	0.329	0.391	18.8
Org-N_s (day^{-1})	0.024	0.029	20.8	0.048	0.065	35.4
$NH_{3\text{-}d}$ (day^{-1})	0.019	0.02	5.2	0.016	0.02	25.0
NH_3ben (day^{-1})	0.751	0.887	18.1	1.526	1.61	5.5
Objective Function Value (mg/L^2)	0.0066	0.0075	13.6	0.0313	0.0164	47.6
$NO_{2\text{-}d}$ (day^{-1})	0.303	0.356	17.5	0.525	0.489	6.86
Objective Function Value (mg/L^2)	0.0466	0.026	44.2	0.0599	0.0413	31
Org-P_d (day^{-1})	0.177	0.188	6.2	0.366	0.405	10.6
Org-P_s (day^{-1})	0.136	0.108	20.6	0.128	0.127	0.78
Diss-Pben (day^{-1})	0.326	0.405	24.2	0.924	0.887	4.17
Objective Function Value (mg/L^2)	0.0003	0.0002	33	0.0007	0.0002	71.4
$CBOD_d$ (day^{-1})	0.004	0.006	50	0.012	0.008	33.3
$CBOD_s$ (day^{-1})	0.326	0.387	18.7	0.342	0.358	4.7
SOD (day^{-1})	0.003	0.004	33.3	0.001	0	0
Objective Function Value (mg/L^2)	4.12	4.270	3.6	0.172	0.068	0.6



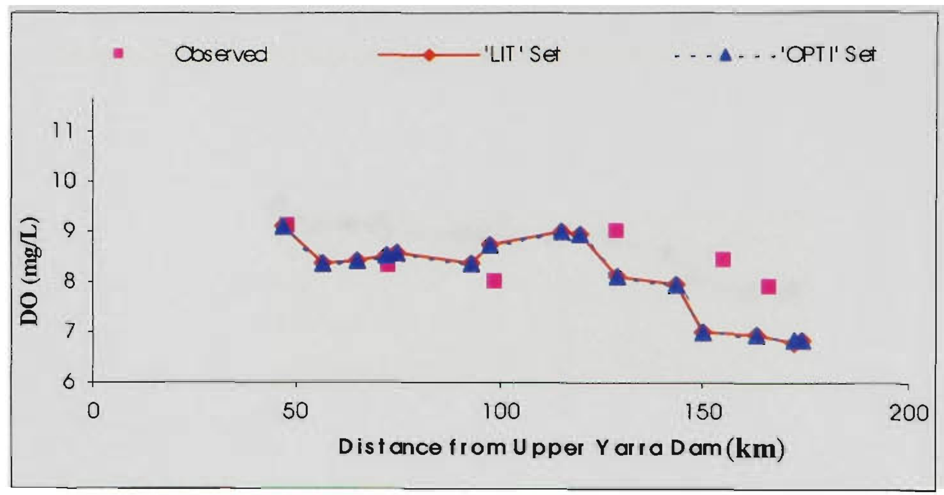
(a) TKN



(b) TN

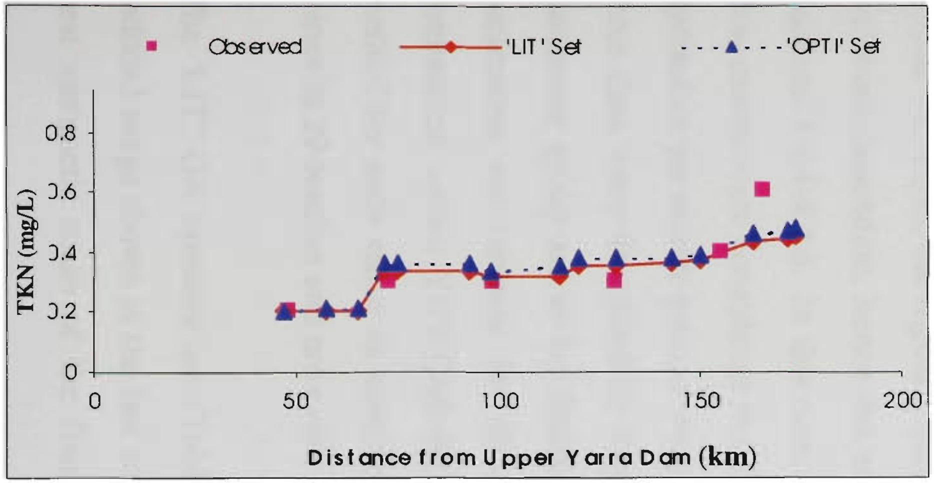


(c) TP

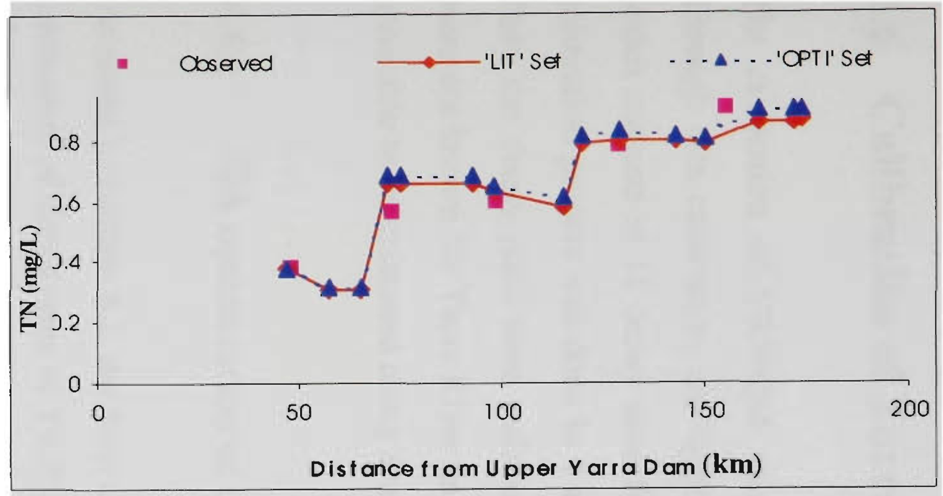


(d) DO

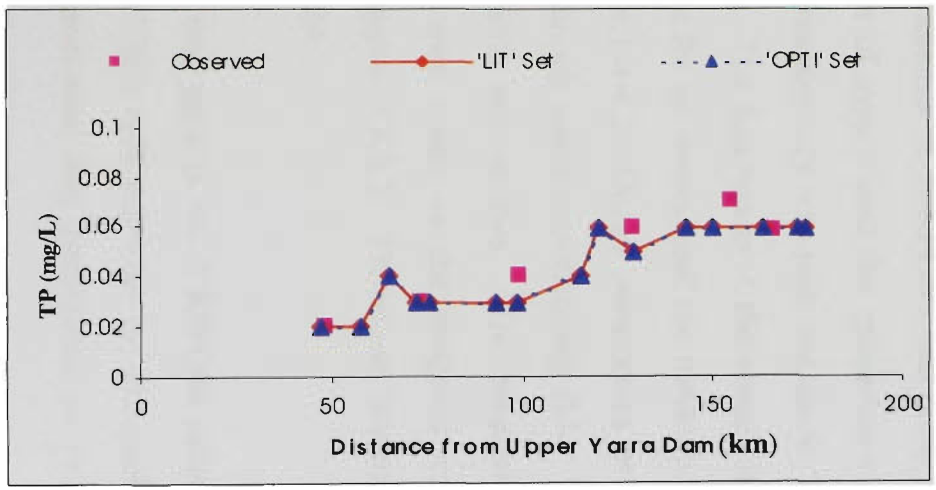
Figure 6.15 YRWQM Water Quality Response Due to Decay Rates Obtained From Different GA Operator Sets: Flow Event 18/2/92



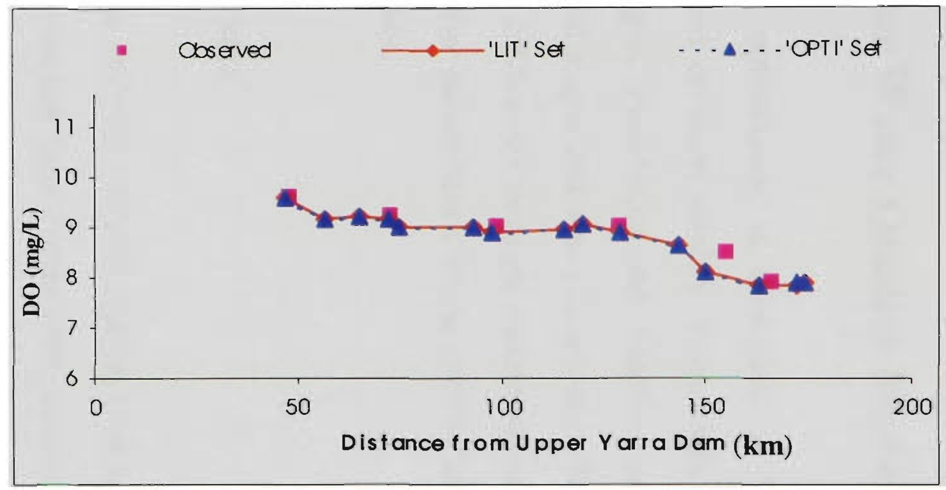
(a) TKN



(b) TN



(c) TP



(d) DO

Figure 6.16 YRWQM Water Quality Response Due to Decay Rates Obtained From Different GA Operator Sets: Flow Event 18/3/92

6.6 Calibration of Yarra River Water Quality Model

The calibration of YRWQM using GA optimisation is discussed in this section. Through this calibration, an 'optimised' set of water quality decay rates was found, which consists of 11 decay rates for nitrogen, phosphorus and dissolved oxygen. The calibration process was done in 4 sequential stages and was discussed in Section 6.3.1. Only the decay rates were estimated (or calibrated) through optimisation, since they were not known for Yarra River and the other parameters such as hydraulics parameters were able to be estimated using available data.

6.6.1 GA optimisation of decay rates

As stated in Section 6.2, the flow events on 18/2/92, 18/3/92 and 3/11/95 were used for calibration of decay rates of YRWQM. These low flow events were used to remove the effect of nonpoint sources, hence the only pollution source considered in this modelling exercise was the effluent discharge from STPs. Of the six parameter groups (Section 4.4) in YRWQM, the inputs required for incremental, headwater and point loads groups are event dependent, hence the estimation of inputs used the procedures described in Sections 4.4.4-4.4.6. In this case, no incremental data was required since 'lower range' flow events were considered in calibration. The data inputs of the remaining global and hydraulics parameter groups were constant for all events, and the methods in obtaining these data were described in Sections 4.4.1-4.4.2. Decay parameters in the reaction parameter group are to be determined through optimisation using GA. These decay parameters are constant for all flow events and reaches. The reaeration rate was determined within YRWQM during run time based on the assigned reaeration rate method for each reach as described in Section 4.4.3.2. Therefore, the reaeration rate varies in 29 reaches and are event dependent.

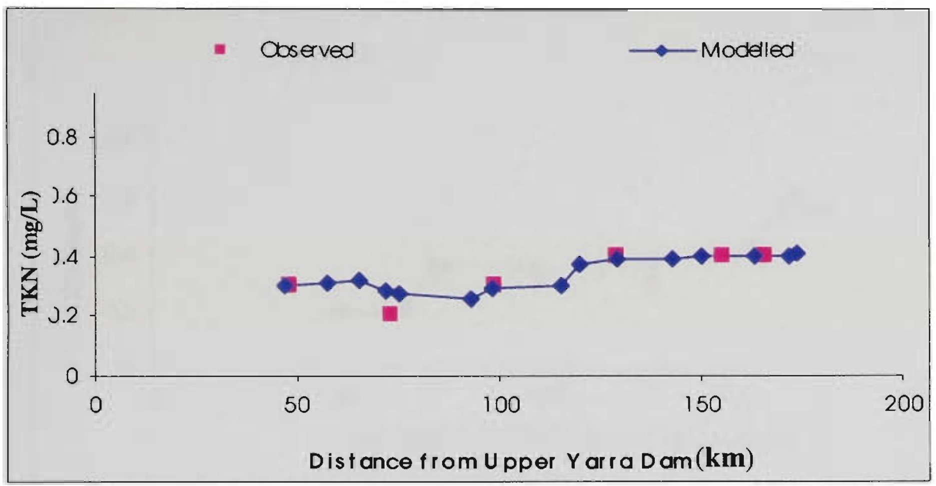
The 'LIT' GA operator set (Table 6.16) was used in the YRWQM calibration. The reduced range shown in the last column in Table 6.16 was used for this calibration. The best arithmetic mean of the final GA generation was considered as the 'optimised' parameter set for each flow event. As explained in Section 6.5.2 (a), the effect of the seed was eliminated by the use of reduced ranges of the parameters of Table 6.16.

The 'optimised' decay rates after 5000 simulations using the reduced search space (of GA run 3 of Table 6.16) for the three events are shown in Table 6.18. The optimised parameter sets from the respective events are shown in columns 2-4. The arithmetic mean of the parameter sets obtained from the 3 events are shown in column 5, as one of the 'optimised' parameter set for YRWQM. The parameter ranges published in literature as given by Bowie *et al.* (1985) and Brown and Barnwell (1987) are given in Table 6.18. In general, the objective function value was fairly low except for DO. This was expected because the error in the objective function propagates from TKN to TN, then to DO. As can be seen from Table 6.18, the 'optimised' decay rates are within the literature range, although there is some difference between the 'optimised' parameter sets obtained from the three flow events. This was also expected because of the insensitivity of model parameters.

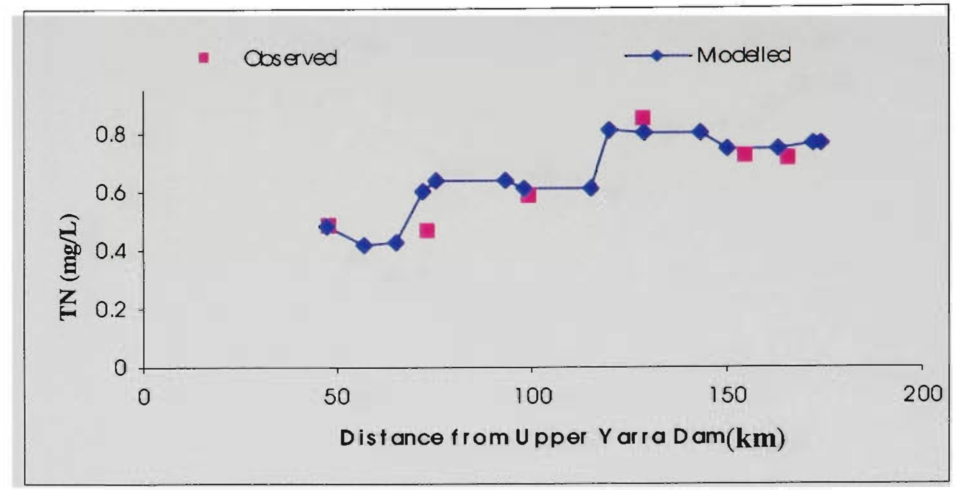
Figures 6.17 - 6.19 show the comparisons of observed and modelled water quality concentrations from Upper Yarra to Lower Yarra for events 18/2/92, 18/3/93 and 3/11/95 respectively. As can be seen from these figures, there is a good match between modelled and observed water quality. Note the fine scale used for TP compared to the other water quality constituents, which explains the relatively larger visual difference in TP. The student t–test also indicated that the observed and modelled water quality concentrations were indistinguishable at 95% significant level for all flow events. As compared with other calibration results, such as Wesolowski (1994), Ghosh and McBean (1998), and Mulligan and Brown (1998), it was generally found that the trend of the water quality prediction can be well achieved. Common to all these studies and YRWQM calibration was the relatively poor mismatch of DO concentration with observations. This could be perhaps due to cascade process in the estimation of decay rates which effects DO, in which errors propagate from one process to another. In other words, the errors of parameters responsible for TKN/TN/TP are propagated to DO prediction.

Table 6.18 'Optimised' Decay Rates From Calibration of YRWQM

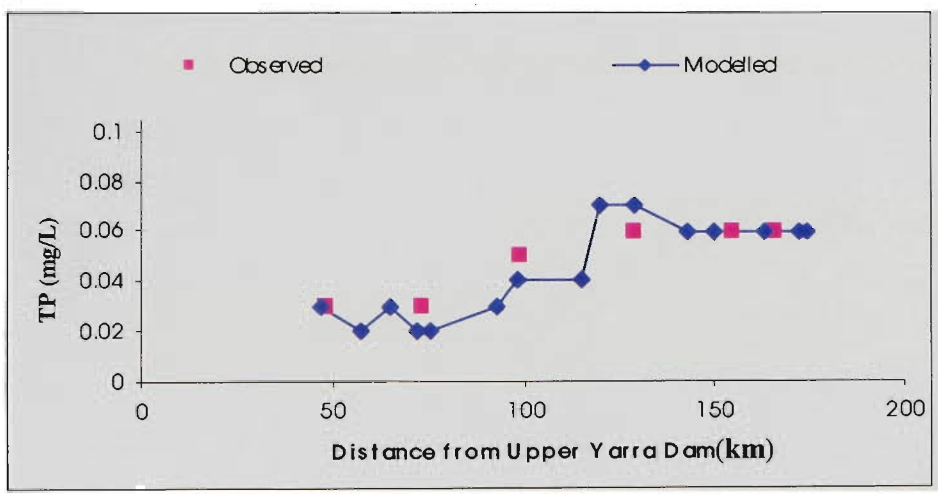
Decay rates	'Optimised' values				Literature values (per day)	Parameters (or additional parameters) responsible for
	Event 18/2/92	Event 18/3/92	Event 3/11/95	Mean		
Org-N_d (day^{-1})	0.211	0.329	0.382	0.307	0.001-0.4	
Org-N_s (day^{-1})	0.024	0.048	0.176	0.083	0.001-0.1	
NH_{3-d} (day^{-1})	0.019	0.016	0.007	0.014	0.003-15.8	
NH_3ben (day^{-1})	0.751	1.526	0.925	1.067	0.004-1.8	
Objective Function value (mg/L^2)	0.0066	0.0313	0.076			TKN
NO_{2-d} (day^{-1})	0.303	0.525	0.596	0.475	0.001-0.7	
Objective Function value (mg/L^2)	0.0466	0.0599	0.08			TN
Org-P_d (day^{-1})	0.177	0.366	0.387	0.310	0.01-0.7	
Org-P_s (day^{-1})	0.136	0.128	0.134	0.133	0.001-0.2	
Diss-Pben (day^{-1})	0.326	0.924	0.271	0.507	0.0004-1.7	
Objective Function value (mg/L^2)	0.0003	0.0007	0.0002			TP
$CBOD_d$ (day^{-1})	0.004	0.012	0.017	0.011	0.004-5.6	
$CBOD_s$ (day^{-1})	0.326	0.342	0.483	0.484	-0.36-0.36	
SOD (day^{-1})	0.003	0.001	0.007	0.004	0.022-44	
Objective Function value (mg/L^2)	4.12	0.172	1.201			DO



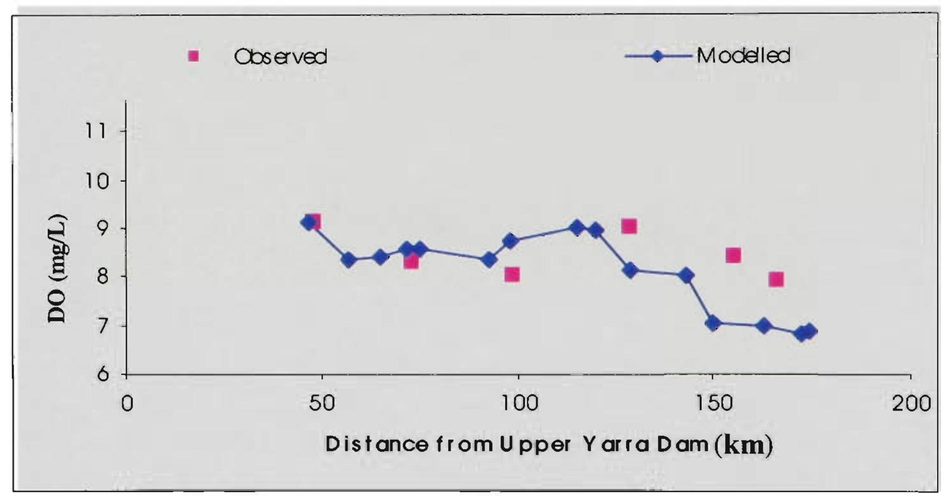
(a) TKN



(b) TN



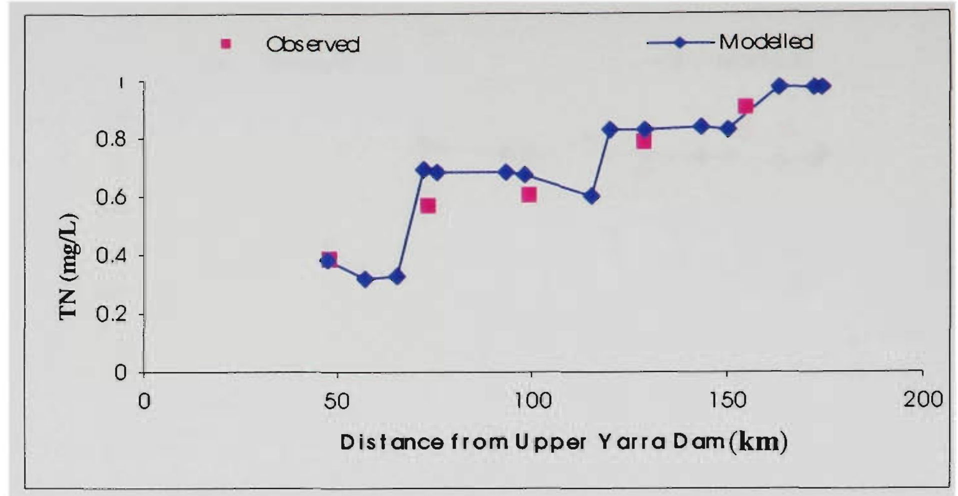
(c) TP



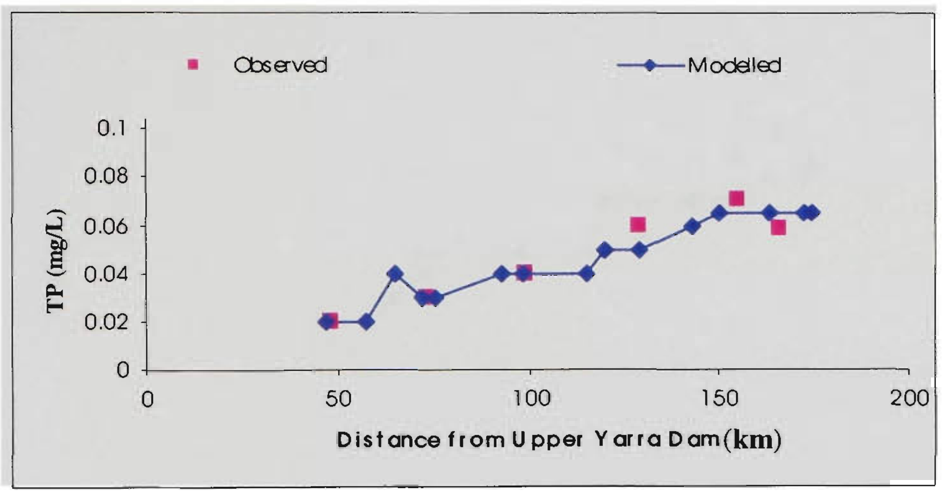
(d) DO

Figure 6.17 Observed and Modelled Water Quality for Event 18/2/92 (Calibration)

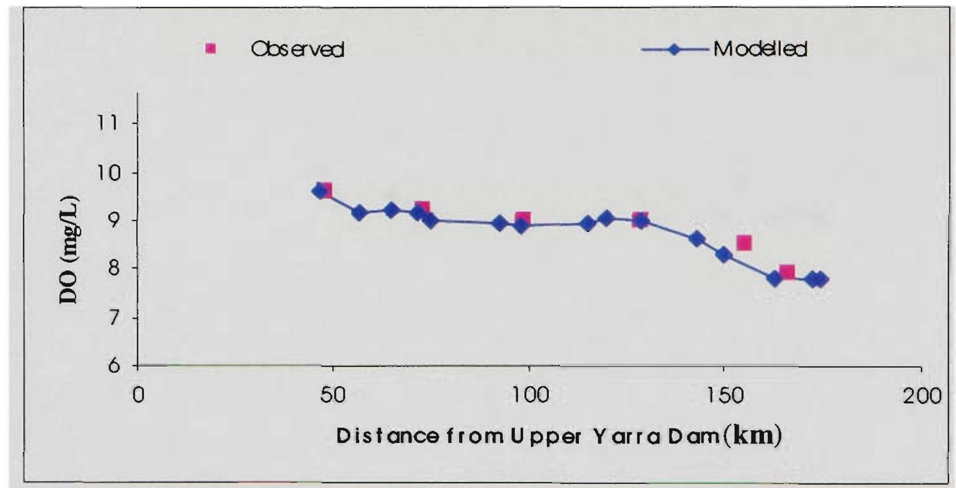
(a) TKN



(b) TN

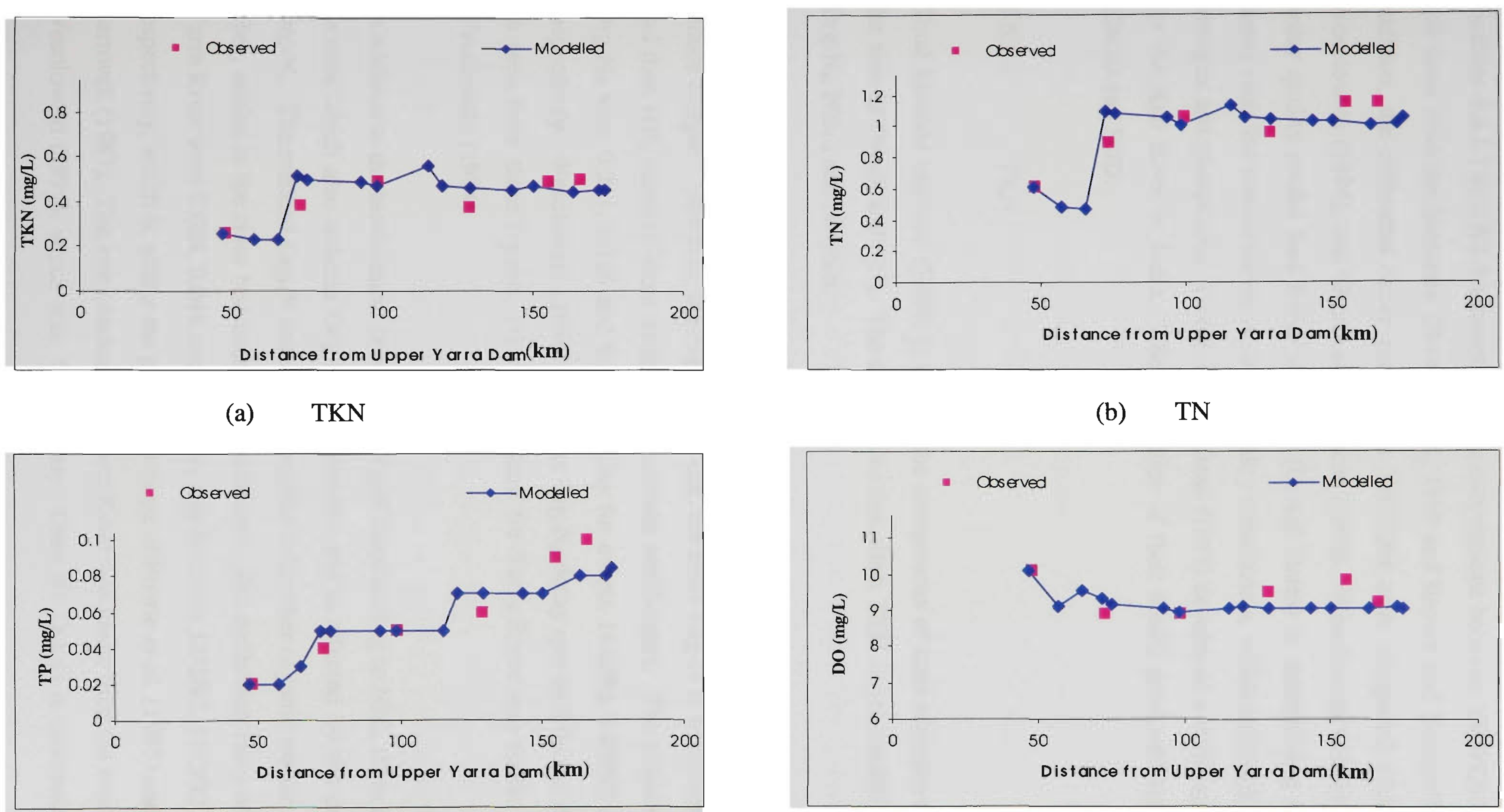


(c) TP



(d) DO

Figure 6.18 Observed and Modelled Water Quality for Event 18/3/92 (Calibration)



(a) TKN

(b) TN

(c) TP

(d) DO

Figure 6.19 Observed and Modelled Water Quality for Event 3/11/95 (Calibration)

Sections 6.6.1.1 to 6.6.1.4 attempt to make comparisons between YRWQM decay rates and rates from the literature (Bowie *et al.,* 1985 and Brown and Barnwell, 1987). In addition, the calibrated decay rates from YRWQM were compared with studies by Wesolowski (1994), and Ghosh and McBean (1998). Wesolowski (1994) calibrated a water quality model Red River at Fargo (United States) in determining a number of decay rates for nonconservative water quality constituents, which included all forms of nitrogen and phosphorus. Ghosh and McBean (1998) developed a water quality model for the Kali River in India. The calibration of their model produced decay rates of CBOD and SOD.

6.6.1.1 TKN

Total kjeldahl nitrogen (TKN) is one of the components of total nitrogen (TN), and is the sum of Org-N and NH_3. The decay rates that affect TKN concentration are Org-N_d, Org-N_s, $NH_{3\text{-}d}$ and NH_3ben.

Org-N_d represents the rate of transformation Org-N to NH_3. This process does not reduce oxygen. However, the higher this rate, the more Org-N is transformed to NH_3, and then NH_3 requires more oxygen to undertake nitrification. The calibrated rates of Org-N_d were 0.211, 0.329 and 0.382 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively. Wesolowski (1994) found the Org-N_d decay rate as 0.04 per day. As can be seen from these figures, the rate obtained for Yarra River was much higher than Wesolowski (1994).

In addition to the biochemical process of Org-N transforming to NH_3, there is a physical process which also reduces Org-N concentration and is affected by the decay rate of Org-N_s. The reduced Org-N substances together with other organic matter such as leaf litter, settles to the river bed under low velocities. The calibrated rates of Org-N_s for Yarra River were 0.024, 0.048 and 0.076 per day for event 18/2/92, 18/3/92 and 3/11/95 respectively, which is within the published range of Bowie *et al.* (1985) and Brown and Barnwell (1987). The rate obtained for Yarra River was lower than the rate obtained by Wesolowski (1994), which was 0.1 per day. Once the Org-N is converted to NH_3, it then undergoes the first part of the nitrification process to degrade to NO_2. The NH_{3-d} decay rate is the rate responsible for this first stage of nitrification. Bowie *et al.* (1985) stated that this rate was highly variable, as shown with the literature range of 0.003-15.8 per day in Table 6.18. The NH_{3-d} rates obtained for Yarra River were 0.019, 0.016 and 0.007 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively, which were comparably less than most studies, such as 1.07 per day in the study by Wesolowski (1994). This suggested that nitrification activity in water column is slower in Yarra River.

Apart from the nitrification in the water column, nitrification can also take place at the bottom of the river, which is influenced by NH_3ben rate. High NH_3ben rates of 0.751, 1.526 and 0.925 per day were found for Yarra River for events 18/2/92, 18/3/92 and 3/11/95 respectively. These values are far greater than those of NH_{3-d}, and agrees with the comments made by Bowie *et al.* (1985) and Williams and Lewis (1986). They stated that the rate of nitrification process in benthos could be far greater than in the water column as nitrifying bacteria populations are two to three times greater in the river bed than in the water column.

6.6.1.2 TN

Total nitrogen (TN) concentration is made up of TKN, NO_2 and NO_3 concentrations. As discussed in Section 6.6.1.1, the product of NH_3 nitrification is NO_2. The decaying process from NO_2 to NO_3 is the second stage of nitrification process and is influenced by the NO_{2-d} rate. This transformation process is generally unstable under aerobic condition, hence the rate is generally high (Dojlido and Best, 1993). The rates obtained for YRWQM were 0.303, 0.525 and 0.596 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively. Wesolowski (1994) found a high rate of 3.08 per day in his Red River at Fargo (United States) water quality modelling study.

6.6.1.3 TP

The total phosphorus (TP) consists of Org-P and Diss-P. The decay rates that affect overall TP concentration are Org-P_d, Org-P_s and Diss-Pben.

Org-P is first transformed to Diss-P and the rate of transformation is determined by the Org-P_d rate. The Org-P_d rates obtained for Yarra River were 0.177, 0.366 and 0.387 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively. The Org-P_d rate of 0.21 per day was found in Wesolowski (1994).

Org-P can also be reduced through settling, and the rate of reduction is determined by the Org-P_s (settling) rate. The Org-P from the water column can subsequently be sorbed into soil particles in suspension and settles in the river bottom (Bowie *et al.*, 1985 and USEPA, 1997b). The Org-P_s rates obtained for YRWQM were 0.136, 0.128 and 0.134 per day for events event 18/2/92, 18/3/92 and 3/11/95 respectively.

Once the Org-P is transformed to dissolved P, it can get further reduced in the river bed to stabilise organic matter. This reduction is influenced by the Diss-Pben rate. The rates obtained for YRWQM were 0.326, 0.924 and 0.271 for events 18/2/92, 18/3/92 and 3/11/95 respectively.

6.6.1.4 DO

DO is considered as one of the most important river health indicator in the river system and is affected by TKN, TN and TP concentrations. Therefore, all parameters in Table 6.18 affect the DO concentration. The additional parameters (other than those responsible for TKN/TN/TP) that affect DO are discussed below.

The decrease in CBOD concentration can be through biochemical decaying via the rate $CBOD_d$. The $CBOD_d$ rates obtained for Yarra River were 0.004, 0.012 and 0.017 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively, which were in the lower range of the literature range. This indicates that the CBOD reduction through biochemical decay is slow, which is expected since all STPs use tertiary effluent treatment, where the stabilisation of waste is greater (USEPA, 1997b and Lung and Sobeck, 1999). This low rate also indicates progressive resistance of further breakdown of organic matter, since high treated waste contains a large proportion of refractory organisms (Bowie *et al.*, 1985 and USEPA, 1997b). Low calibrated $CBOD_d$ rate is quite common in studies cited in the literature. For example, NCASI (1982) calibrated a low rate of 0.02 per day in their Ouachita River (in United States) water quality model. Mulligan and Brown (1998) used a reach varying rate and found most $CBOD_d$ rates were approximately 0.02 per day. A high rate for $CBOD_d$ should be expected with low level of treatment as experienced in the study by Ghosh and McBean (1998), where they found the $CBOD_d$ range to be from 0.1-0.85 per day.

The CBOD concentration can also be reduced through settling to the river bed, and is governed by the settling rate (i.e. $CBOD_s$). This rate should also be low for Yarra River as suspended solids in wastewater is low and waste has been stabilised under tertiary treatment (USEPA, 1997b). The $CBOD_s$ rates obtained for Yarra River were 0.06, 0.023 and 0.081 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively, which are fairly low.

The overall oxygen demand for the breakdown of organic matter in the river bed is affected by the sediment oxygen demand rate (i.e. SOD). The SOD rate obtained for Yarra River were 0.003, 0.001 and 0.007 per day for events 18/2/92, 18/3/92 and 3/11/95 respectively. In contrast, the SOD rate obtained by Ghosh and McBean (1998) ranged from 0.5-7.5 per day in different reaches. As stated in Brown and Barnwell (1987), the SOD rate is considered to be highly variable.

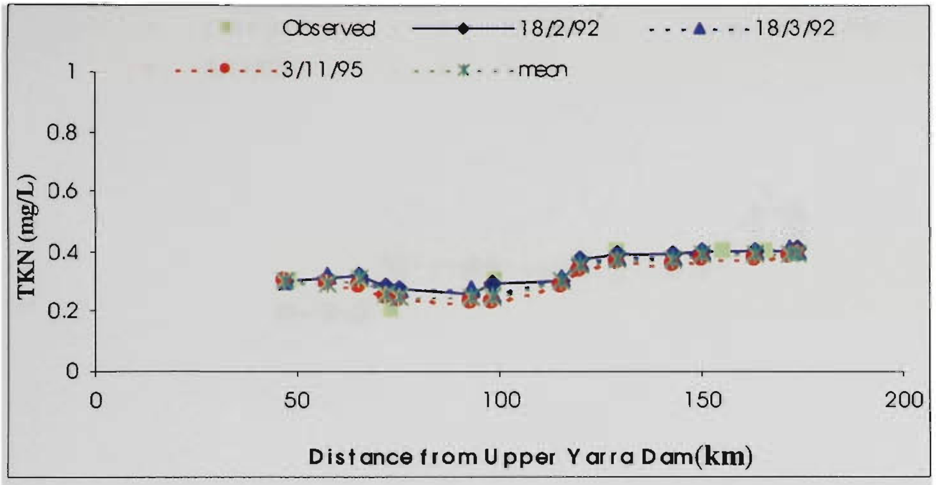
The overall prediction of DO concentrations based on the 'optimised' decay rates were less than the observed values, especially in the lower reaches. However, the trend for DO concentration was matched well between modelled and observed values. The difference in modelled DO and observations suggested that the model did not consider additional processes that are occurring in the water column such as the release of photosynthesis by algae and other aquatic plants.

6.6.2 Selection of single optimum parameter set for YRWQM

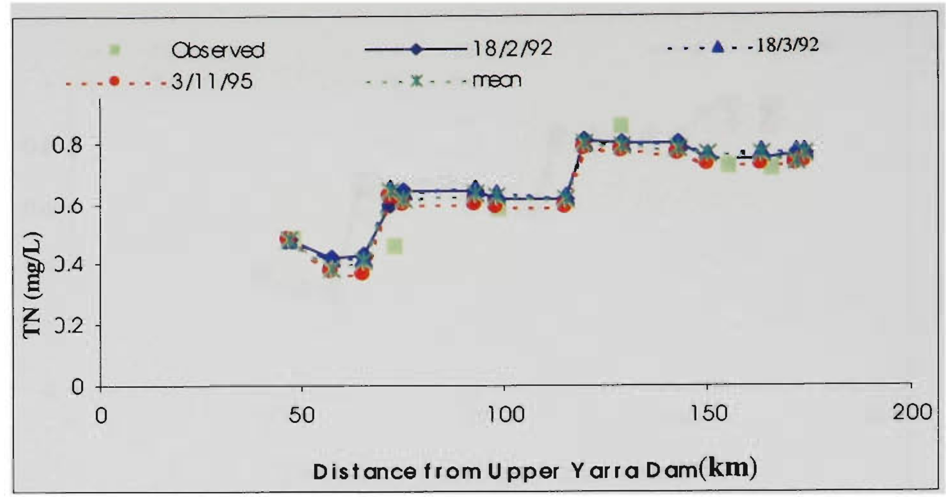
In Section 6.6.1, three 'optimised' decay rates sets were obtained corresponding to three calibration events. The statistical analyses of the results showed that water quality predictions of these parameter sets matched observations of their respective events at 95% significant level. Selection of a single 'optimum' parameter set from these 3 'optimised' parameter sets for use in YRWQM requires subjective judgement. An attempt was made in this section to select a single 'optimum' parameter set, which models the observed water quality across all three flow events with reasonable accuracy.

The three 'optimum' parameter sets and the mean parameter set (of the 3 optimised sets) were used in YRWQM to simulate the three events used for calibration. Hydraulics and other inputs except the decay rates were same as for calibration in these simulations. The comparison of water quality corresponding to four decay rates sets with observations are shown in Figures 6.20, 6.21 and 6.22 for events 18/2/92, 18/3/92 and 3/11/95 respectively.

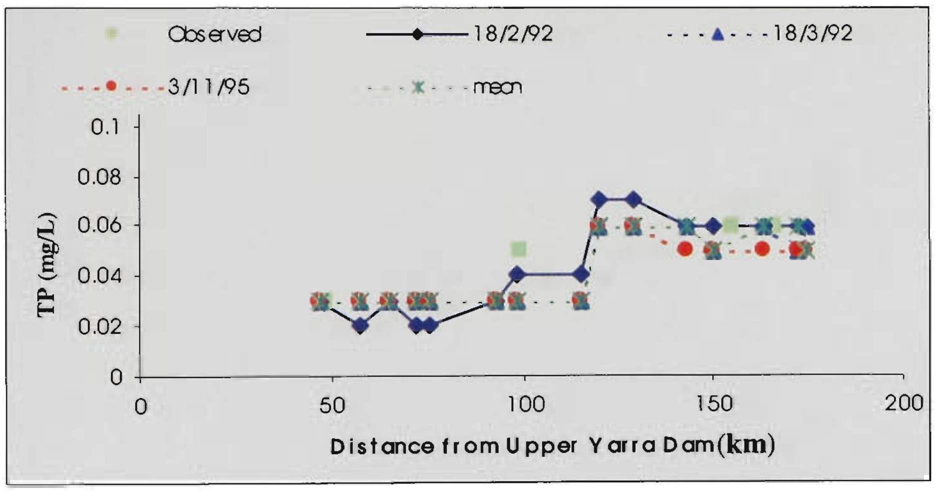
The solid line in these figures represents the output response simulated using the parameter set which was derived from the event described in the figure. Obviously, this line should be the closest to the observed concentration. Note also the fine scale used for TP compared to the other water quality constituents, which explains the relatively larger visual difference in TP. Similar DO differences were observed in the downstream as in the calibration events. As can be seen from these figures, it is very difficult to select a single 'optimum' parameter set from the four sets, since all parameter sets produced similar water quality. This also shows that the decay rates are not sensitive in YRWQM, as demonstrated in Chapter 5. This suggests that any of the four parameter sets can model the water quality of Yarra River with reasonable accuracy, since the response surface is fairly flat.



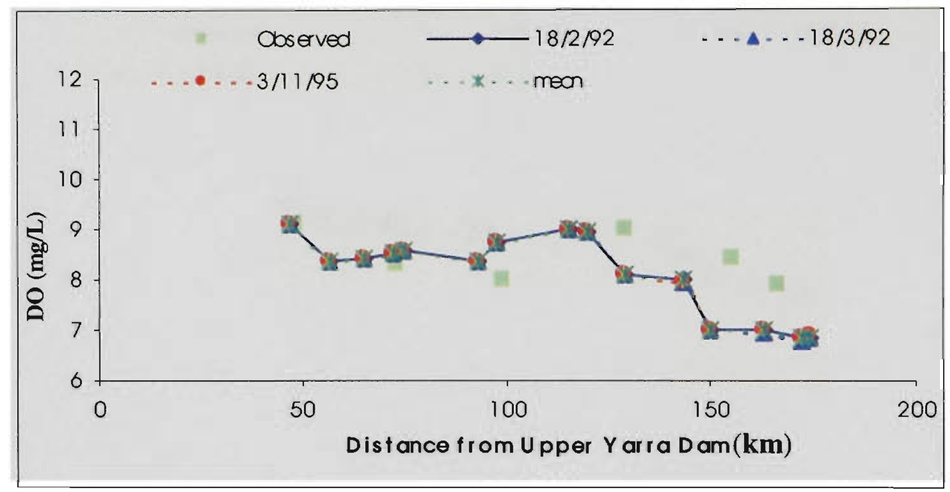
(a) TKN



(b) TN

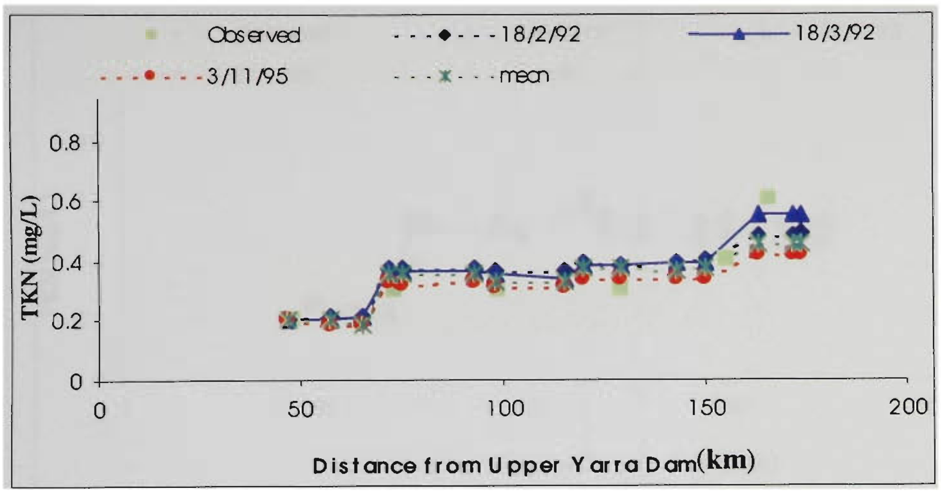


(c) TP

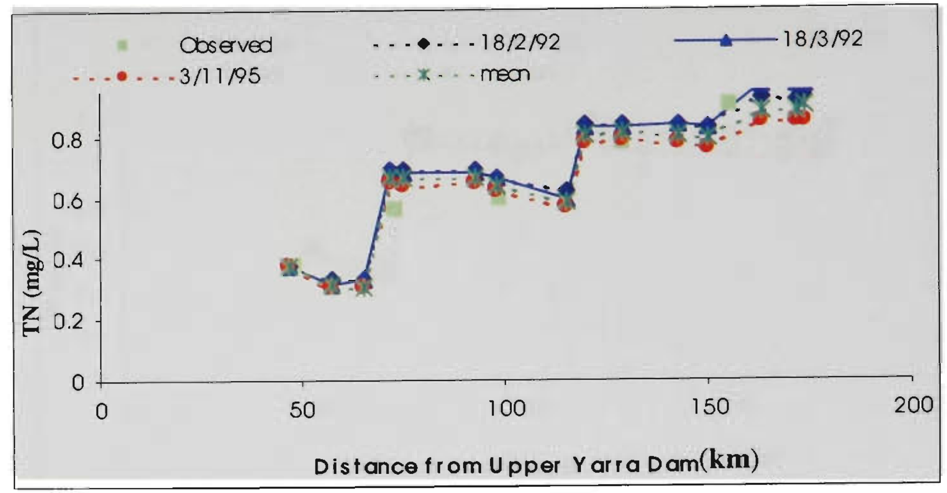


(d) DO

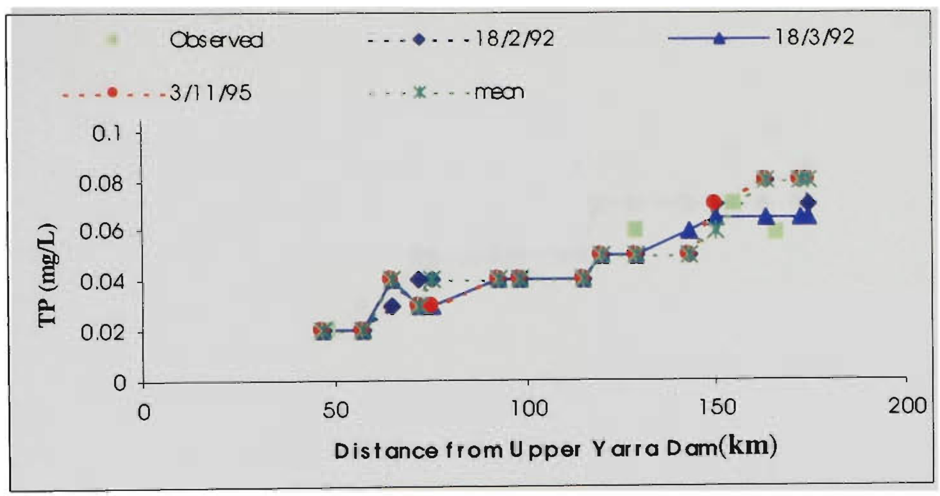
Figure 6.20 Comparison of Water Quality From Different 'Optimum' Parameter Sets for Event 18/2/92



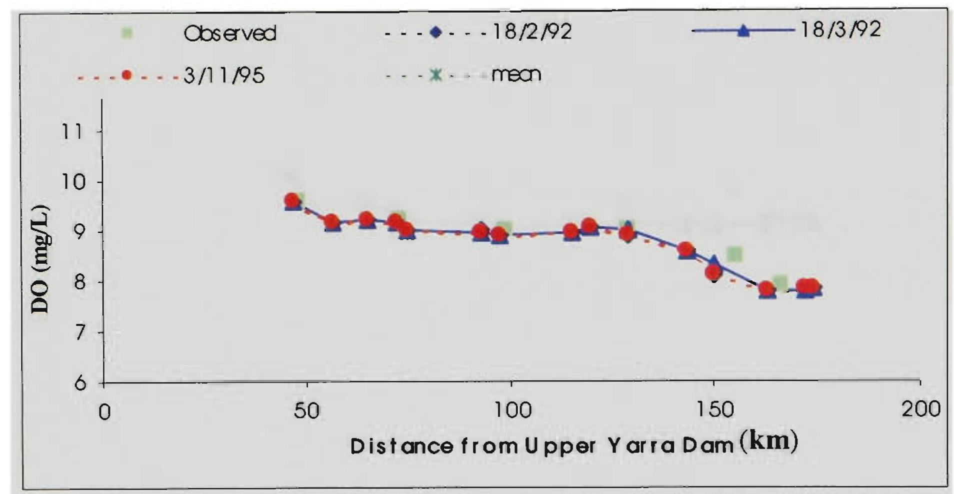
(a) TKN



(b) TN

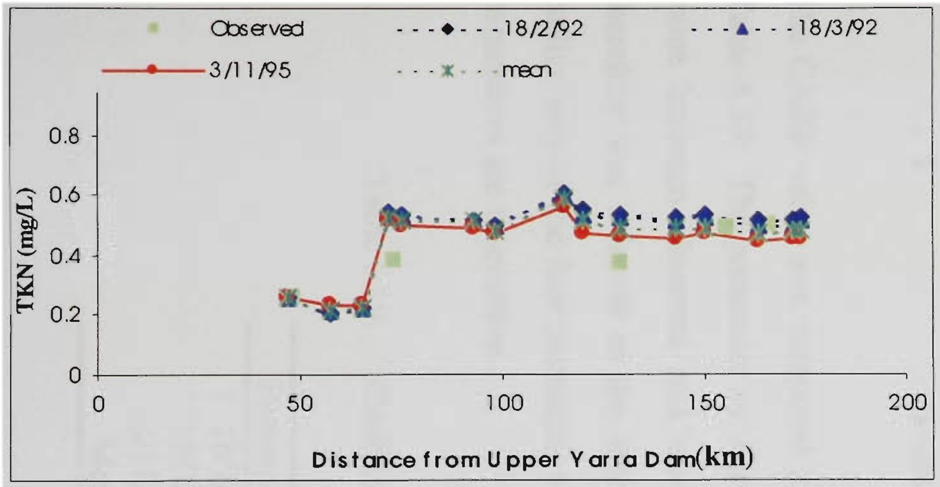


(c) TP

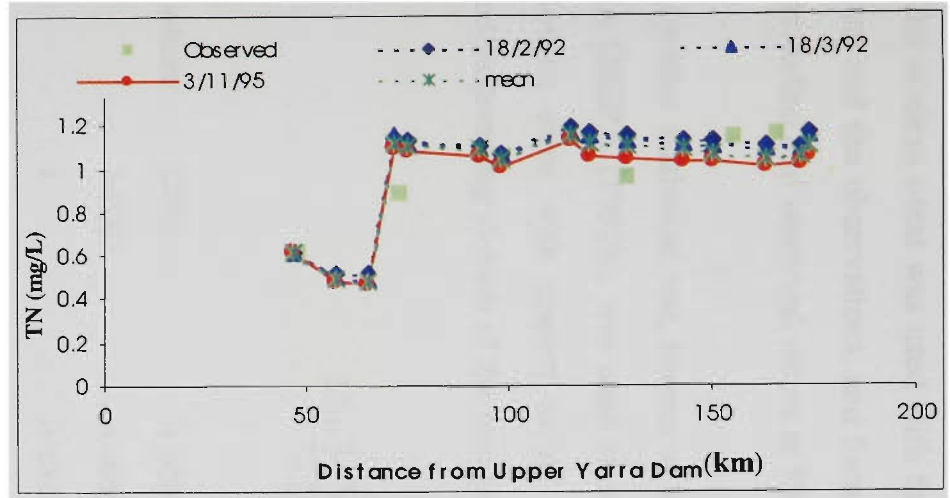


(d) DO

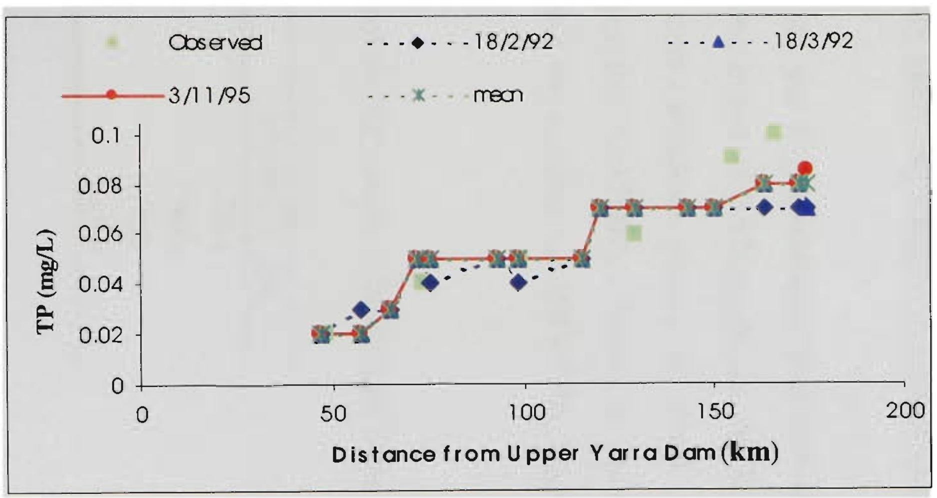
Figure 6.21 Comparison of Water Quality From Different ‘Optimum’ Parameter Sets for Event 18/3/92



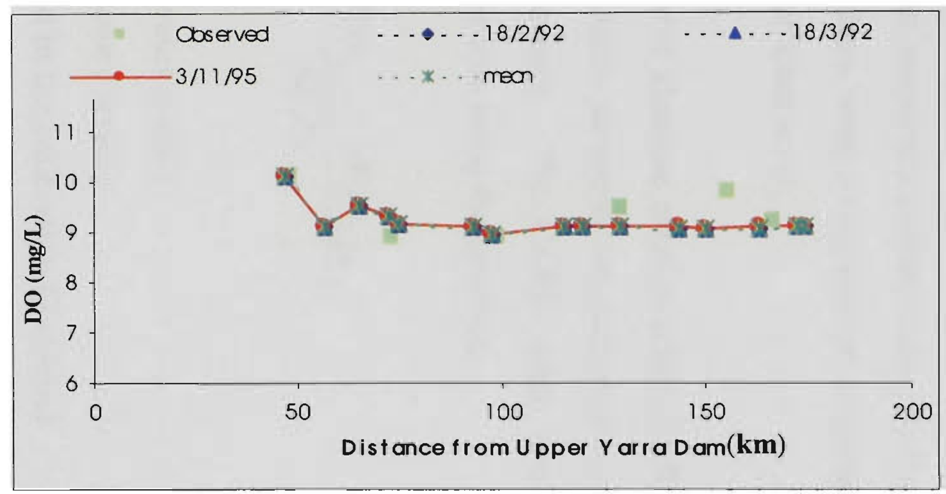
(a) TKN



(b) TN



(c) TP



(d) DO

Figure 6.22 Comparison of Water Quality From Different 'Optimum' Parameter Sets for Event 3/11/95

The student t-test was used with the output responses corresponding to four parameter sets and the observations, and found that there were no significant differences between modelled and observed values at 95 % significant level.

Another statistical test, known as cumulative absolute relative error (CARE) and cited in USEPA (1997b), was used to assess which parameter set had produced the overall lowest error with respect to all three events. The CARE value was determined corresponding to each of the four parameter sets using the function,

$$\mathrm{Min}\sum_{i=1}^{3}\sum_{j=1}^{4}\sum_{k=1}^{6}\left(\frac{OBS_{i,j,k}-MOD_{i,j,k}}{MOD_{i,j,k}}\right) \qquad 6.2$$

where	OBS	is observed water quality
	MOD	is modelled water quality
	i	is events used in the calibration (3 events)
	j	is output water quality constituents of TKN, TN, TP and DO (4 Constituents)
	k	is water quality sampling stations (6 EPAWQ stations)

The CARE value was computed for each of the four parameter sets and are shown in Table 6.19. The parameter set derived from event 18/3/92 produced the lowest CARE value between observed and modelled concentrations across all three events, and therefore was selected as the optimum set for YRWQM. However, as pointed out earlier, any of the four parameter sets would be suitable for YRWQM, since the model parameters are insensitive.

Table 6.19 CARE Produced by 'Optimised' Parameter Sets

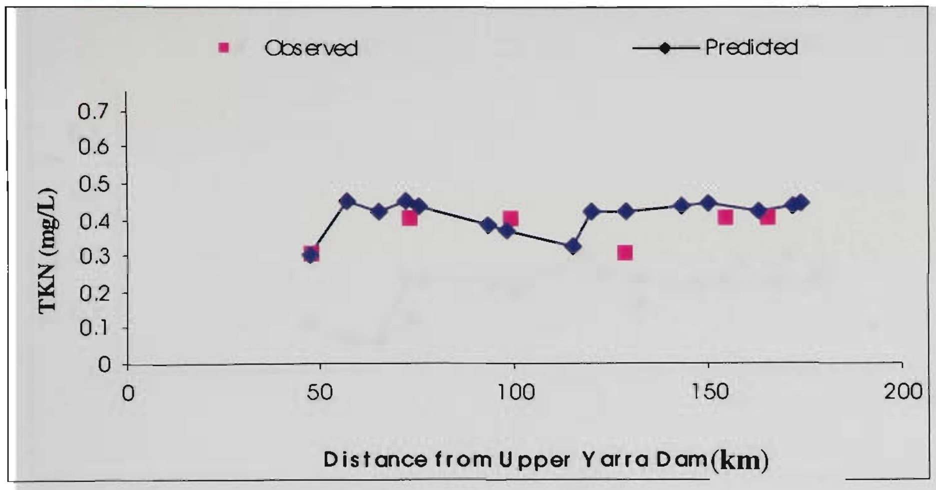
Parameter set	CARE Value
18/2/92	1.654
18/3/92	1.468
3/11/95	1.738
Mean	1.606

6.7 Verification of Yarra River Water Quality Model

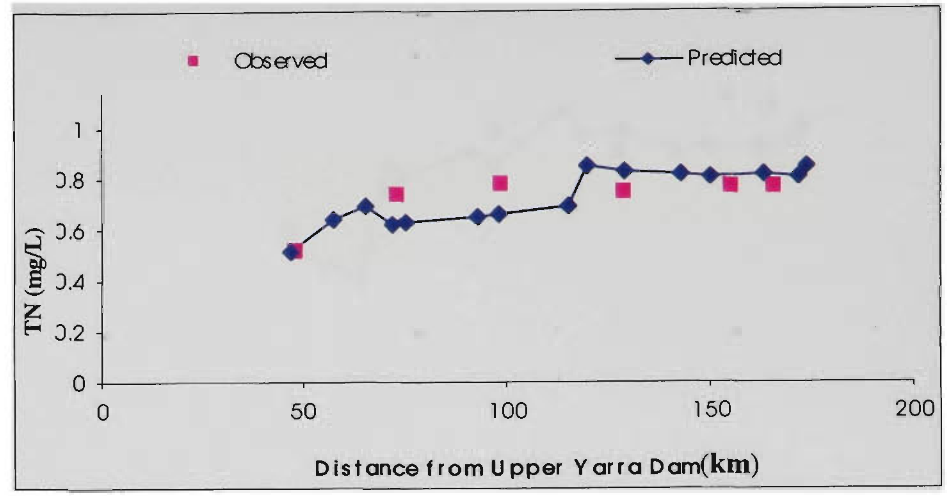
Model verification is a process which assesses the predictability of the model once it has been calibrated. This is done using independent events, which have not been used in calibration. The model verification necessary in this study, since the 'optimum' parameter set found in Section 6.6 were optimised using only three flow events and to find out whether these parameters are valid for other flow events. The model verification will enhance the confidence in using YRWQM for analysis of various management schemes in improving river water quality.

Flow events used for verification were discussed in Section 6.2. Three events namely 21/1/92, 11/6/96 and 2/4/97 were considered for model verification. Similar to calibration, the data inputs for incremental, headwater and point loads groups are event dependent, hence the estimation of these inputs used the procedures described in Sections 4.4.4-4.4.6. Again, similar to calibration events, no incremental data were required since 'lower range' flow events were considered. The global and hydraulics parameter groups were constant for all events, and they were described in Sections 4.4.1-4.4.2. The decay parameters in the reaction parameter group were obtained in Section 6.6.2, which were the 'optimised' decay parameters. The reaeration rates are determined within YRWQM during run time through the chosen reaeration rate methods as described in Section 4.4.3.2. The 'optimised' parameter set found in Section 6.6.2 was used to simulate TKN, TN, TP and DO for each of these verification events. The water quality responses were then compared with the observations at the six EPAWQ stations. Both visual inspection and the student t-test were used to assess the predictability of YRWQM with 'optimised' decay rates for verification events.

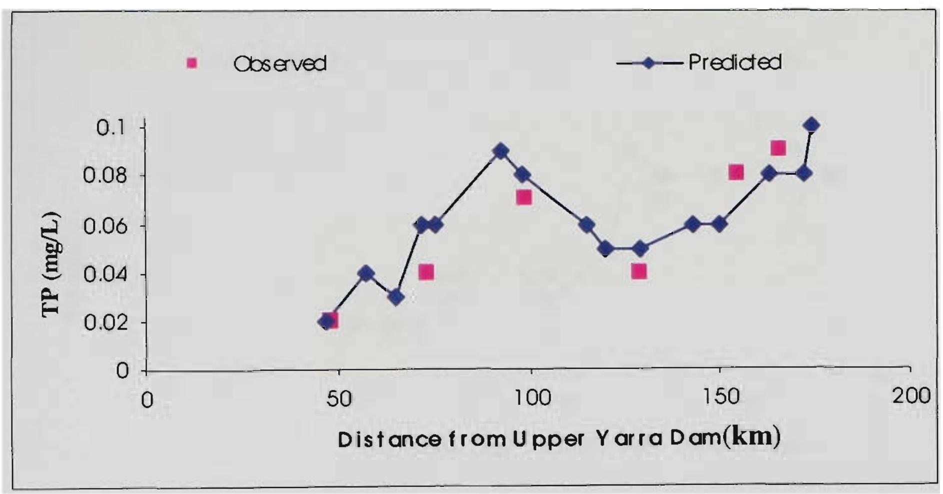
Figures 6.23, 6.24 and 6.25 show the comparison of observed and modelled TKN, TN, TP and DO concentration for verification events of 21/1/92, 11/6/96 and 2/4/97 respectively. In general, a good match between observed and modelled water quality concentrations for all three verification events were obtained. Water quality trends were matched well from Upper Yarra to Lower Yarra. The student t-test results also showed that the observed and modelled water quality concentrations were not significantly



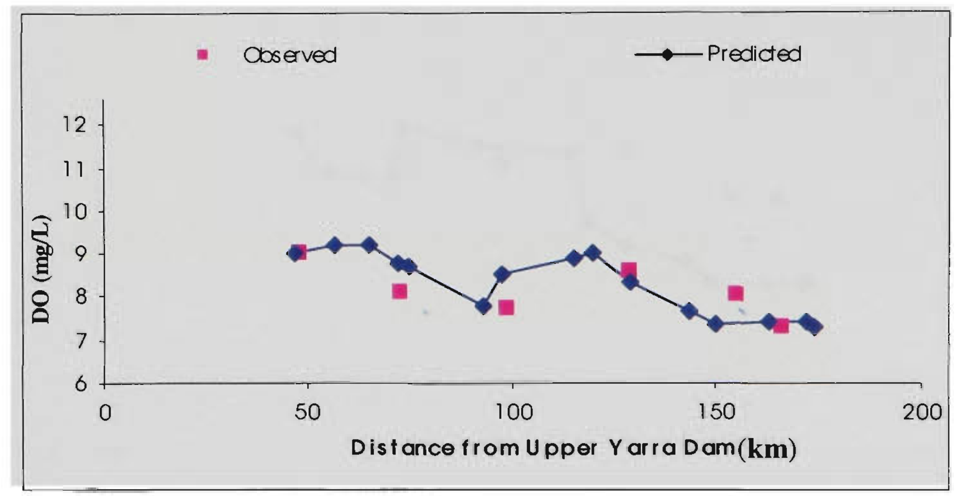
(a) TKN



(b) TN

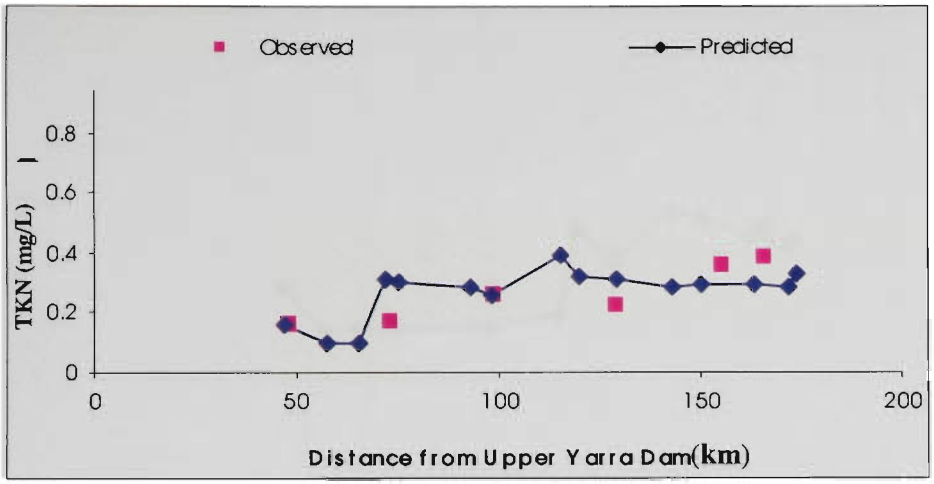


(c) TP

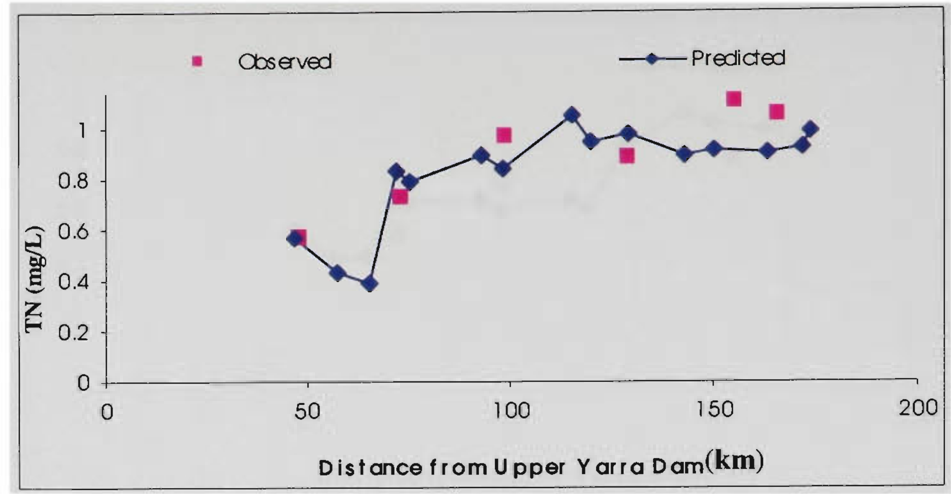


(d) DO

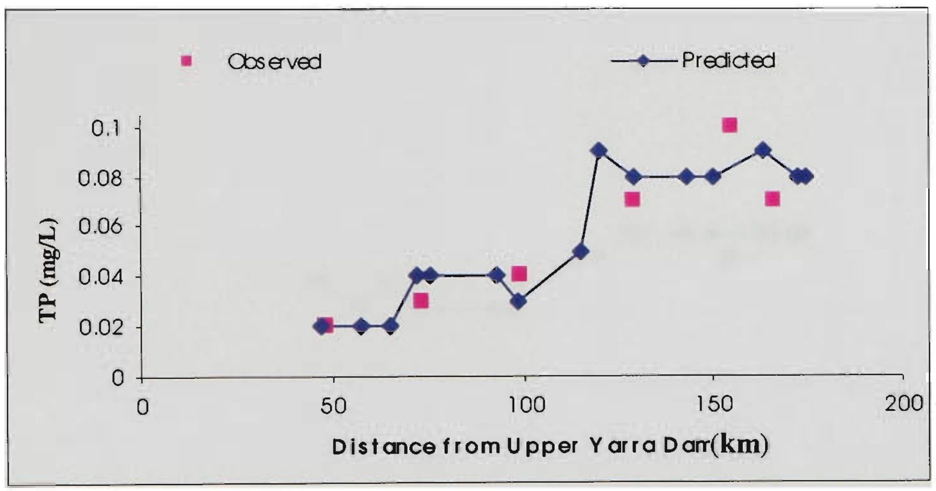
Figure 6.23 Observed and Modelled Water Quality for Event 21/1/92 (Verification)



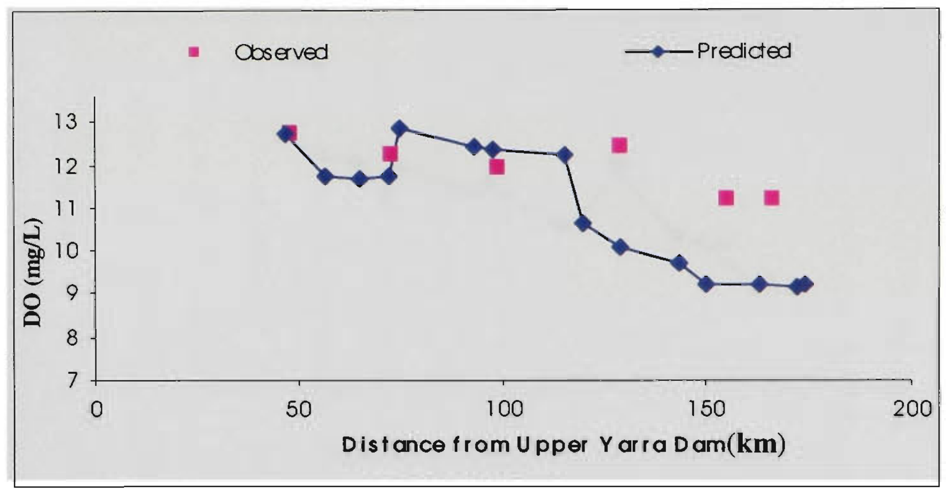
(a) TKN



(b) TN

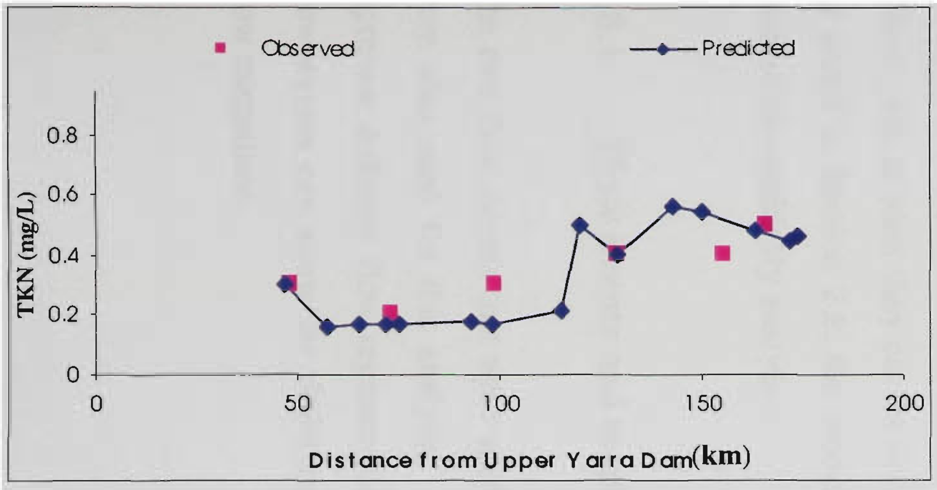


(c) TP

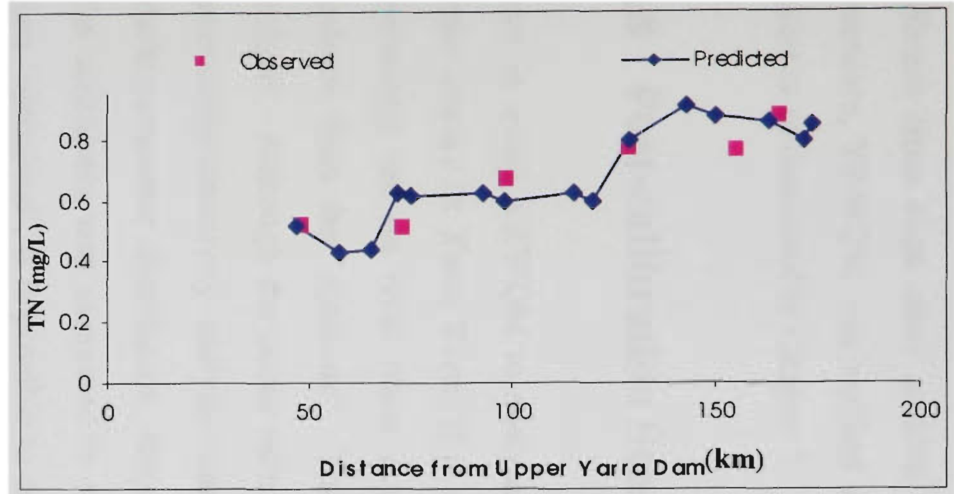


(d) DO

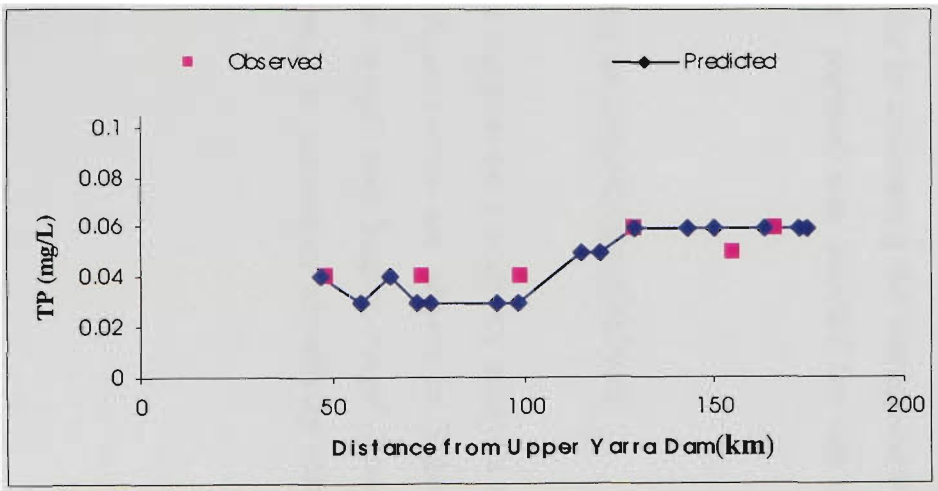
Figure 6.24 Observed and Modelled Water Quality for Event 11/6/96 (Verification)



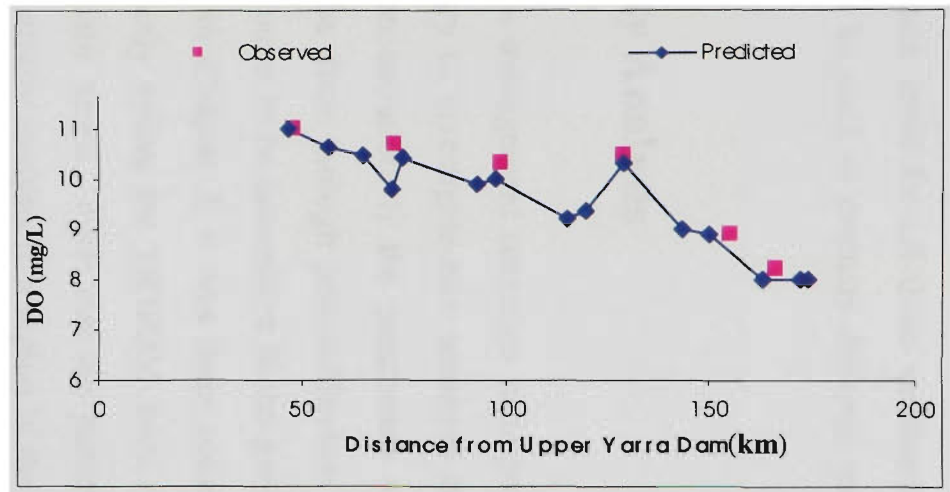
(a) TKN



(b) TN



(c) TP



(d) DO

Figure 6.25 Observed and Modelled Water Quality for Event 2/4/97 (Verification)

different from each other at 95% significant level for all three verification events. Therefore, YRWQM was verified and can be used to evaluate different management schemes as discussed in Chapter 7.

6.8 Post-calibration Sensitivity Analysis

Prior in using YRWQM to analyse various management scenarios in improving river water quality in Yarra River, it is necessary to investigate how sensitive these model parameters are on river water quality concentration, if the parameters are slightly deviates from the 'optimum'. This can be done through post-calibration sensitivity analysis. Although the decay rates were found to be insensitive in the pre-calibration uncertainty/sensitivity analysis conducted in Chapter 5, it was done considering the whole parameter distribution, since the likely values for YRWQM were not known. The sensitivity was expressed in probabilistic terms. Now that the parameters have been 'optimised', it is possible to do a sensitivity analysis in a region in the vicinity of the 'optimum' parameters values. This is the purpose of the post-calibration sensitivity analysis. If the parameters were found to be sensitive in the vicinity of the 'optimum' values, then at least they can be accounted for in evaluating the management scenarios. As stated in Section 2.8, the 'one-at-a-time' method was selected for use in the post-calibration sensitivity analysis.

6.8.1 Flow events and reaches used in sensitivity analysis

The two flow events that were used for pre-calibration sensitivity analysis (Chapter 5) were also used for this analysis. These flow events are shown in Table 6.20 and represent different flow regimes (i.e. 'lower range' and 'higher range' flows) . These flow events can show the differences (if any) in parameter sensitivity relative to the flow magnitude.

Table 6.20 Flow Events Used for Post-Calibration Sensitivity Analysis

Flow Event	Streamflow at most downstream point (m^3/s)	Flow Regime
3/11/95	7.4	'Lower range'
26/7/95	40.0	'Higher range'

As in Chapter 5, four reaches namely Reach 5, 15, 18 and 24 were selected for output analysis in comparing with observations. These reaches were selected because they were identified as critical in terms of worst water quality because of the closer proximity to STP effluent discharges.

6.8.2 Input requirements and output responses

Inputs required for 'one-at-a-time' sensitivity analysis method are the 'optimised' decay rates, and the percentage perturbation that deviates away from this 'optimised' decay rates. The other inputs in the parameter groups of incremental, headwater and point loads were kept at their actual input values determined from the flow events. The power function values determined in Section 4.4.2.1 was the input of the hydraulic parameter group. The perturbation percentages selected for this study were ±10%, ±25% and ±50% from the 'optimised' decay rates values. This range of perturbation can clearly show the level of sensitivity of each decay rate to the output water quality response.

The 'optimised' decay rates set obtained in Section 6.6.2 is shown in Table 6.21. These decay rates are constant for all 29 Reaches, as stated in Section 6.3.1. In addition to these decay rates, the reaeration rate was also considered in this section, although it was not part of the optimisation. As stated in Chapter 5, the value of reaeration rate (not the method) needs to be specified to conduct the sensitivity analysis. Therefore, one YRWQM simulation was run for each of the event to obtain the reaeration rate for each reach from the reaeration method in YRWQM. These values (i.e. 'optimised' decay rates and reaeration rates) were then used as the base (or 'optimised') values to conduct the sensitivity analysis. The output responses of TN, TP and DO were used to assess the sensitivity of the 'optimised' decay rates.

Table 6.21 'Optimised' Decay Rates of YRWQM

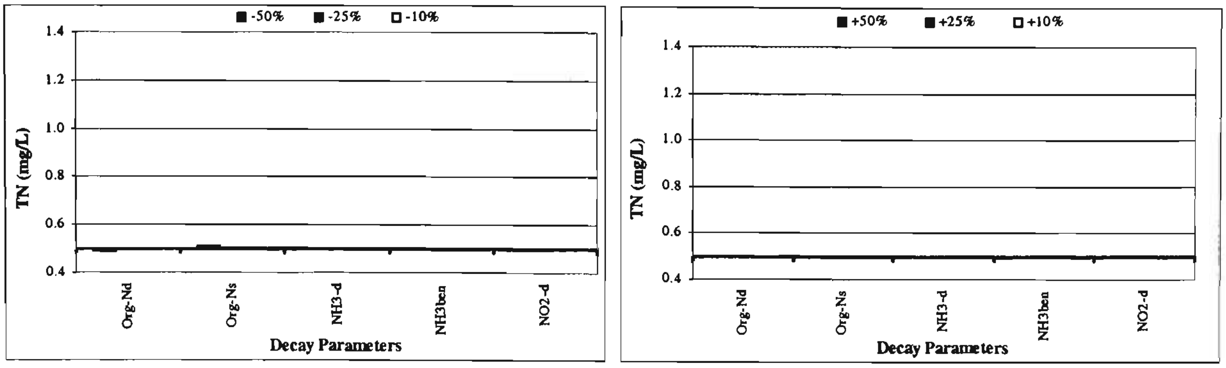
Decay rates	'Optimised' decay rates (day^{-1})
Org-N_d	0.329
Org-N_s	0.048
$NH_{3\text{-}d}$	0.016
NH_3ben	1.526
$NO_{2\text{-}d}$	0.525
Org-P_d	0.366
Org-P_s	0.128
Diss-Pben	0.924
$CBOD_d$	0.012
$CBOD_s$	0.342
SOD	0.001

6.8.3 Results and discussion

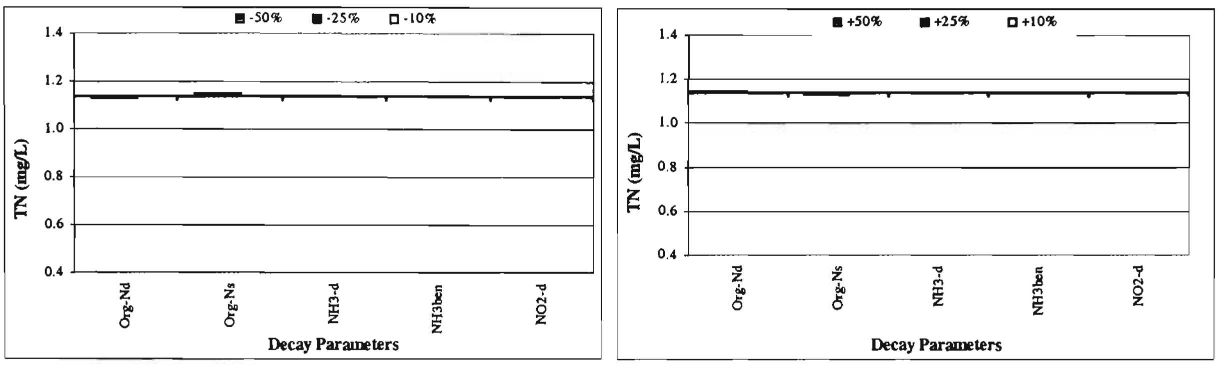
Figures 6.26 and 6.27 show the sensitivity of decay rates on TN and DO concentrations for perturbations of ±10%, ±25% and ±50% for the 'lower range' flow event respectively, at Reaches 5, 15, 18 and 24. Similar plots for the 'higher range' flow event is shown in Figures 6.28 and 6.29. Note that no plots were shown for TP concentration, because no decay rates were identified as sensitive to TP concentration. The output results produced from the 'optimised' decay rate set was treated as the base line result (indicated with the thick black line), and the sensitivity is shown by the deviations from this base line.

(a) TN concentration

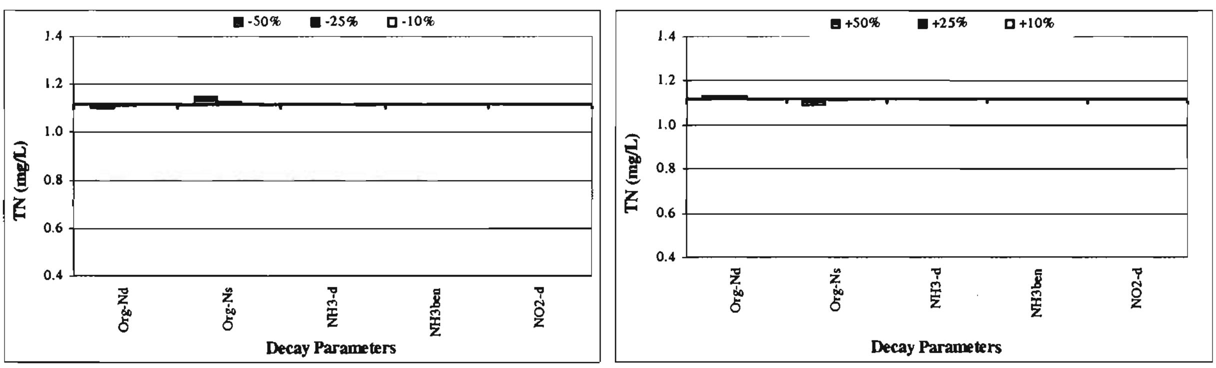
As seen from Figures 6.26 and 6.28, the effect of nitrogen decay rates (i.e. Org-N_d, Org-N_s, $NH_{3\text{-}d}$ and NH_3ben) on TN concentration is small. Of the four nitrogen decay rates, only deviations in Org-N_d and Org-N_s rates have caused small sensitivity to TN concentration under both flow conditions when large deviations away from the 'optimised' value occurs.



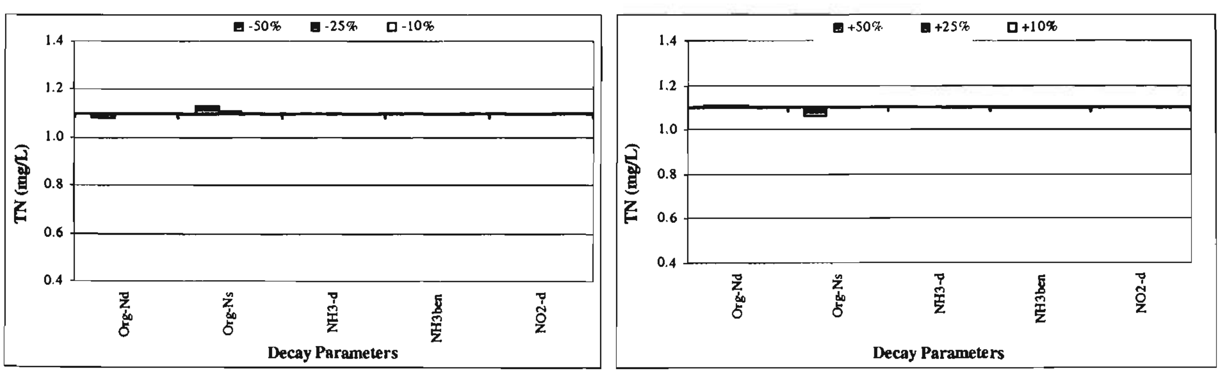
(a) Reach 5



(b) Reach 15

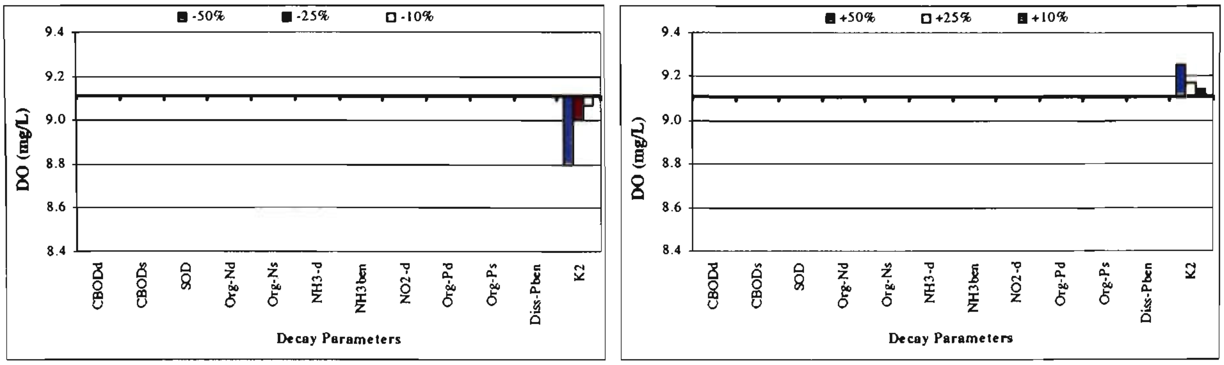


(c) Reach 18

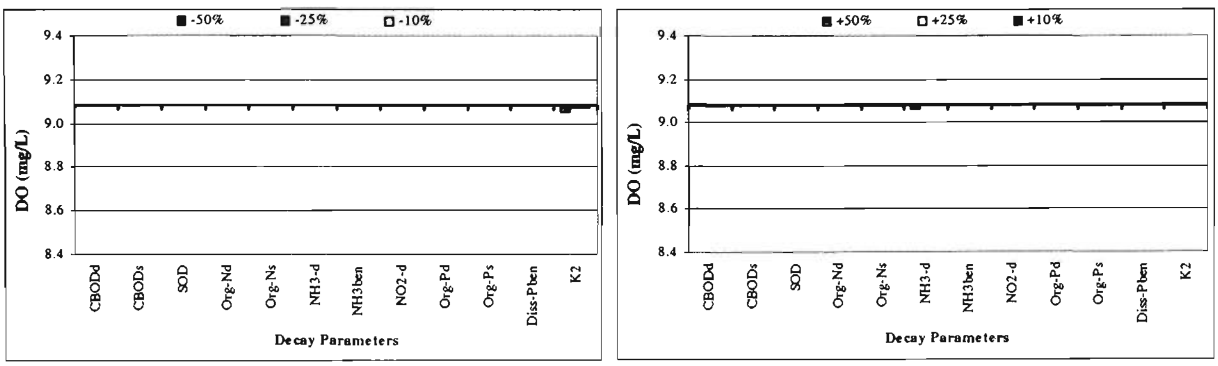


(d) Reach 24

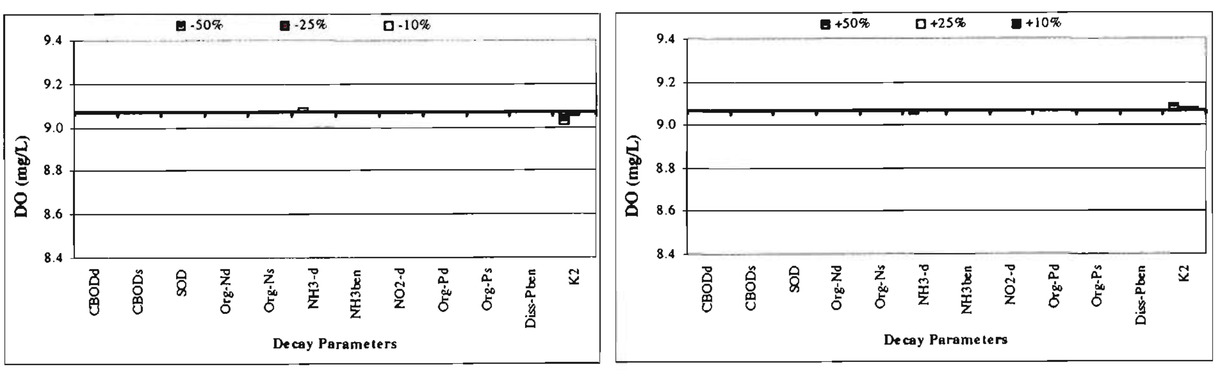
Figure 6.26 Sensitivity of TN Decay Rates (for 'Lower Range' Flow Event)



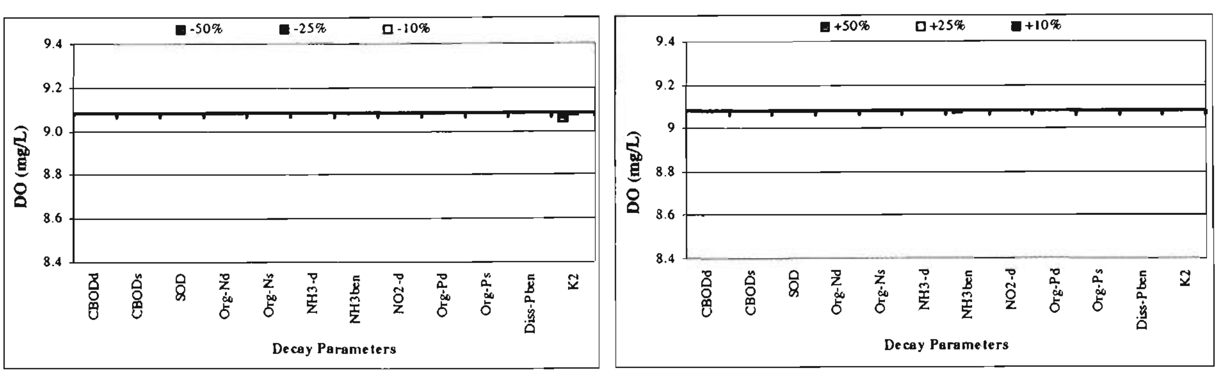
(a) Reach 5



(b) Reach 15

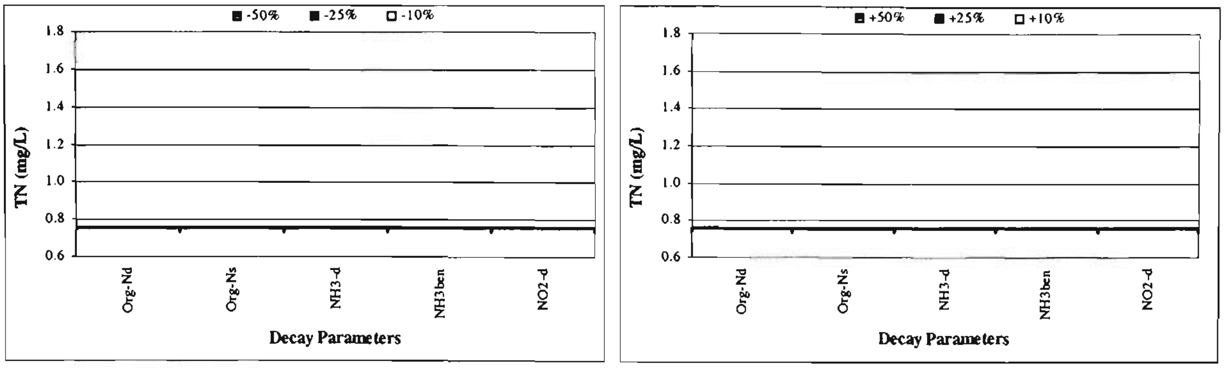


(c) Reach 18

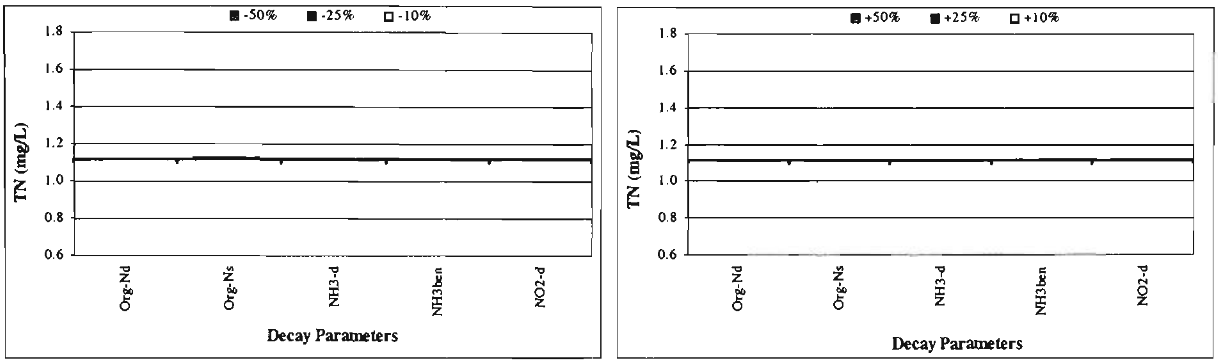


(d) Reach 24

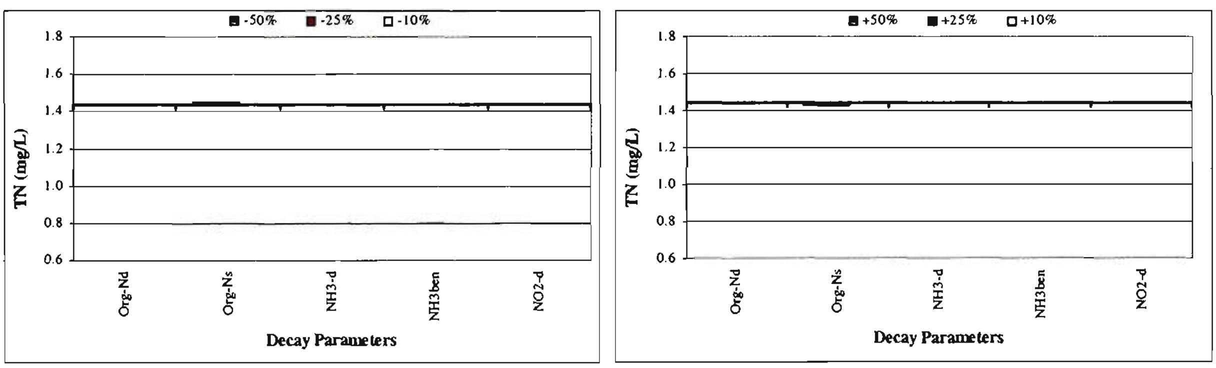
Figure 6.27 Sensitivity of DO Decay Rates (for 'Lower Range' Flow Event)



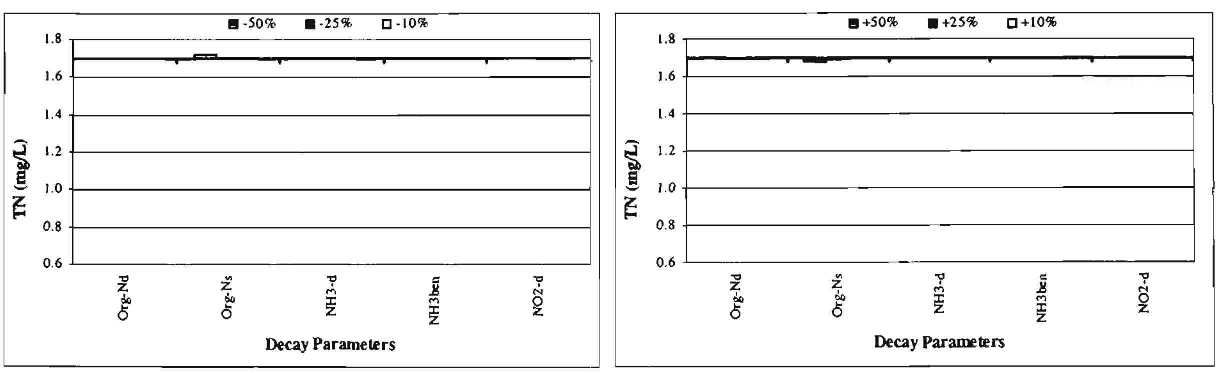
(a) Reach 5



(b) Reach 15

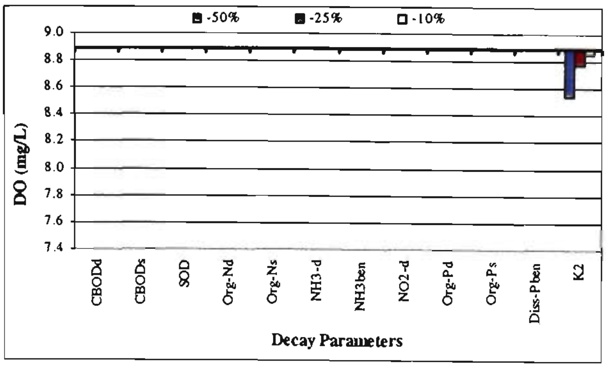


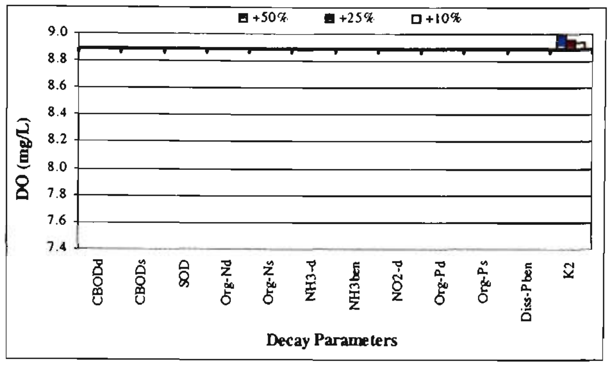
(c) Reach 18



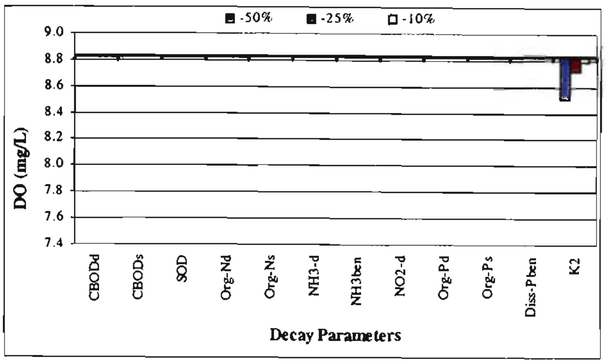
(d) Reach 24

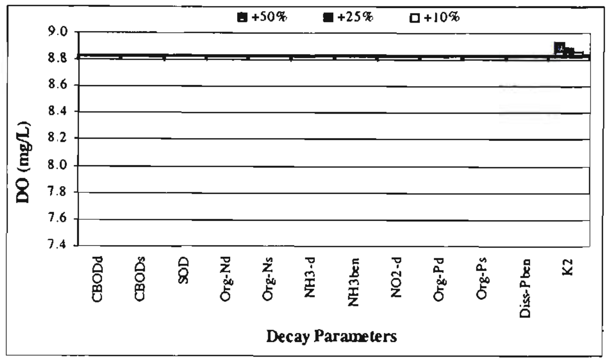
Figure 6.28 Sensitivity of TN Decay Rates (for 'Higher Range' Flow Event)



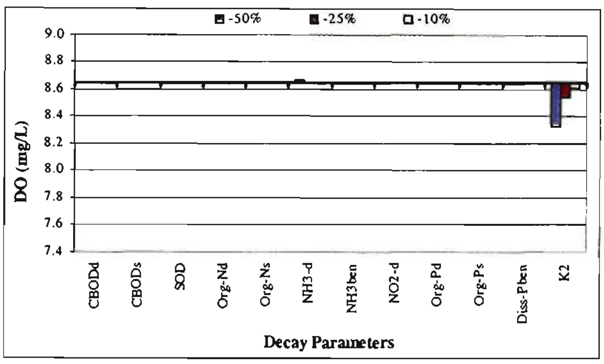


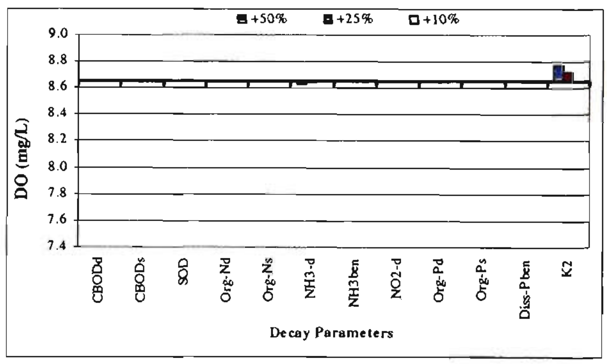
(a) Reach 5



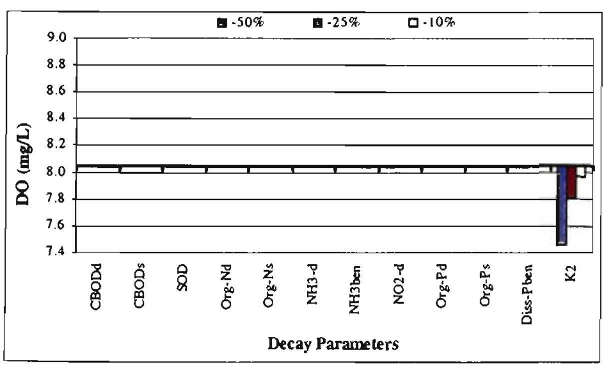


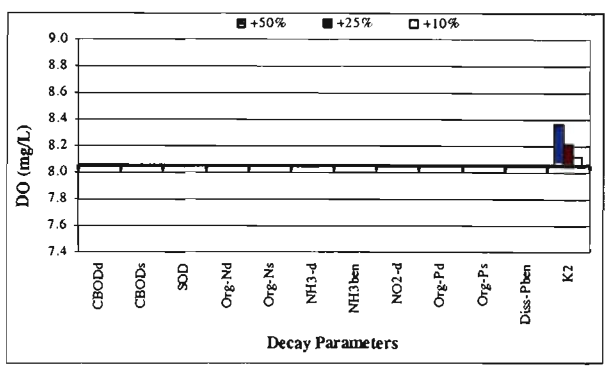
(b) Reach 15





(c) Reach 18





(d) Reach 24

Figure 6.29 Sensitivity of DO Decay Rates (for 'Higher Range' Flow Event)

When Org-N_d increases 50% from the 'optimised' value, the TN concentration increases from the base case. As the rate increases, the level of concentration should decay faster. However, the decay of nitrogen is via a cascade process in which Org-N_d is transformed to NH_3, where NH_3 concentration increases, hence the TN concentration in turn, increases. This behaviour is pronounced in the 'lower range' flow than in the 'higher range' flow, as decay activities is more pronounced under 'lower range' flow because of generally high temperature.

When Org-N_s increases 50% from the 'optimised' value as shown in Figure 6.26, the TN concentration shows a decrease in concentration of around 0.2% from the base value at reaches 18 and 24 under 'lower range' flow. As Org-N_s decreases 50% from the 'optimised' value, the TN concentration increases to around 0.2% from the base case as shown in Figure 6.26, which is also expected. The settling effect is more pronounced in 'lower range' flow than in 'higher range' flow conditions (comparisons of Figures 6.26 and 6.28), because under 'higher range' flow, the settling activity is less and, hence sensitivity is less.

(b) TP concentration

No phosphorus decay rates were found as sensitive to TP concentration for both 'lower range' and 'higher range' flow conditions and therefore, no sensitivity plots were shown. This was also found in Section 5.6.3.

(c) DO concentration

As can be seen from Figure 6.27, most decay rates except the reaeration rate and $NH_{3\text{-}d}$ decay rate were found insensitive to DO concentration under 'lower range' flow conditions. This was also valid for 'higher range' flow conditions. However, the effect on DO concentration was significantly more in 'higher range' flows, in particular downstream reaches.

As the reaeration rate decreases to 50% from the 'optimised' value, the DO concentration only decreased by about 7% from the base DO concentration at Reach 24 under 'higher range' flow condition, which is the largest difference observed from both 'lower range' and 'higher range' flow events. When the reaeration rate decreases to 10% from the 'optimised' value, the DO concentration at Reach 24 reduces only 1% from the base value.

This shows the sensitivity of reaeration rate is insignificant to DO concentration prediction, because 50% deviation away from the 'optimised' value can still predict DO concentration within 95% of concentrations obtained from 'optimised' parameters. This finding was comparable with the findings in the preliminary uncertainty and sensitivity analysis on decay rates in Section 5.6.3. Based on the results, it can be said that the reaeration rate is insensitive to DO concentration in Yarra River as indicated from this analysis and in Chapter 5, although some small sensitivity was shown in Figures 6.27 and 6.29.

The effect of $NH_{3\text{-}d}$ on DO concentration was less compared to the effect of reaeration. Its effect only shows when the deviation from the 'optimised' value is 50%. Due to transformation from NH_3 to NO_2 and subsequently to NO_3, which requires oxygen, the decrease in DO concentration should be expected. This effect is small, only around 0.1% from the base value even for a 50% increase in $NH_{3\text{-}d}$ rate from the 'optimised' value, and is mostly affected under 'lower range' flow conditions, as shown in Figure 6.27. This is also expected as nitrification is more pronounced under 'lower range' flow condition where the temperature is generally higher which enhances the decaying process. This result was also comparable with the findings in Chapter 5, where $NH_{3\text{-}d}$ has a small sensitivity on DO, shown with a value of RDR less than 1.

Although there are some changes in DO when both reaeration rate and $NH_{3\text{-}d}$ deviate 50% from the 'optimised' values, they can still able to predict DO concentrations within 95% of concentrations obtained from 'optimised' parameters. Hence, it can be said that the 'optimised' decay rates can be used confidently in YRWQM in predicting DO.

(d) Final remarks

Based on the above analysis, it was found that the decay parameters and the reaeration rates are not sensitive to TKN, TN, TP and DO if these parameters are changed by 50% from their 'optimised' values, provided that the hydraulics parameters and other flow event dependent input parameters are determined accurately.

6.9 Summary and Conclusions

Simulation models are used to assess various management scenarios in improving river water quality. In order to use these simulation models confidently, the models must be well-calibrated. Model calibration (or often referred to as parameter estimation) can yield a set of model parameters which best estimate conditions that match with the observations. The calibrated model can then be used to simulate various management scenarios so that the implementation of water quality policy can be done in the most efficient way.

Model calibration can be done using manual and automatic methods. The manual methods are usually trial and error approaches, which are time consuming and require subjective judgement in defining the 'optimum' parameter set. They can often miss the 'optimum' parameter set. On the other hand, the automatic calibration methods provide some measure of objectivity in calibrating the models and obtaining the 'optimum' parameters. Genetic algorithm (GA) is one such automatic calibration method and was used in this study to calibrate the Yarra River Water Quality Model (YRWQM). GA was chosen because it was found to be one of the widely used optimisation methods in recent times and has proven to provide the global optimum solution in complex search spaces. A public domain GA software called GENESIS was linked with YRWQM to perform the parameter optimisation in this study.

Several previous studies have investigated the effect of GA operators in achieving the 'optimum' parameter set, however, the findings were inconclusive. Therefore, the importance of GA operators was initially investigated using a hypothetical river network system, for which data inputs were known from Chapra (1998). Since the decay rates and other inputs were known, the DO output response was simulated. The aim of this part of the exercise was then to investigate the effect of different GA operator sets in yielding the 'optimum' decay rates (i.e. equal to actual parameters) to achieve the above DO output response. The Monte Carlo simulation (MCS) uncertainty/sensitivity analysis showed that the parameters of this model was insensitive. However, with small modifications to effluent quality from the STP, a sensitive model was also developed to study the effect of GA operator sets on the optimum decay parameter set in this model.

The investigation on the number of parameter set taken from the last generation and the effect of the seed were conducted first for both hypothetical river network models and YRWQM. This was done by using a GA operator set compiled from literature (known as 'LIT' set). It was found in both networks that the best 10 parameter sets obtained from the final generation can be used to determine the 'optimum' parameter set. The effect of seed was considered negligible on the optimum parameter set when several GA runs were conducted sequentially with a different seed. These number of runs yield reduced search space from one to the next. In this study, three GA runs were able to obtain the reduced search space which can remove the effect of seed.

To compare the effect of GA operators, first the 'LIT' set was used on the hypothetical river networks. It was found that by using the 'LIT' GA operator set, the 'optimum' parameter set was able to converge in the sensitive model, but not in the sensitive model. Although the convergence was unable to reach for the insensitive model, the prediction of output water quality response was very close (within 0.25% of the actual DO concentration). Since the 'LIT' GA operator set was unable to converge in the insensitive model, a systematic optimisation of GA operator sets was conducted to obtain a better GA operator set which yields the 'optimum' decay parameter set as the actual parameter set. The optimum GA operator set (known as 'OPTI') was found with a population size of 125, 15000 simulations, crossover rate of 0.69 and mutation rate of 0.005.

Since the YRWQM was also an insensitive model (as was shown in Chapter 5), similar approaches to the hypothetical river network insensitive model was employed in YRWQM to study the effect of GA operators on the optimum parameter set. The effect of 'LIT' and 'OPTI' GA operator sets in finding the 'optimum' parameter set was investigated. It was found that both sets yield comparable water quality prediction in the river, although, the actual 'optimised' parameters were different. Due to the insignificant difference in the output water quality prediction, the 'LIT' GA operator set was adopted in the YRWQM calibration.

Eleven decay rates (i.e. model parameters) were considered in YRWQM model calibration. The estimation of these decay rates was done systematically by considering parameters of the water quality constituents which are not affected by other water quality constituents are estimated. Then, these parameters are kept constant, and the parameters of other water quality constituents are estimated as previously. Therefore, the calibration process was first to estimate all decay rates that affect total kjeldahl nitrogen (TKN), and then total nitrogen (TN) concentration. Decay rates that affect TP was conducted in conjunction with TKN or TP, since decay rates of total phosphorus (TP) do not affect TKN and TN. Finally, when all decay rates were 'optimised', additional decay rates that affect DO were then 'optimised'. The data on water quality concentrations of TKN, TN, TP and DO measured at the Environment Protection Authority Water Quality (EPAWQ) sampling points were used in calibration.

Three 'lower range' flow events were used in YRWQM calibration. These 'lower range' flow events were considered since the effect of nonpoint sources can be eliminated in the calibration, which do not have any effect on decay rates. In other words, once the decay rates are determined, they are equally valid for both 'lower range' flow events (which do not have nonpoint source flows) and 'higher range' flow events (which may include nonpoint sources). Use of 'lower range' flow events in YRWQM calibration reduces complexity in the estimation of decay rates. The decay parameters were obtained from GA optimisation using the three flow events. All three parameter sets were able to match the observed water quality trend and predicted water quality to match observations at 95% significant level. Generally, all decay rates obtained from calibration were found within the published range. The water column decay rates (e.g. NH_{3-d}) were mostly found to be very low in Yarra River because of the progressive resistance of further breakdown of organic matter due to highly treated STP effluent wastewater. On the other hand, benthos rate was generally high in Yarra River as many organic matter such as leaf litter settles in the river bed during low velocities. Of the three parameter sets obtained from GA optimisation, a single 'optimised' set was selected using a statistical test known as cumulative absolute relative error (CARE).

Three other 'lower range' flow events (which were not used in calibration) were used in model verification. The single 'optimised' parameter set obtained during calibration was used with these verification events. It was found that no significant differences in the observed and predicted water quality in the three events at 95% significant level. Since the model was verified, it can be confidently used as a simulation tool to evaluate various management scenarios to efficiently improve Yarra River water quality.

A post-calibration sensitivity analysis of model parameters was conducted to assess the effect on the output water quality for deviations from the 'optimised' decay parameters. This sensitivity analysis can enhance the confidence in using this model for decision making. The 'one-at-a-time' sensitivity analysis method was used in this study where one decay parameter was varied by ±10%, ±25% and ±50% from the 'optimised' parameter set at a time, while the other decay rates were kept constant. Other parameter groups (i.e. hydraulics, incremental, headwater and point load) had been obtained from data (i.e. hydrographic and flow event data), while the parameters of the global parameter group were kept at the default values of QUAL2E. One 'lower range' and a 'higher range' flow event were used for this analysis. It was found that in general the decay parameters were insensitive to TN, TP and DO in YRWQM. This study also showed that any deviations upto 50% of the 'optimised' parameter set in YRWQM would not effect the output results.

CHAPTER 7

EVALUATION OF POLLUTION POINT SOURCE MANAGEMENT POLICIES USING YRWQM

7.1 Introduction

In many instances, the receiving waters such as rivers, streams and oceans have become disposal sites for treated effluent discharges, especially in developing countries. Even in developed countries, limited treated effluent discharges are disposed to receiving waters. Efficient water quality management is necessary to minimise the impact of effluent disposal into receiving waters, and can be achieved through a well-calibrated river water quality models.

There is a direct relationship between the strength of the effluent discharged into the receiving stream and stream assimilative capacity. The stream assimilative capacity is the ability to digest the incoming pollution sources. Effluent treatment can be reduced (to a certain level which will not violate the river water quality) as the assimilative capacity of the stream is high. The assimilative capacity also varies with flow and temperature. The greater the flow (e.g. during winter), the greater the assimilative capacity and vice versa. Similarly, the higher the temperature, the lower the assimilative capacity.

The seasonal variability of flows in rivers can be used to manage the effluent disposal to rivers. Effluent with higher concentration can be disposed to rivers during high flow periods without deteriorating the river water quality below the required standards. Therefore, it is possible to define different effluent license limits for sewage treatment plants (STPs) in different seasons of the year. These effluent license limits define the allowable flow and its concentration (in terms of critical water quality constituents) that can be discharged to the receiving waters.

The Environment Protection Authority of Victoria (EPAVIC) sets regulations on STP effluent license limits based on Best Available Technology (BAT) in protecting and managing waterways in Victoria. Using BAT in setting STP effluent license limits may not be the most efficient, because less consideration is given to the assimilative capacity of receiving waters. Prior to 1997, the EPAVIC has set effluent license limits on STPs on Yarra River and its tributaries to the level of tertiary treatment. Since then, further stringent effluent license limits were set and all STPs which discharge effluent into Yarra River and its tributaries use these stringent effluent license limits. This upgrade of effluent license limits caused significant increases in costs (around 50% both capital and operating costs) to water authorities (EPA Victoria, 1995a). Despite the unknown level of improvement from one effluent license limit to another, the EPAVIC has proposed further stringent effluent license limits in Year 2004 (EPA Victoria, 1999). This proposal will increase substantial costs again to water authorities. However, the actual benefits in relation to water quality improvement of this proposal are unknown and therefore require an investigation.

The Yarra River Water Quality Model (YRWQM) development which described from Chapters 4 to 6 can be used to assess the river responses due to various effluent discharge strategies. The assessment of output water quality from YRWQM can indicate the assimilative capacity of the Yarra River. Dissolved oxygen (DO) concentration can be used to give an indication of river assimilative capacities. When the DO concentration shows a higher concentration than the water quality standard imposed on the river, the assimilative capacity of the river can be considered. Once the assimilative capacity of Yarra River is known, the effectiveness of further increase in effluent license limits can be assessed. Therefore, the main aim of this chapter is to evaluate different point source management strategies on STPs in improving the water quality in Yarra River. In order to achieve this main aim, a comparison of the modelled river water quality due to different effluent license limits (which include 'Prior 1997', 'current' and '2004') was conducted. In addition, the feasibility of using an alternative seasonal effluent discharge program in managing point source pollution of Yarra River STPs was investigated. These seasonal effluent discharge programs are known to be efficient (Ferrara and Dimino, 1985). These investigations are discussed in this chapter.

This chapter begins by introducing the types of water quality management strategies that can be used to manage and to improve the Yarra River water quality, focusing on the point source management. Then, a comparison of the water quality responses due to different STP effluent license limits was conducted. Finally, an investigation on the feasibility of applying seasonal effluent discharge limits was explored.

7.2 Water Quality Management in Yarra River Catchment

The management of river catchments in Victoria (Australia) and their water quality began in 1970s through the State Environmental Protection Policy (SEPP) (EPA Victoria, 1999), which provided the overall framework for the protection of all waters in Victoria. Due to different land use and catchment activities, the EPAVIC then realised that specific management strategies were required for different rivers. Therefore, a policy which specifically targeted the Yarra River catchment was evolved in 1984, later revised in 1997 and recently released in 1999, as Schedule F7 (Waters of the Yarra catchment) (EPA Victoria, 1999). Similar Schedules for other waters in Victoria were also published. Examples are Schedule F5 for waters of Latrobe and Thomson River Basins, and Schedule F6 for Merriman Creek catchment and waters of Port Phillip Bay.

The Schedule F7, identified seven environmental threats as the cause of degradation of the Yarra River catchment and its quality, and they are:

- Effluent licensed discharges
- Waterway degradation
- Urban stormwater runoff
- Modified flow regimes
- Runoff from nonurban land
- Losses from the sewerage system
- Unsewered areas

Management action plans were developed in Schedule F7 to mitigate the above problems in the Yarra River catchment. It was realised in this document that multiple problems caused the degradation of water quality in Yarra River, and therefore the Yarra River and its catchment should be managed through multiple strategies undertaken at the same time using the integrated catchment management (ICM) techniques. ICM considers all relevant cause and effect relationships in the catchment and makes the most use of resources to resolve the water quality problem in the catchment as a whole.

Of the seven identified problems above, effluent (or treated wastewater) discharged from STPs into the receiving streams has often been identified as the major contributor to the water quality pollution (EPA Victoria, 1999). This is perhaps due to wastewater being continuously discharged at a single concentrated point, which causes significant deterioration of river condition at that point. In addition, the management of licensed discharges is straightforward which can be easily monitored and controlled. Due to these reasons and other possible reasons, the effluent license limits on STPs were progressively made stringent since 1997 and there are plans for further stringent license limits.

Prior to 1997, the effluent license limits set by the EPAVIC on the 13 STPs of Yarra River catchment are shown in Table 7.1. The effluent discharge license volume varied for each STP. The wastewater treatment process used in each STP was activated sludge biological treatment with either Ultra Violet irradiation or chlorine, as the final disinfection. In July 1997, the EPAVIC set new stringent effluent license limits for Yarra River catchment STPs, which are currently in use. These new effluent license limits led to closures and amalgamation of some STPs, because the plants were unable to meet the required standard. Details of these STP modifications were discussed in Section 3.2.3. These effluent license limits were uniform across all STPs and defined by 10 mg/L of biochemical oxygen demand (BOD), 10 mg/L of total nitrogen (TN), and 1 mg/L of total phosphorus (TP), as shown in Table 7.2. The treatment process used was the same as in prior 1997 STPs (i.e. activated sludge biological treatment with either Ultra Violet irradiation or chlorine, as the final disinfection). Further stringent effluent license limits are planned for Year 2004 so that the effluent discharge should not cause any river water quality differences between upstream and downstream of the

Table 7.1 Effluent License Limits Prior to 1997

STP	BOD (mg/L)	TN (mg/L)	TP (mg/L)	License Volume (kL/d)
Wesburn	30	25	2	550
Yarra Junction	30	25	2	300
Woori Yallock	30	25	2	500
Monbulk	20	25	2	100
Symons Road	20	25	2	290
Ferres Road	20	25	2	180
Seville	30	25	2	205
Healesville	20	25	2	300
Bluegum Dr	20	25	2	300
Lilydale	30	25	2	10000
Brushy Creek	20	25	2	13200
Whittlesea	30	25	2	450
Craigieburn	40	25	2	3000

Table 7.2 Current Effluent License Limits

STP	BOD (mg/L)	TN (mg/L)	TP (mg/L)	License Volume (kL/d)
Upper Yarra	10	10	1	4300
Monbulk	10	10	1	100
Symons Road	10	10	1	290
Ferres Road	10	10	1	180
Seville	10	10	1	205
Healesville	10	10	1	1200
Lilydale	10	10	1	12000
Brushy Creek	10	10	1	15500
Whittlesea	10	10	1	450
Craigieburn	10	10	1	3000

discharge point. This requires a possible effluent license limit of 5 mg/L of BOD, 5 mg/L of TN and 0.1 mg/L of TP based on BAT in 2004 (Personal communication with Julie Baud of Yarra Valley Water Pty Ltd, 2001).

Although, the EPAVIC has claimed that the upgrade of 'Prior 1997' to 'current' effluent license limits has improved the overall water quality in Yarra River (EPA Victoria, 1999), the actual improvement may only be marginal and has not been properly assessed for its efficiency relative to the upgrade. Therefore, further setting of effluent license limits based on BAT in 2004 lack realism. Additional increases in effluent license limits beyond a certain level are known to further increase in operating costs but do not significantly improve receiving waterways (Arundel, 2000). Therefore, an investigation on the effectiveness in further setting of effluent license limits on STPs is conducted Sections 7.3.

7.3 Evaluation of EPAVIC Effluent License Limits

Although the overall water quality in Yarra River and its tributaries has improved as a result of progressively set stringent effluent license limits of STPs (EPA Victoria, 1999), these limits were not evaluated for the efficiency before their implementation. Therefore, an assessment of these effluent license limits, together with a likely effluent license limit representing 2004 proposed scenario, was conducted using YRWQM. The water quality obtained under these effluent licenses limits were compared with the water quality standard specified in SEPP (EPA Victoria, 1995b). The assessments were conducted using design low flow conditions. The 3 effluents license limits considered were:

- 'Prior 1997' effluent license limit
- 'Current' effluent license limit
- '2004' effluent license limit

As can be seen from Table 7.1, the 'Prior 1997' effluent license limits change from STP to STP in terms of BOD license limit, however, in this study, the 'Prior 1997' effluent license limit was assumed as 30 mg/L of BOD, 25 mg/L of TN and 2mg/L of P. The 'current' effluent license limit considered was the actual limit, which was 10 mg/L of BOD, 10 mg/L of TN and 1 mg/L of P. The likely effluent license limit for '2004' was 5 mg/L of BOD, 5 mg/L of TN and 0.1 mg/L of P as per discussion with Julie Baud of Yarra Valley Water Pty Ltd (Personal communication, 2001).

7.3.1 Low flow frequency analysis

The evaluation of effluent license limits was conducted for several river conditions, which uses different magnitude of flow and water quality concentrations of the river. In most countries, the 'worst' river condition is used to set the effluent license limit. This is because the 'worst' river condition represents the lowest assimilative capacity of the river, which produces a conservative result (Chadderton *et al.*, 1981 and Lence *et al.*, 1990). The river condition used to get the effluent license limits is termed critical or design conditions, and such conditions are a combination of low flow and high temperature. Therefore, in this study, a low flow frequency analysis was first conducted to obtain several low flows from historical daily flow records for use in the evaluation of various effluent license limits. Two methods have been used in the past to define low flow condition in rivers for setting the effluent license limits, namely flow duration curves (Jung and Bau, 1996) and low flow frequency analysis (Chadderton *et al.*, 1981; Reheis *et al.*, 1982 and Lence *et al.*, 1990). Both methods define the critical low flow condition by considering the lowest flow that occurs consecutively for N days for each year of record.

The flow duration curve is a curve that displays river flow and the percentage of times that flow equals or exceeded during the period of record. This percentage exceedance of flow is estimated from the standard plotting position formulae such as Weibull (1939), Blom (1958), and Cunnane (1978). The required low flow indices can then be obtained from the flow duration curve. The low flow index used in flow duration curve approach defines the flow corresponding to a certain probability of exceedance. For example, Q90(7) gives the 7-day low flow that exceeded 90% of the time (Smakhtin, 2001). This method is less commonly used in determining the critical flow for use in setting effluent license limits, because the frequency of low flow events found is not as precise as that found using the frequency analysis method (Jung and Bau, 1996). Therefore, the low flow frequency analysis method is used in this study.

The low flow frequency analysis method performs a frequency analysis on the lowest N day flows for each year of record, using standard probability distribution functions (PDFs). Weibull, Gumbel, Log-Pearson Type III (LPIII) and Log-normal distributions are the four commonly used PDFs in connection with low flow analysis (Smakhtin, 2001). The commonly used low flow index is the 7-day 10-year low flow (7Q10), which is defined as the lowest average flows that occur for a consecutive 7-day period at the recurrence intervals of 10 years (Smakhtin, 2001). Other types of indices such as 7Q5, 7Q2 have also been considered (Gu and Dong, 1998). As can be seen from the above indices, 7-day averaged flows seemed to be commonly used. By averaging flows over a week (7 days) can also eliminate fluctuations in streamflow variation (Male and Ogawa, 1984). The 10-year return period was found as the most economical in terms of wastewater treatment (William and Walker, 1968), as increasing return periods would cause low assimilative capacity which requires higher treatment of the effluent, while decreasing return periods would cause higher assimilative capacity which is less conservative. This may not be suitable if the design flow is exceeded frequently. In this study, the indices 7Q10, 7Q5 and 7Q2 were used.

A computer package called DFLOW (Rossman, 1990) was used to determine the design low flows in this study. The program uses the low flow frequency analysis method to compute design low flows for use in river water quality studies. It uses the Log-Pearson Type III (LPIII) distribution for low flow frequency analysis. The LPIII was used in DFLOW, since it can account for large variety of distribution shapes and has been widely used in streamflow frequency analysis (Rossman, 1990). Inputs required to run DFLOW are historic daily flow data and user specified low flow index, (e.g.7Q10).

The design low flows estimates (i.e. 7Q10, 7Q5 and 7Q2) were required for headwaters of the main Yarra River and its tributaries. Flows of the other reaches in the Yarra River were then computed within YRWQM through volume balance. The design flow at the Yarra River headwater remains at 0.1157 m^3/s (Section 4.4.5.1), since it is a regulated flow release from Upper Yarra Dam. The design low flows for all tributary headwaters were computed using DFLOW and are given in Table 7.3.

Table 7.3 Design Flows Determined Using Frequency Analysis (m^3/s)

Tributary	Gauging station used	$7Q_{10}$ (m^3/s)	$7Q_5$ (m^3/s)	$7Q_2$ (m^3/s)
Little Yarra River	229214	0.300	0.380	0.580
Woori Yallock Creek	229215	0.230	0.320	0.530
Watts River	229144	0.006	0.013	0.057
Olinda Creek	229602	0.004	0.008	0.029
Brushy Creek	229665	0.001	0.002	0.003
Watsons Creek	229608	0.004	0.006	0.013
Diamonds Creek	229618	0.016	0.019	0.027
Plenty River	229616	0.001	0.002	0.003
Darebin Creek	229611	0.015	0.035	0.130
Merri Creek	229149	0.027	0.039	0.075

7.3.2 Scenario development

Several scenarios as shown in Table 7.4 were developed to evaluate the 3 effluent license limits. These scenarios were based on combinations of design low flow conditions, STP discharge volumes and headwater conditions. As discussed in Section 7.3.1, three design low flow conditions were considered. Two cases of STP discharge volumes were also considered namely 100% and 60% of the design license volume of each STP. As discussed with Ms. Julie Baud of Yarra Valley Water Pty Ltd (personal communication, 2001), the STPs in Yarra River catchment never reached the full critical STP discharge volume limit, and in most cases discharged at about 60% of the STP discharge volume limit. The maximum headwater concentrations for all water quality constituents measured within the period 1992 to 1997 were considered as one of the worst scenarios. However, in most circumstances, this maximum headwater concentration did not occur, and therefore less critical headwater concentration value was considered for all water quality constituents with their 90 percentile (90ile) values. This value is still a reasonably higher water quality concentration with only 10% of the data.

Table 7.4 Scenarios Considered in Evaluation of Effluent License Limits

	Scenarios Considered							
	Design low flow events				Actual low flow events			
	A	B	C	D	(18/3/92)		(21/1/92)	
Design flow								
7Q10	√	√						
7Q5			√					
7Q2				√				
STP discharge volume at								
100% license volume	√	√			√		√	
60% license volume			√	√		√		√
Headwater concentrations								
Maximum	√							
90ile		√	√	√				
Actual					√	√	√	√

In total, 12 different combinations of design conditions can be generated from these design flows, STP discharge volume and headwater concentrations. However only 4 scenarios were arbitrarily selected from these 12 combinations and they were considered sufficient to demonstrate the water quality response.

In addition to design flow events, two low flow events listed in Table 3.9 were also considered. They were the events 18/3/92 and 21/1/92, which were the lowest and the highest flow event respectively in the 'lower range' flow group. These actual flow events can realistically indicate the response from the two STP discharge volumes. The headwater flows for both of these events were analysed with DFLOW and found to have a return period of greater than 1 in 2 years but less than 1 in 1 year. The event 18/3/92 generally has a lower headwater flow and higher headwater concentration than event 21/1/92.

As can be seen from Table 7.4, a total of 8 scenarios were considered for this investigation. The most critical river conditions are the combinations of low flows, high effluent discharges, and high headwater concentrations. Therefore, *Case A* is the most critical condition. Although *Case A* may not be realistic, the analysis of *Case A* will provide the response under critical conditions.

YRWQM was run using inputs related to these 8 scenarios and 3 effluent license limits (i.e. 24 runs) and compared water quality against the water quality standard of EPA Victoria (1999).

The effluent license limits are given in terms of BOD, TN and TP (Table 7.2). However, YRWQM requires all forms of N and P, as input for point source pollutants. Therefore, the disaggregation factors given in USEPA (1997b) were used to disaggregate TN and TP effluent license limits to their various forms. These disaggregation factors were checked with available data of STPs in the Yarra River catchment and found that they were generally consistent with the actual breakdown proportion for TN and TP. The effluent license limits for various forms used in this investigation are listed in Table 7.5.

Table 7.5 Effluent Limits for TN and TP Forms (mg/L)

	Effluent proportion (USEPA, 1997b)	'Prior 1997'	'Current'	'2004'
Org-N	NH3 *0.53	8.48	3.39	1.70
NH3	TN * 0.64	16	6.4	3.2
NO2	(TN*0.02)/2	0.25	0.1	0.05
NO3	(TN*0.02)/2	0.25	0.1	0.05
TN		**25**	**10**	**5**
Diss-P	0.7*TP	0.6	0.3	0.03
Org-P	0.3*TP	1.4	0.7	0.07
TP		**2**	**1**	**0.1**

7.3.3 Results and discussion

(a) Design Case A

Figure 7.1 shows the modelled TN, TP and DO water quality concentrations for the 3 effluent license limits. The water quality standard (or objective) required in 3 river segments (i.e. Upper, Middle and Lower Yarra) is also marked on each plot. As stated earlier, the design *Case A* is the most critical case out of all scenarios studied (i.e. lowest flow, highest STP discharge volume and highest headwater concentration). As expected, the highest water quality concentration in terms of TN and TP was produced from 'Prior 1997' effluent license limit followed by the 'current', and the least from the '2004'. None of the 3 effluent license limits achieved the water quality standard except in a small section of the Upper Yarra segment.

In general, DO concentration does not vary greatly along the Yarra River and has achieved the required water quality standard in all segments, even under this worst river condition. The effect of different effluent license limits on DO concentration was insignificant, as shown in Figure 7.1. This indicates that the stringent effluent license limits do not effect the DO in the Yarra River, although there is a significant difference in TN and TP. Furthermore, the insignificant changes in DO concentrations from different effluent license limits show high assimilative capacity of the river.

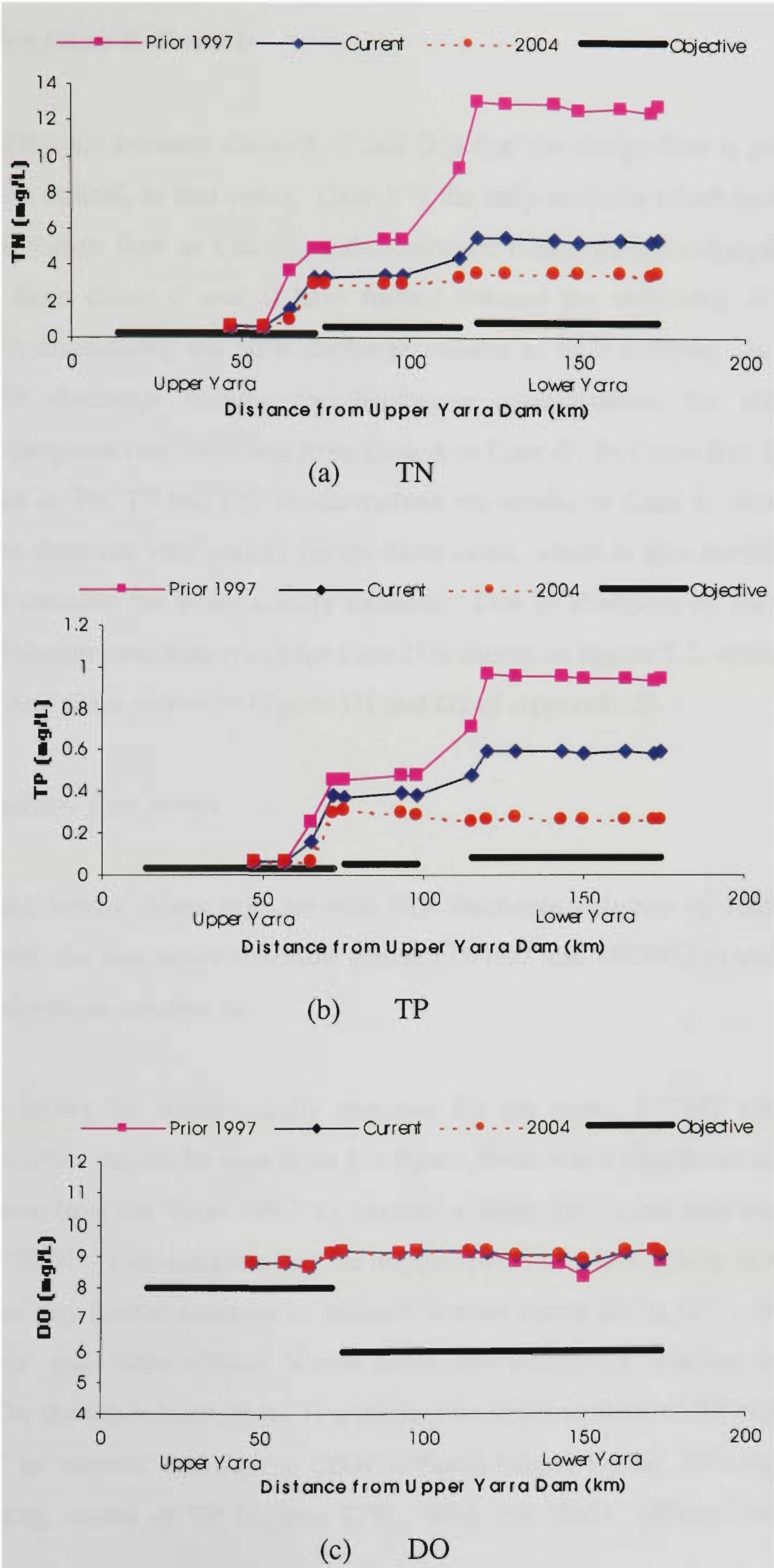


Figure 7.1 Water Quality Comparisons for Design Case A

(b) Design Cases B, C and D

The main difference between *Cases B, C and D* is that the design flow is progressively becoming less critical, in that order. *Case B* is the only scenario which has almost the same critical design flow as *Case A*, with headwater concentration reduced to the 90^{th} percentile. Both *Cases C and D* have further reduced the sensitivity of the design conditions by considering the 60% discharge volume as STP outflow. As a result of reduced STP discharge volume and headwater concentration, the water quality condition is progressively improved from *Case A to Case D*. In *Cases B to D*, the water quality trends in TN, TP and DO concentrations are similar to *Case A*. Generally, DO concentration does not vary greatly for the three cases, which is also similar to *Case A* and they all satisfied the water quality standard. Due to similarity on the results, the least critical design condition result for *Case D* is shown in Figure 7.2, while the results for *Cases B and C* are shown in Figures D1 and D2 of Appendix D.

(c) Actual low flow events

Three effluent license limits together with STP discharge volumes of 100% and 60% were used with the two actual low flow events (21/1/92 and 18/3/92) to study the river response under these conditions.

Figures 7.3 shows the water quality response for the event 21/1/92 with full STP discharge volume. As can be seen from this figure, there was a significant improvement in TN response from the 'Prior 1997' to 'current' effluent limits, but only marginal from 'current' to '2004'. This suggests that the improvement on water quality in terms of TN is limited for any further increase of effluent license limits (from the 'current' limit). Both 'current' and '2004'effluent license limits can satisfy the required water quality standard. On the other hand, there is a distinctive improvement of TP response from 'Prior 1997' to 'current' and then to '2004' effluent license limits. This suggested that the dominating source of TP is from STP. With the '2004' effluent license limits,

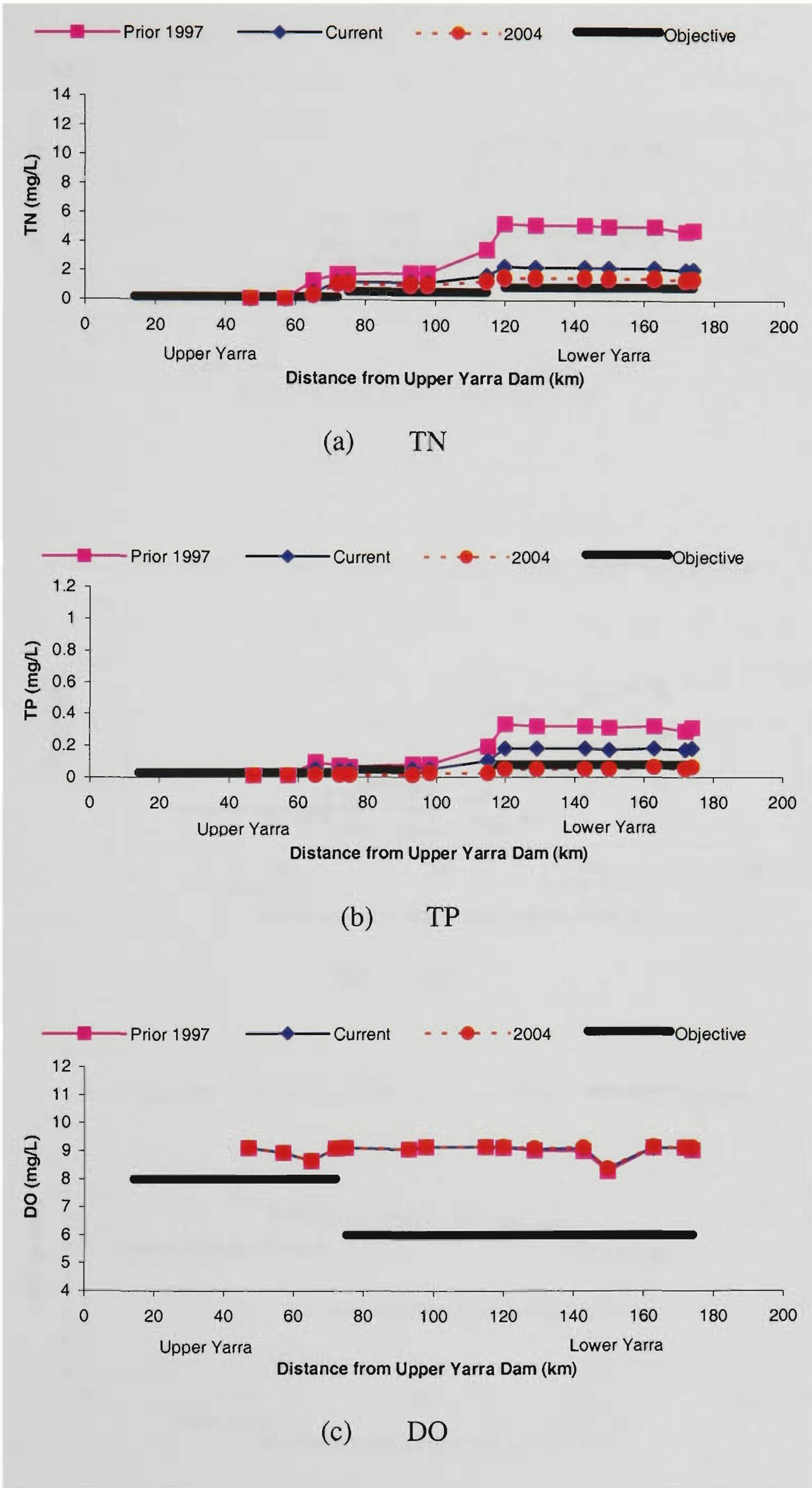
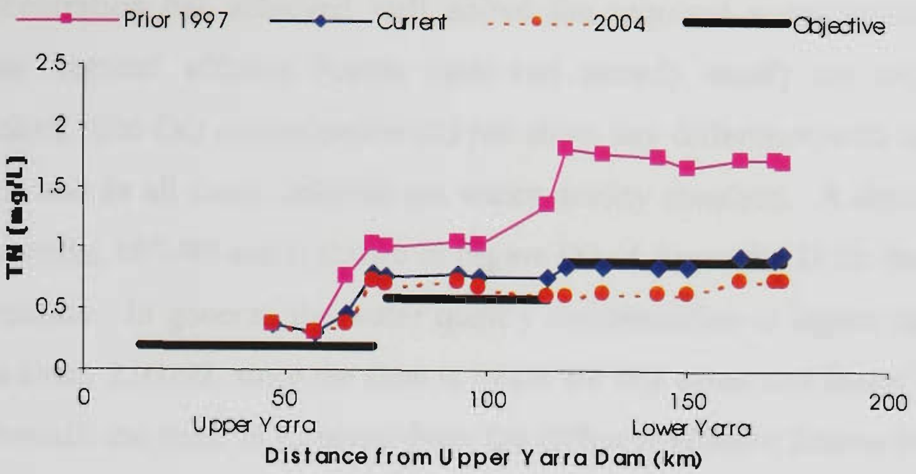
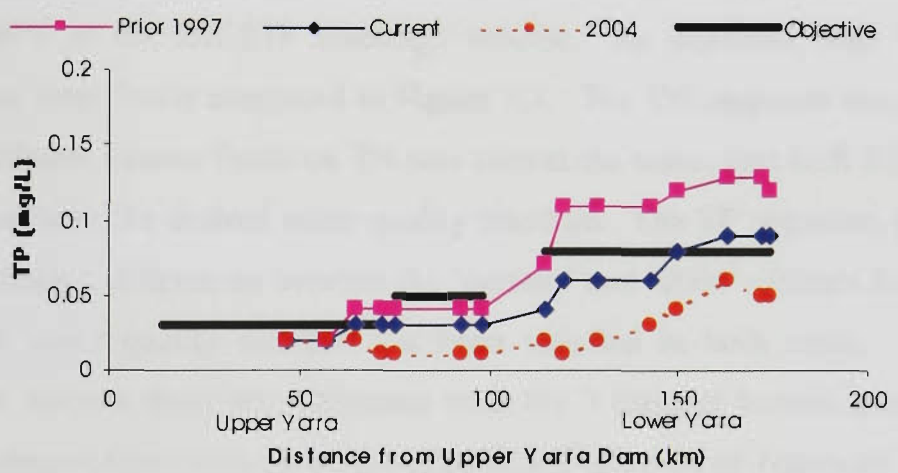


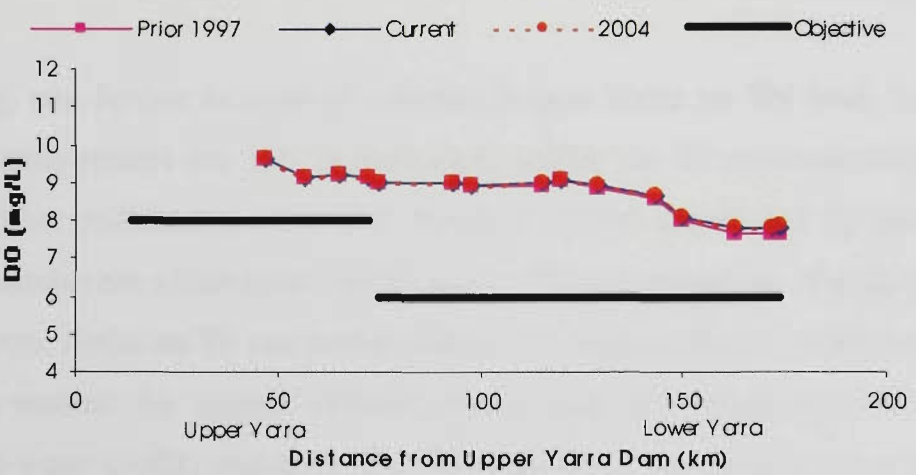
Figure 7.2 Water Quality Comparison for Design Case D



(a) TN



(b) TP



(c) DO

Figure 7.3 Water Quality Comparisons for Event 21/1/92 with 100% STP Discharge Volume the TP concentration has achieved well above the required water quality standard, however, the 'current' effluent license limit can already satisfy the required water quality standard. The DO concentration did not show any difference with the 3 effluent license limits and in all cases satisfied the water quality standard. A similar plot was produced for event 18/3/92 and is shown in Figure D3 of Appendix D for the 100% STP discharge volume. In general, the water quality concentration is higher for this event compared to event 21/1/92, since the flow is lower for this event and hence less dilution capacity. Overall, the trend of response from the different effluent license limits for this event was very similar to event 21/1/92.

Figure 7.4 shows the results for event 21/1/92 but with the STP discharge volume reduced to 60% of the full STP discharge volume. As expected, both TN and TP concentrations were lower compared to Figure 7.3. The TN response due to 'current' and '2004' effluent license limits on TN was almost the same, and both STP discharge volume can achieve the desired water quality standard. The TP response, on the other hand shows distinct differences between the 'current' and '2004' effluent license limits, however, the water quality standard has been satisfied in both cases. Again, DO concentration did not show any difference with the 3 effluent license limits. Similar results were obtained for event 18/3/92, as shown in Figure D4 of Appendix D.

7.3.4 Summary of evaluation of EPAVIC effluent license limits

In summary, any further increase in effluent license limits on TN from 'current' does not significantly reduce the TN concentration, unlike the TP concentration. This may be due to other pollutant sources (e.g. benthos matter) dominated by other activities around the catchment which have contributed to TN concentration. Further increases of effluent license limits on TP can further reduce TP concentrations in the river, however, for all cases studied, the 'current' effluent license limit was already capable of achieving the required water quality standard. DO concentration was not affected under different effluent license limits, which suggested that the assimilative capacity of Yarra River is high.

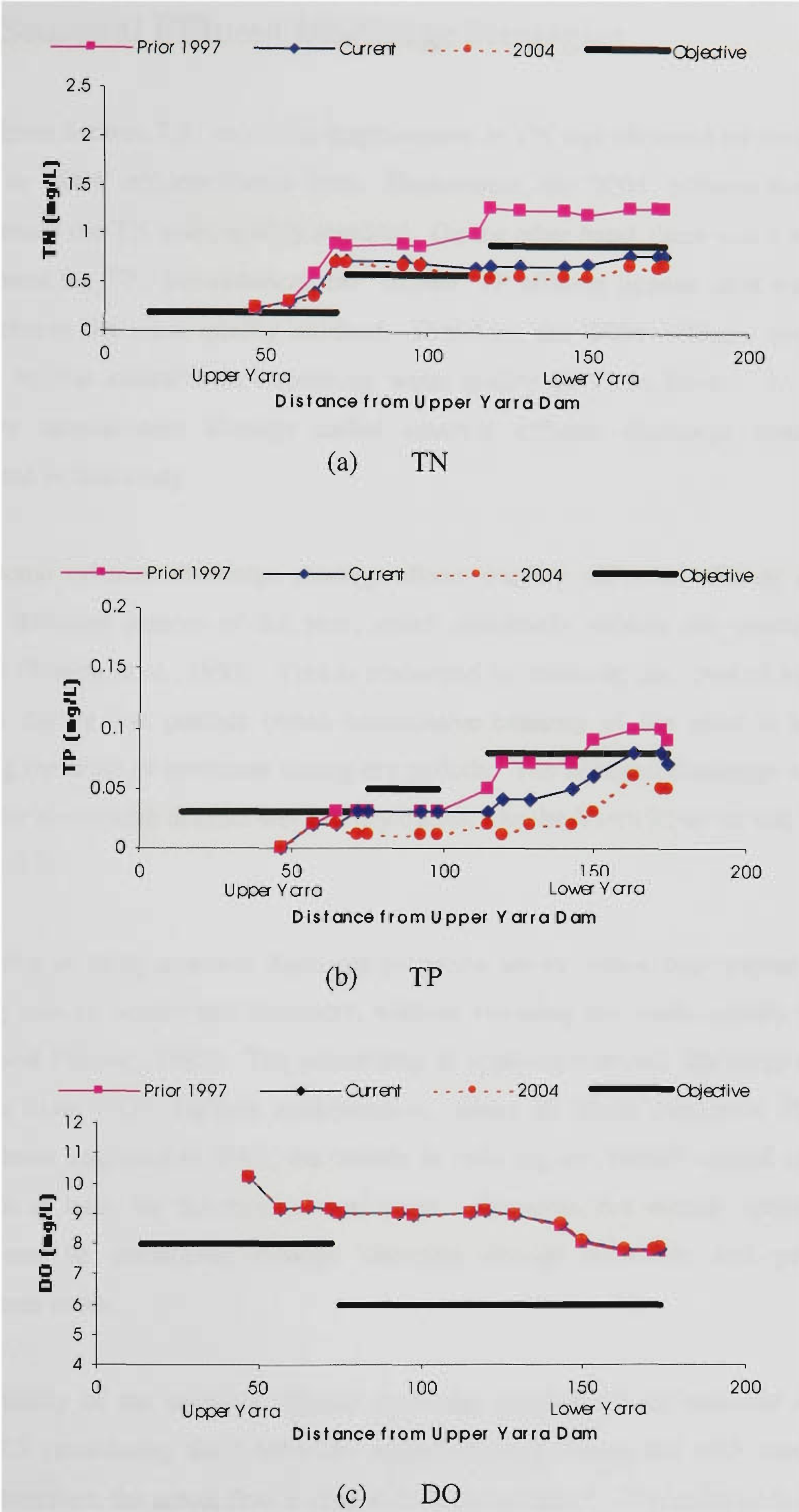


Figure 7.4 Water Quality Comparisons for Event 21/1/92 with 60% STP Discharge Volume

7.4 Seasonal Effluent Discharge Strategies

As seen from Section 7.3, very little improvement in TN was obtained by moving from 'current' to '2004' effluent license limit. Furthermore, the '2004' effluent license limit did not satisfy the TN water quality standard. On the other hand, there was a significant improvement for TP. Nevertheless, the 'current' TP effluent license limit was already able to achieve the water quality standard. Therefore, the '2004' effluent license limit may not be that effective in improving water quality in Yarra River. As such, an alternative management strategy called seasonal effluent discharge strategy was investigated in this study.

The seasonal effluent discharge strategy allows varying different effluent discharge limits in different seasons of the year, which effectively reduces the overall cost of treatment (Reheis *et al.*, 1982). This is performed by reducing the level of wastewater treatment during wet periods (when assimilative capacity of the river is high) and increasing the level of treatment during dry periods. The seasonal discharge strategy is feasible for rivers with distinct wet and dry cycles, like the Yarra River as was shown in Section 3.3.2.

The benefits in using seasonal discharge programs are to reduce both capital cost and operating cost in wastewater treatment, without violating the water quality standards (Ferrara and Dimino, 1985). The practicality of applying seasonal discharge programs for Yarra River STPs requires consideration. Since all Yarra catchment STPs have recently been upgraded to BAT, the benefit in reducing the overall capital cost is not applicable at least for the next several years. However, the overall operating cost savings can be considered through chemical dosage reduction and power and maintenance costs.

The feasibility of the seasonal effluent discharge programs were assessed similar to Section 7.3 considering the 3 EPAVIC effluent license limits, but with seasonal low flows. Therefore, the actual flow events were not considered. The analysis is discussed in Sections 7.4.1-7.4.3.

7.4.1 Seasonal flow frequency analysis

Since the streamflow data of Yarra River and its tributaries showed a clear difference between flows in dry and wet periods (Section 3.3.2), these two seasonal flows were considered in seasonal discharge program analysis. The periods considered for dry and wet flows were:

- November to June for dry period
- July to October for wet period

Design low flows for respective dry and wet periods were determined as in Section 7.3.2 using DFLOW software package. The design seasonal low flows are given in Table 7.6.

Table 7.6 Seasonal Design Flows

Tributary	Gauging station used	Dry season (m^3/s)			Wet season (m^3/s)		
		$7Q_{10}$	$7Q_5$	$7Q_2$	$7Q_{10}$	$7Q_5$	$7Q_2$
Little Yarra River	229214	0.223	0.287	0.444	0.524	0.605	0.779
Woori Yallock Creek	229215	0.205	0.256	0.371	0.955	1.006	1.149
Watts River	229144	0.002	0.008	0.022	0.138	0.202	0.395
Olinda Creek	229602	0.003	0.005	0.007	0.014	0.023	0.033
Brushy Creek	229665	0.001	0.001	0.002	0.001	0.002	0.003
Watsons Creek	229608	0.003	0.005	0.013	0.005	0.008	0.013
Diamonds Creek	229618	0.012	0.016	0.023	0.019	0.023	0.033
Plenty River	229616	0.001	0.001	0.001	0.005	0.007	0.017
Darebin Creek	229611	0.011	0.023	0.093	0.023	0.041	0.144
Merri Creek	229149	0.016	0.022	0.037	0.046	0.052	0.088

7.4.2 Generation of design conditions

Similar to Section 7.3.2, several scenarios were considered to evaluate the 3 effluent license limits (i.e. 'Prior 1997', 'current' and '2004') and compare the response of Yarra River under these effluent license limits. These design scenarios are shown in Table 7.7.

Table 7.7 Scenarios Considered in Seasonal Effluent Discharge Program

	Dry season			Wet season		
	D1	D2	D3	W1	W2	W3
Design flow						
7Q10	√			√		
7Q5		√			√	
7Q2			√			√
STP discharge volume at						
100% license volume	√			√		
60% license volume		√	√		√	√
Headwater concentrations						
Maximum	√			√		
90ile		√	√		√	√

As can be seen from Table 7.7, there are 6 scenarios. YRWQM was run using inputs for these 6 scenarios and 3 effluent discharge limits (i.e. total of 18 runs) and compared water quality against the water quality standard of EPA Victoria (1999).

7.4.3 Results and discussion

The results and the discussion are presented in the following sections to compare the effect of river water quality response under dry and wet design flow conditions, keeping other variables (i.e. STP discharge volumes and headwater concentrations) the same.

(a) Scenarios D1 and W1

Figures 7.5 and 7.6 show the predicted TN, TP and DO water quality concentrations due to the 3 effluent license limits for dry and wet period design flows respectively. This is the most critical case out of all scenarios under investigation. As expected, TN and TP concentrations have improved under wet season flows for all 3 effluent license limits compared to the dry season flows, because of the effect of dilution. This shows a lower treatment can be used under wet conditions as compared to dry condition. The DO concentration was practically the same for both seasons and under different effluent license limits. The water quality standard for TN was not achieved even with the '2004' effluent license limit during the wet period in spite of higher dilution flows. The TP water quality standard was achieved with the '2004' effluent license limits during both dry and wet seasons.

(b) Scenarios D2 and W2

The predicted TN, TP and DO water quality concentrations due to the 3 effluent license limits for scenarios D2 and W2 are shown in Figures D5 and D6 of Appendix D respectively, with very similar trend to case (a). As expected, the water quality concentration for TN and TP is better than in case (a) due to design condition is less critical. The TN concentration from 'current' and '2004' effluent license limits is very similar under wet season flows in the Middle Yarra River. This implies that the water quality concentration in the Middle Yarra River using the '2004' treatment level can be achieved under the wet condition using the 'current' treatment level.

(c) Scenarios D3 and W3

The predicted TN, TP and DO water quality concentrations based on the least critical design condition for dry and wet season are shown in Figures D7 and D8 of Appendix D respectively. Similar to case (b), the TN concentration produced from the 'current' and '2004' effluent license limits is approaching the same concentration during wet periods in the Middle Yarra region. The 'current' effluent license limit of TP being discharged under wet periods can already satisfy the required water quality standard for

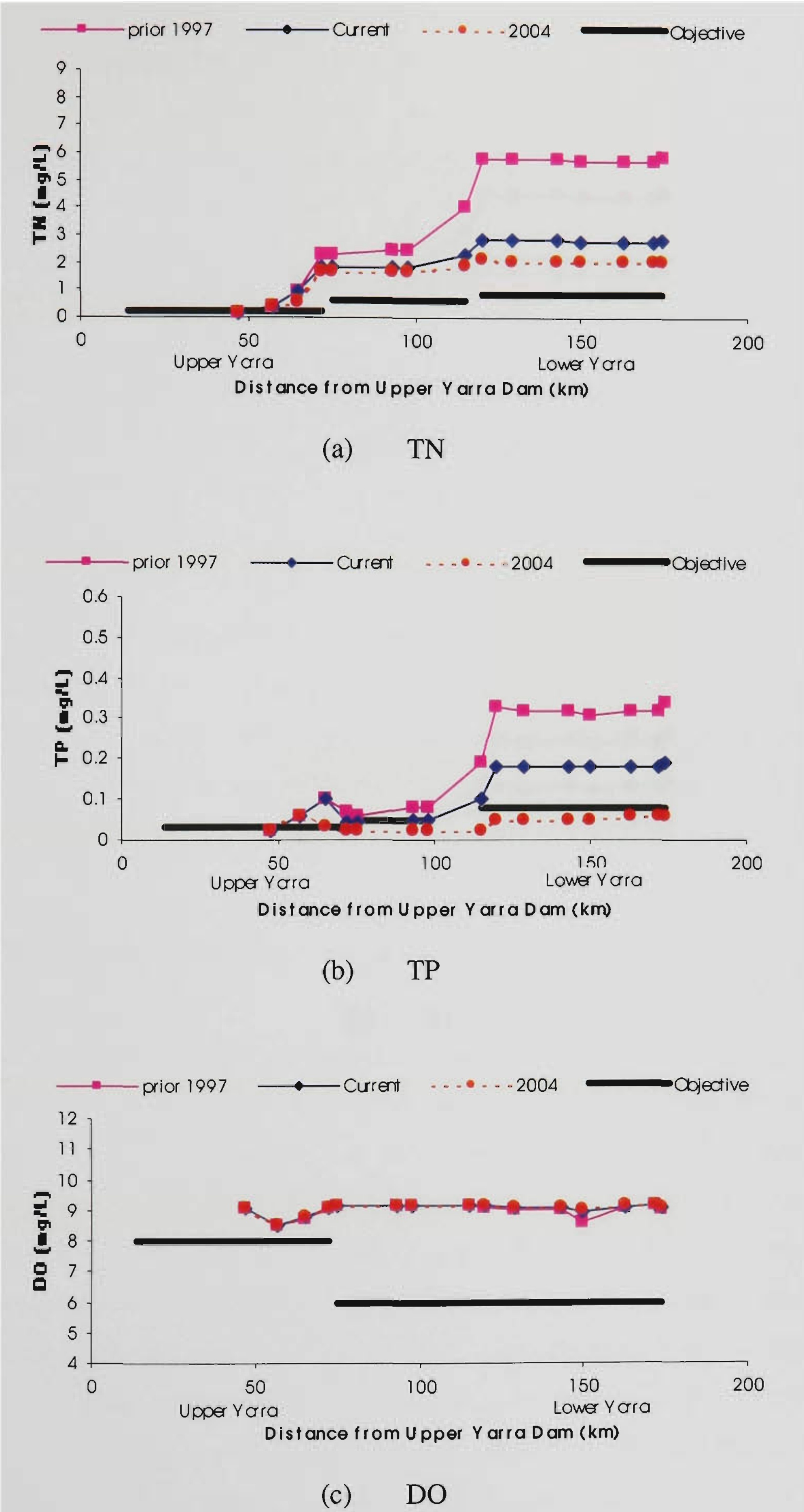


Figure 7.5 Water Quality Comparisons for Scenario D1 of Seasonal Discharge Program

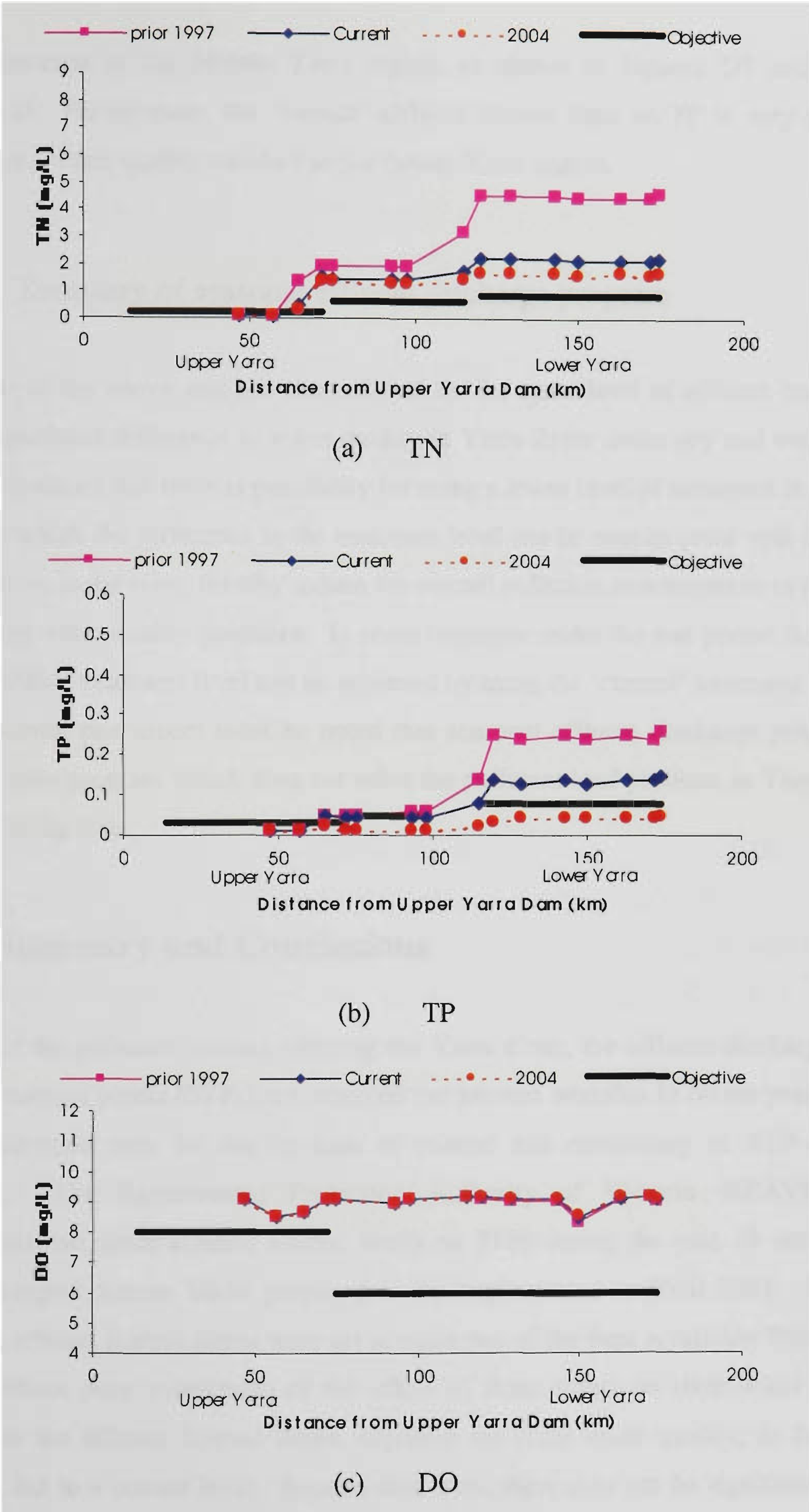


Figure 7.6 Water Quality Comparisons for Scenario W1 of Seasonal Discharge Program

TP concentration in the Middle Yarra region, as shown in Figures D7 and D8 of Appendix D. Furthermore, the 'current' effluent license limit on TP is very close in satisfying the water quality standard in the Lower Yarra region.

7.4.4 Summary of seasonal effluent discharge program

The results of the above analysis showed that for the same level of effluent treatment, there is significant difference in water quality in Yarra River under dry and wet period flows. This shows that there is possibility for using a lower level of treatment in the wet seasons in which the difference in the treatment level can be counteracted with the high dilution flows in the river, thereby reduce the overall pollution concentration in the river in satisfying water quality guideline. In some instances under the wet period flows, the effect of '2004' treatment level can be achieved by using the 'current' treatment level of TN. However, one aspect must be noted that seasonal effluent discharge program is only a dilution program which does not solve the pollutant load problem in Yarra River and Port Phillip Bay.

7.5 Summary and Conclusions

In terms of the pollutant sources entering the Yarra River, the effluent discharge from sewage treatment plants (STPs) has attracted the greatest attention in recent years. This focused attention may be due to ease of control and monitoring of STP effluent discharge. The Environment Protection Authority of Victoria (EPAVIC) has progressively set strict effluent license limits on STPs during the past 10 years, with further stringent license limits proposed to be implemented in Year 2004. In most cases, the effluent license limits were set to make use of the Best Available Technology (BAT) without prior assessment of the effect of these limits on river water quality. Increase in the effluent license limits improves the river water quality, as found by EPAVIC, but to a certain level. Beyond this level, there may not be significant water quality improvement. Therefore, the efficiency in managing point source pollution in

Yarra River using STP effluent license limits requires an investigation, and was studied in this chapter.

The Yarra River Water Quality Model (YRWQM) developed in this thesis was used to investigate the effect of different STP effluent license limits on river water quality. This was done by considering 3 effluent license limits, namely 'Prior 1997', 'Current' and '2004'. A number of design scenarios were generated to study the effect of these 3 effluent license limits. The design scenarios were developed using different design low flow events, different STP discharge volumes from STPs and different headwater concentrations. The design low flow events were determined using a standard low flow frequency analysis and was computed using DFLOW software (Rossman, 1990). The 100% and 60% of STP discharge volume were considered as the effluent volume from STPs for these scenarios. The maximum and 90^{th} percentile headwater concentrations were also used in development of design scenarios. In addition to these design scenarios, two actual low flow events were considered in this investigation.

The results obtained from this investigation showed that any increase from the 'current' effluent license limit on STPs generally does not improve total nitrogen (TN) and dissolved oxygen (DO) concentrations in the river, and only marginal improvement for total phosphorus (TP). Beyond the 'current' treatment level, the STP effluent discharges may not be the dominant pollutant sources responsible for TN (i.e. other pollution sources becomes more dominant) and this may be the reason for poor improvement in TN. The minimum difference on DO was due to high assimilative capacity of Yarra River. The dominating pollutant source of TP in Yarra River is from effluent as different levels of treatment (via different effluent license limits) did show a difference in river response for TP. Although further increase in effluent license limit from the 'current' limit improves the TP concentration, the effluent 'current' license limit could already satisfy (or very close to satisfying) the water quality guideline. By considering the results of this study in relation to TN, TP and DO, it can be said that additional increase in effluent license limit will not provide any significant improvement to water quality of Yarra River.

As identified, the assimilative capacity of Yarra River is high as indicated from the insignificant response from different effluent discharge. Therefore, an economically viable program known as seasonal effluent discharge program was investigated to assess whether it is a feasible alternative point source management strategy for Yarra River in the future. The seasonal effluent discharge program is a dilution program, which allows the discharge of effluent to vary with the season, utilising different assimilative capacities of the river, through dry and wet period flows. A similar study to the above investigation (i.e. the effect of different effluent license limits on river water quality) was conducted using YRWQM, but with dry and wet period design flows. The results showed that the water quality response was better during the wet season compared to dry season, which was to be expected. This allows a lower effluent treatment during high flows (generally in winter seasons) which can be counteracted with the high dilution capacity in the river, thereby reducing the overall pollutant concentration in the Yarra River. The seasonal effluent discharge program, being a dilution program, does not reduce the pollutant load. Therefore, the application of seasonal discharge programs to Yarra River STPs can be considered as a feasible management strategy in the future to reduce in pollutant concentration and operating costs.

CHAPTER 8

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary and Conclusions

The main aim of this thesis was to develop a well-calibrated water quality model for Yarra River and its tributaries using the Genetic Algorithm (GA) optimisation method. The developed model was named Yarra River Water Quality Model (YRWQM). The study involved the analysis of data, the assembly of the model, a pre-calibration uncertainty/sensitivity analysis of model parameters, the model calibration and verification, a post-calibration sensitivity analysis of model parameters, and finally the analysis of various point source pollutant management scenarios. A brief summary of each of these stages and the conclusions drawn from studies relevant to these stages are presented in the following sections.

8.1.1 Literature review

The following conclusions were drawn from the literature review conducted in this study.

From the detailed assessment of available public domain river water quality software, it was found that QUAL2E software was the most appropriate for use in the development of YRWQM. The major reason for this was the type of data available. The available data at sewage treatment plants (STPs) and water quality monitoring stations were collected mainly for regulatory purposes and were grab samples. At best, they can be represented as steady flow data, and therefore suitable for use in a steady state river water quality modelling tool, such as QUAL2E. The other reason was the purpose of development of the model. YRWQM was developed to study different 'what if' scenarios on point source management, such as different effluent license limits.

QUAL2E software can be effectively used for this purpose. It should also be noted that QUAL2E is credible and is considered as the standard river water quality software.

Many methods have been used in the past for model calibration ranging from trial and error manual methods to automatic optimisation methods. In recent times, there has been a wide use of automatic methods, since they provide some measure of objectivity and confidence in estimating model parameters than the traditional manual methods. Furthermore, it is not time-consuming for the modeller. However, the efficiency and the effectiveness of the automatic methods depend on the type of search algorithm used in the optimisation. Genetic Algorithm (GA) is one of those automatic optimisation methods, which has proven to be successful in optimising parameters in the water resource applications, and therefore considered in this study. However, limited applications exist in optimising parameters in river water quality models.

As a general rule, it is widely accepted that the efficiency of GA in achieving the 'optimum' parameter set depends on the proper selection of GA operators. These GA operators include parameter representation, population initialisation, selection of subsequent populations, crossover and mutation. Although broad guidelines are available for GA operators as applicable to any application, the guidelines are not available for specific applications such as river water quality model parameter optimisation. Few studies have attempted to study the importance of GA operators on water resource optimisation studies, but the results were inconclusive. Therefore, it was proposed that an investigation on this issue be undertaken in this study dealing with model parameter optimisation of river water quality models.

Uncertainty and sensitivity analysis of model parameters has been often considered together in the past, because it is necessary to identify the sensitivity in the output caused by the error in the inputs. As cited in the literature, many methods are available to conduct uncertainty/sensitivity analysis of model parameters and their effect on output results. Generally, these methods can be grouped into two groups, namely global and local. The global methods consider many possible values within the entire input parameter distribution for the analysis, whereas the local methods address the sensitivity of output responses for input parameters in the vicinity of the chosen parameter value. This chosen value could be the 'optimum' parameter set. The global methods can be used to identify the sensitive and insensitive model parameters, when the modeller does not know the likely values of the parameters for the model. This uncertainty/sensitivity analysis should be conducted prior to model calibration, so that more effort can be given to sensitive parameters during the calibration stage. This analysis was called pre-calibration uncertainty and sensitivity analysis in this thesis. The well-known Monte Carlo simulation (MCS) is the best global method available, which is also built into the QUAL2E software. Therefore, the MCS was proposed to be used in pre-calibration uncertainty/sensitivity analysis of model parameters in this study. It is often a good practice to check the sensitivity of the 'optimised' parameters after calibration so that the decision-makers can be more confident about the model and decision making using the results of the model. This analysis is called post-calibration sensitivity analysis in this thesis. Since the parameter values have already been 'optimised', only the parameters within a narrow band which deviates from the 'optimised' values need to be considered in this analysis. Based on the literature review, the 'one-at-a-time' method was considered the most suitable method for this purpose, which is also available in QUAL2E software.

Although many methods are available to interpret the output sensitivity results, the relative deviation ratio (RDR) was suggested for use in the pre-calibration uncertainty and sensitivity analysis because this index can be used to rank model parameters, which also requires the least output information from the simulation run. A plot of the difference in output water quality response versus the percentage perturbation from the 'optimised' value of each input parameter was used to interpret the output sensitivity in the post-calibration sensitivity analysis. This method does not rank the model parameters in terms of sensitivity to output responses, but shows the response of the sensitivity of each parameter away from its 'optimum'.

8.1.2 Yarra River data analysis

The Yarra River data analysis was conducted to develop YRWQM, and also to understand river flow characteristics and water quality processes in the river. It was found that distinctive flow patterns exist in the Yarra River. Low streamflows occur in the months from November to June, while high streamflows occur in the months from July to October. These distinctive flow patterns suggested that seasonal effluent discharge programs can be applied as future point source management strategies for Yarra River catchment STPs. The seasonal effluent discharge programs allow the discharge of effluent to vary with the season with different flows, utilising different river assimilative capacities.

Flow events were required to develop YRWQM, in particular to calibrate and verify the model, to perform uncertainty/sensitivity analysis of model parameters, and to estimate preliminary decay rates of water quality constituents for later use in model calibrations. They were also required to investigate various point source management scenarios for their effectiveness. Flow event selection was based on the available data on flow in the river and tributaries, effluent data from STPs and water quality measurements. Furthermore, since steady state river water quality model was developed in this study, it was considered that river flow and STP data should be fairly constant over a period of at least 3 days, which was the time of concentration of the catchment. Based on the above criteria, it was possible to select only 10 flow events and they were all found to be low flows, as expected. These events covered a reasonable flow range from 6.1 m^3/s to 40 m^3/s measured at the downstream end of the catchment. Six events were arbitrarily grouped into 'lower range' flows and the remaining 4 events into 'higher range' flows based on a critical flow of 10 m^3/s.

Nonconservative water quality constituents which include all forms of nitrogen (N), phosphorus (P) and dissolved oxygen (DO) were considered in this analysis of water quality. The major physical and biochemical processes were considered for the nonconservative water quality constituents in this study. These processes include microbial decay, settling, benthos uptake and reaeration.

Analysis of data showed certain relationships as a function of the magnitude of flows. The total nitrogen (TN) concentration decreased as temperature increased under both flow conditions. In general, the TN concentration was higher during 'higher range' flows compared to 'lower range' flows. The variation of total phosphorus (TP) between flow events was small, (perhaps) due to adsorption of soil particles to phosphorus. As temperature increased, the DO concentration decreased for 'lower range' flows, whereas the DO concentration was relatively constant for 'higher range' flows.

8.1.3 Assembly of YRWQM

YRWQM was assembled using the QUAL2E software, and requires the discretisation of river reaches. In total, 29 reaches (which include 5 tributary reaches) were considered from Upper Yarra Dam (UYD) to Dights Falls (DFS). Each reach was then subdivided into 1-km length computational elements, which were considered sufficient for this study.

Preparation of input data for six parameter groups (i.e. global, hydraulics, reaction, incremental, headwater, and point load) were required in QUAL2E. Data required in the global, hydraulics and reaction groups are not dependent on flow events, therefore they are constant for all 10 flow events (Section 8.1.2). On the other hand, the parameters for incremental, headwater and point load groups are dependent on flow events and require estimation from flow events. The default values obtained from the QUAL2E user's manual was used for the global parameter group, since they were found to be insensitive in YRWQM.

The power function method used in YRWQM to determine the depth and velocity for various flows in the hydraulic parameter group. The power functions were determined for 29 reaches using one of three methods depending on the availability of rating curves and cross-sectional details at river reaches.

The reaction parameter group consists of decay rates, which are estimated through calibration using GA. However, the GA parameter optimisation requires search spaces (or parameter ranges) for each of the decay rates. The available water quality data were used in the first order reaction rate equation to produce several estimates of decay rates relevant to N, P and DO at different river reaches with respect to 6 'lower range' flow events. These estimated decay parameters were then compiled as a range and was compared with the literature. It was found that the estimated decay rates lie within the range published in literature.

In addition to the decay rates, the reaeration rate is a parameter in the reaction group. However, the reaeration rates in YRWQM were determined using the QUAL2E built-in reaeration rate methods. The velocity-depth plot developed by Covar (1976) was used to select the most suitable reaeration rate method for use in YRWQM. Ten flow events were considered in obtaining velocities and depths of different reaches and they were plotted on the Covar plot. It was found from the Covar plot that O'Connor and Dobbins, and Owens-Gibbs were the two most suitable methods for use in YRWQM.

8.1.4 Pre-calibration uncertainty and sensitivity analysis

The pre-calibration uncertainty and sensitivity analysis of model parameters was conducted to identify the sensitive and insensitive parameters of YRWQM, so that more effort could be given to the sensitive parameters during the model calibration. The commonly used Monte Carlo simulation (MCS) was used in this study because it has the ability to consider many different input parameter sets sampled from their distributions, covering the whole range of parameters. The relative deviation ratio (RDR), which was defined as the ratio of coefficient of variation (CV) of the output water quality response to the CV of input model parameters, was used to define the sensitivity. RDR was computed for each combination of the output water quality responses and input model parameters. A threshold value of 1 was used to define sensitivity. When RDR>1, the input parameter was considered to be sensitive to the output water quality response and vice versa.

There were two parts to this analysis. In the first part, the sensitive parameter groups were identified: all six parameter groups of YRWQM (and QUAL2E), namely global, hydraulics, reaction, incremental, headwater and point load groups were considered in this part. The second part considered the sensitivity of decay rates as individual parameters in an attempt to identify the most sensitive decay rates. Several inputs were required to conduct the MCS uncertainty and sensitivity analysis, in both parts of the analysis, in addition to the flow events. One 'lower range' and one 'higher range' flow event was used in this study. The other inputs were the probability density function (PDFs) and statistics (i.e. mean and CV) of input parameters, and the number of parameter sets (or simulations) to conduct MCS. These information (i.e. PDFs and statistics of input parameters) were required to sample parameters from their distributions. The output responses considered were total kjeldahl nitrogen (TKN), TN, TP and DO.

Normal and lognormal are the two types of PDFs available in QUAL2E for parameter sampling. An investigation was first conducted by comparing the output cumulative frequency distributions (CFD) obtained with input parameters sampled from their PDFs. The MCS results showed that there were no differences in output responses due to these two PDFs. Therefore, it was concluded that either normal or lognormal PDF could be used for uncertainty and sensitivity analysis of model parameters of YRWQM. Hence, the normal PDF was selected for sampling model parameters in this study.

The mean values of each parameter of each group were obtained from particular flow events considered, except for global, hydraulics and reaction parameter groups. The mean values of global parameters were considered as the default values given in QUAL2E user's manual. The mean values of hydraulics parameter group were the derived power functions coefficients. On the other hand, the mean values of the reaction parameters (or decay rates) were obtained from the determined decay rates summarised in Section 8.1.3. Based on the selected reaeration method using Covar's method, the reaeration rates were obtained for all 10 flow events and the mean value was then determined.

Due to limited data available on Yarra River, the input CVs was unable to be determined. Therefore, 3 different CV values (i.e. the mean, minimum and maximum values of the typical CV range given in QUAL2E user's manual) were considered in pre-calibration uncertainty/sensitivity analysis. These CV values were considered to study the effect of input parameter CV on output results. Numerical experiments conducted using these input CVs showed that the output results were similar and therefore, it can be concluded that reasonable CV values published in the literature can be used in the uncertainty/sensitivity analysis. However, it should be noted that these conclusions are valid only for YRWQM and required verification if they are to be used with other river water quality models.

Selection of number of parameter sets to be used in MCS is subjective and depends on the accuracy required. In this study, 2,000 parameter sets were found to be adequate, which have also been commonly used in other studies.

The uncertainty and sensitivity analysis of the input parameter groups of YRWQM, revealed that the most sensitive parameter group to output water quality (i.e. TKN, TN, TP and DO) was the hydraulics group. The point load group was identified as sensitive for 'lower range' flows, while the incremental group was sensitive for 'higher range' flows. However, all these sensitive parameter groups (i.e. hydraulics, point load and incremental) have parameters that can be physically measured or estimated from measured data. Therefore, it can be concluded that the parameters in these groups should be accurately measured or estimated to reduce uncertainty of these parameters.

Although the reaction parameter group was not identified as a sensitive parameter group, it was still necessary to determine the most sensitive decay rates within the reaction parameter group for calibration purposes, since the decay rates were considered as model parameters in calibration. The initial temperature was also considered as a decay parameter in QUAL2E, since it modifies the reaction parameters to account for variation in temperature. After analysing the MCS results, it was found that the initial temperature had dominated the overall sensitivity in the reaction parameter group to all output water quality constituents (i.e. TKN, TN, TP and DO). This finding suggests that the initial temperature should be measured with greater accuracy to improve the calibration and to use YRWQM for analysis of management scenarios dealing with river water quality improvement. The decay rates of YRWQM were found to be insensitive to the output water quality responses.

8.1.5 Importance of GA operators

A detailed study was conducted to investigate the effect of GA operators on model parameter optimisation of YRWQM (and in general river water quality models) and to produce guidelines on the appropriate GA operator values for use in river water quality models. The approach adopted in this investigation was different from many previous studies. The significance of GA operators was first hypothesised as a function of the sensitivity of the parameters to output water quality. This hypothesis was studied with a sensitive and an insensitive hypothetical river network model using a set of GA operators compiled from the literature. A sensitive model was defined as a model where all decay rates were found to be sensitive to the output water quality responses based on RDR>1 of a MCS analysis, and vice versa. These two models were assembled with known decay rates and therefore the GA parameter optimisation can be aimed to achieve these actual parameters. It was found that the GA operator sets obtained from the literature were able to achieve the 'optimum' model parameter set for the sensitive model, but not for the insensitive model. This analysis proved that the GA operator sets could be a function of the sensitivity of the model parameters to output water quality, in optimising model parameters.

Since the hypothetical river network insensitive model was unable to provide the 'optimum' model parameter set using the literature GA operator set, a systematic procedure was undertaken to optimise the GA operators to achieve the 'optimum' model parameter set. It was then found that the optimised GA operator set was able to achieve the 'optimum' parameter set. However, it was also found that there were no significant differences of the water quality responses due to model parameters obtained from the literature and 'optimised' GA operator set. This is because of the insensitivity of model parameters to output water quality. Since the purpose of river water quality models is to predict the water quality in the river, it can be said that the GA operators have not played a significant role in predicting the water quality in the hypothetical river network models considered in this study.

The above finding was then verified with YRWQM, which was also found to be an insensitive model based on MCS results. The literature GA operator set and the above optimised set (optimised with respect to the hypothetical river network model) were then used with YRWQM. Unlike in the hypothetical river network models where the model parameters were known, it was not possible to compare the 'optimised' parameters with the actual parameters for YRWQM, since there were no actual parameters for YRWQM. However, as found with hypothetical river network insensitive model, the two GA operator sets produced different 'optimised' decay rates, yet the water quality responses were not significantly different.

Based on these limited numerical experiments, it was found that the use of GA in optimising decay rates of river water quality models could be done efficiently by selecting a robust GA operator set from the literature. These GA operator sets will provide 'optimum' parameter set for sensitive models, while they provide near-optimum parameters for insensitive models. Although the 'optimised' GA operator set can provide the 'optimum' model parameter set even for insensitive models, the amount of effort required to obtain the 'optimised' GA operator sets needs to be considered, because they do not contribute a great difference in the overall water quality prediction. Although the findings from this investigation were based on QUAL2E, this software uses standard river water quality advection-dispersion mass transport equation, which are also considered in other river water quality modelling software tools. Therefore it can be said that these conclusions may be extended for other software tools which have similar model structure to QUAL2E.

8.1.6 YRWQM calibration, verification and sensitivity

Three flow events were used in the calibration of YRWQM using GA, which produced three 'optimum' decay parameter sets. It was found that all three parameter sets were able to predict to the observed water quality with difference being insignificant at 95% significant level for all three flow events used in the calibration. It was also found that these decay rates were within the published range. The predicted water quality was similar under the three parameter sets, since the model parameters were insensitive to output responses. Therefore, the set with the least cumulative absolute relative error (CARE) value with respect to 3 calibration events and the mean of those 3 parameter sets was selected as the single 'optimised' decay parameter set. In selecting the single 'optimised' parameter set for YRWQM, the CARE index considered the absolute difference between the observed and the predicted water quality response of TKN, TN, TP and DO at the six water quality stations due to three calibration events.

The single 'optimised' parameter set obtained from the CARE index was then verified with 3 other independent events, which were not used in calibration. It was found that the 'optimised' parameter set was able to predict the observed water quality with difference being insignificant at 95% significant level. Therefore, YRWQM was verified.

To further enhance the confidence in YRWQM, a post-calibration sensitivity analysis was also undertaken, using 'one-at-a-time' sensitivity analysis method. It was found that deviations of upto 50% from the 'optimised' decay rates can still be able to predict river water quality, with difference between predicted and observed being insignificant at 95% significant level. Therefore, it can be concluded that YRWQM is a credible model which has been verified, and can be used confidently to analyse various management scenarios to improve water quality in the Yarra River and its tributaries.

8.1.7 Analysis of point source management scenarios using YRWQM

The calibrated YRWQM was used to assess the effectiveness of several effluent license limits as point source management strategies for Yarra River catchment, and also to investigate the feasibility of using seasonal discharge programs as future management strategies. A comparison of river water quality responses under several effluent license limits was undertaken. It was found that the increase in effluent license limits does not significantly improve TN and DO concentrations, but marginally improves the TP concentration. For all cases studied, the current effluent licenses limits on STPs produce very close to or has already satisfied the required water quality standard of the Environment Protection Authority of Victoria (EPAVIC). This means that further increase in effluent license limit as a point source management strategy in Yarra River STPs is not an effective solution.

The seasonal effluent discharge program is a type of innovative point source management strategy, which is aimed at utilising the seasonal effect of flow in river to discharge the effluent, thereby reducing the overall pollutant concentration in the river. In this study, an investigation was conducted by comparing river water quality when the same level of effluent is being discharged under dry and wet season design flows. For Yarra River catchment, the seasonal discharge program was found to be a feasible point source management strategy, since the river has distinct dry and wet period flows. Furthermore, it was found that with the same effluent treatment, the water quality was significantly better in wet period flows than in dry period flows. This shows that a lower effluent treatment can be considered during the wet periods, which can be counteracted with the high dilution capacity of the river, thereby reduce the pollutant concentration in the river. This will obviously reduce the operational costs associated with effluent treatment during wet period flows. However, one aspect must be noted that the seasonal effluent discharge program does not reduce the overall pollutant load in the river and Port Phillip Bay.

8.2 Recommendations

Based on the findings of this research project, several recommendations are suggested for future studies, as discussed below.

8.2.1 Data improvement

The STP and river water quality data used in this project were not collected for the purpose of developing a water quality model. They were collected for regulatory and compliance requirements. Therefore, they were not measured concurrently. In order to develop a water quality model, data should be concurrent. This was one of the reasons that only 10 flow events were selected in this study. The available data were satisfactory for a steady state model, as was done in this study. However, if high flows are to be considered together with non-point source pollution, then it is necessary to conduct a detailed sampling program, dedicated for developing an unsteady model for Yarra River. High flows are generally associated with storm events and they are unsteady.

As a recommendation, a three month (preferably in summer) intensive sampling program with regular observations during the 24-hour period is proposed to study the change in water quality and occurrence of reaction activities in the river. This is especially important to monitor the change in DO at day and night (diurnal effect), as well as other water quality reactions and processes occurring in the river column and bed. In addition, the decay rates measurement should be conducted through dye tracers.

8.2.2 GA operators

GENESIS provides only the basic (or traditional) methods for various GA operators and therefore, the study of GA operators in this thesis was limited to the capabilities of GENESIS. As a recommendation, it is proposed to modify (or enhance) the GENESIS code to allow for additional recent GA operators, such as

- Real coding system for parameter coding
- Additional crossover methods (e.g. uniform crossover)
- Additional mutation methods (e.g. non-uniform mutation)
- Additional selection and sampling methods (e.g. tournament)

These additional GA operators can further examine the importance of GA operators on river water quality model parameter estimation and will be able to provide further guidelines on the appropriate GA operator sets.

The investigation on the importance of GA operator sets in this study was conducted based on limited numerical experiments of two river networks. To substantiate the findings of this study in relation to sensitive/insensitive models, and the relationship between these and the GA operators, it is recommended that further studies should be conducted using different river settings and different water quality modelling software. Although the studies conducted in this thesis on the importance of GA operators are empirical, they provide useful results when such studies are done with a large number of river settings and different software tools.

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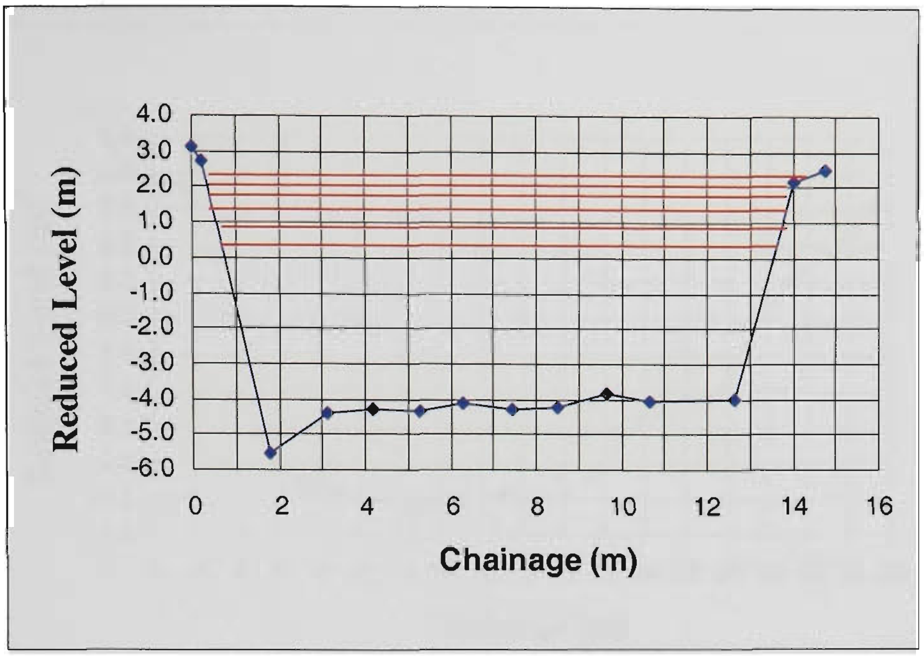
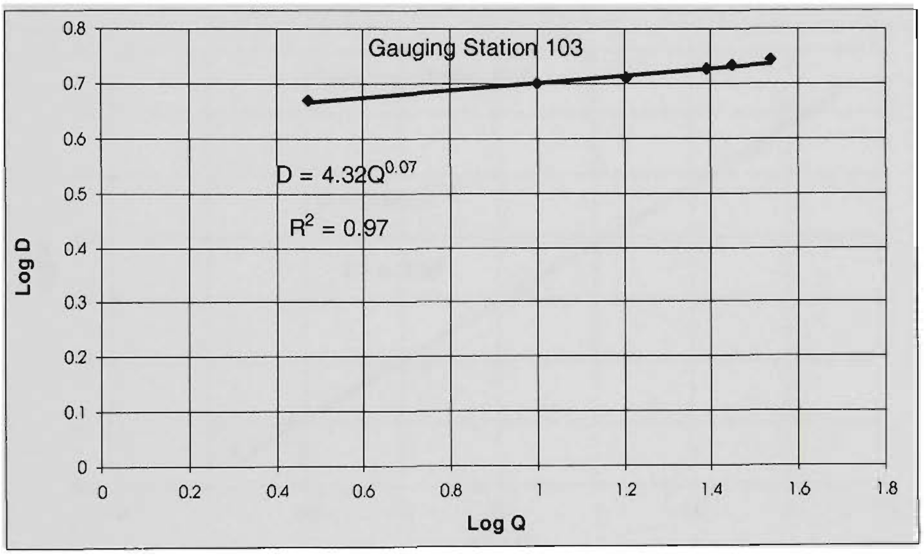
APPENDIX A1

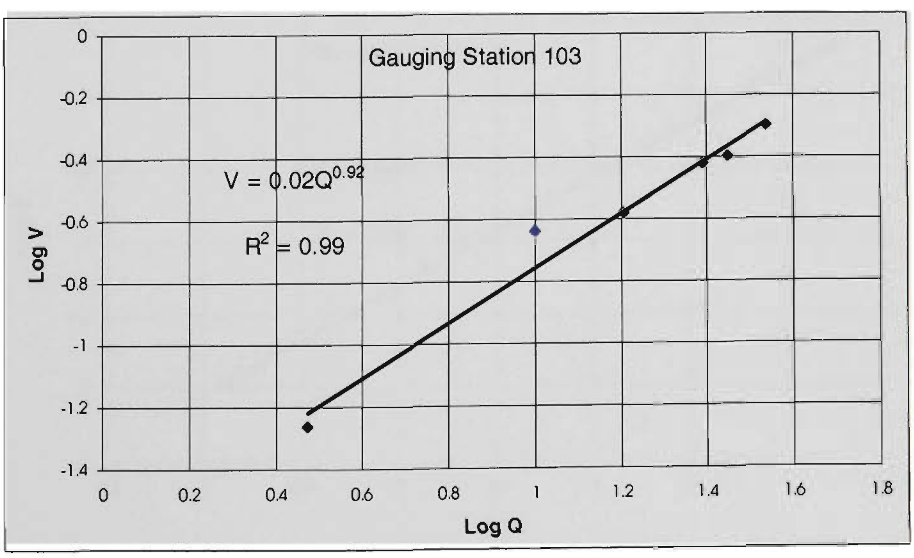
DERIVATION OF POWER FUNCTIONS FOR GROUP (a) REACHES



(a) Cross-Section Gauging Station 103

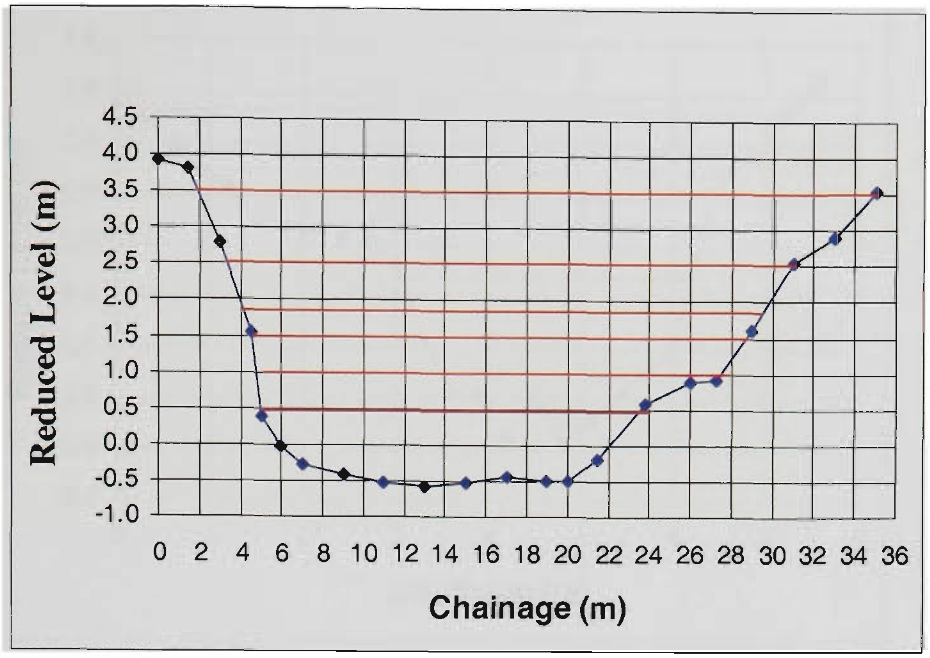


(b) D-Q Relationship

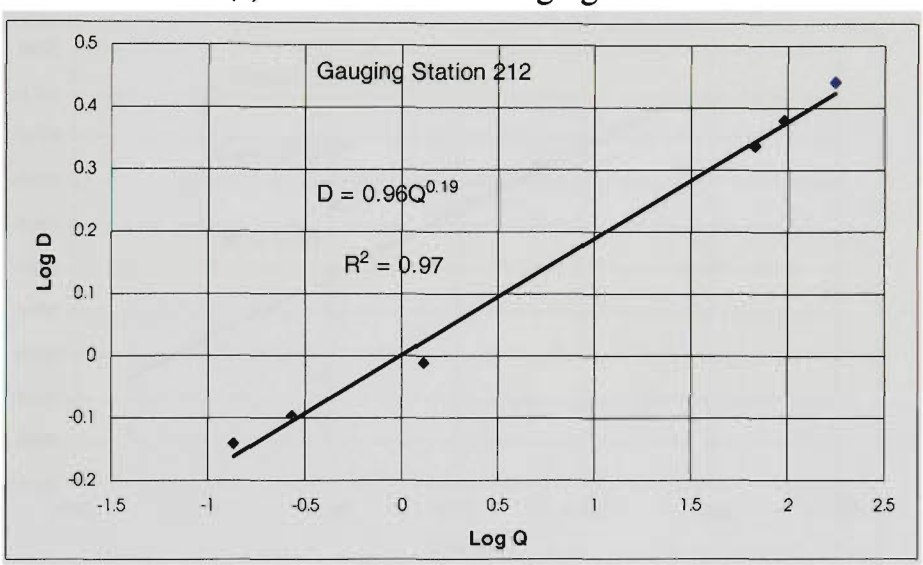


(c) V-Q Relationship

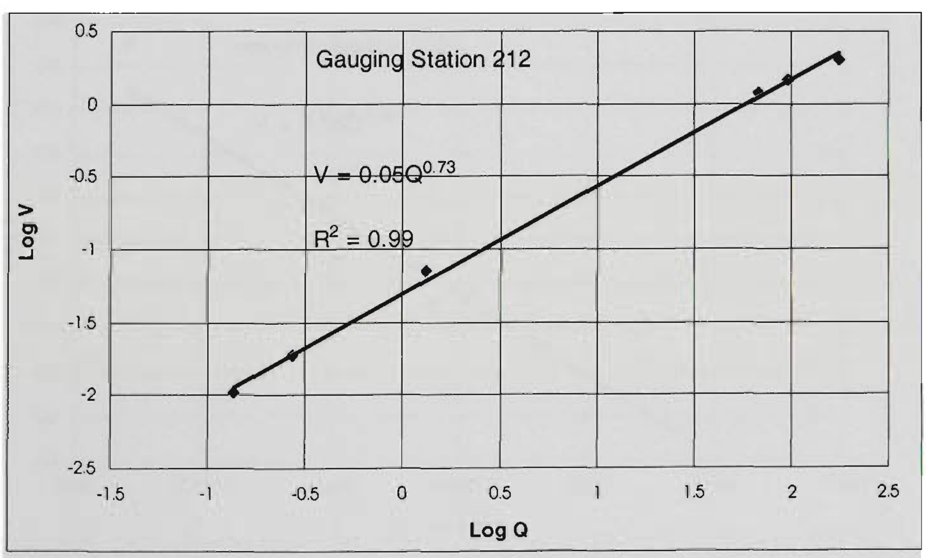
Figure A1-1 Cross-Section for Gauging Station 103
and Power Function for Reach 1



(a) Cross-Section Gauging Station 212

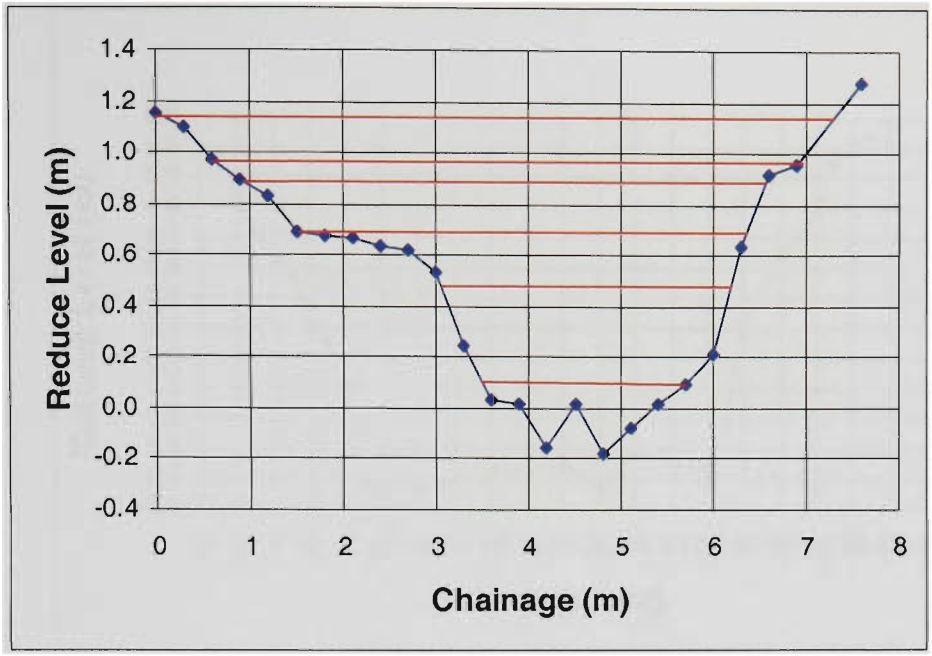


(b) D-Q Relationship

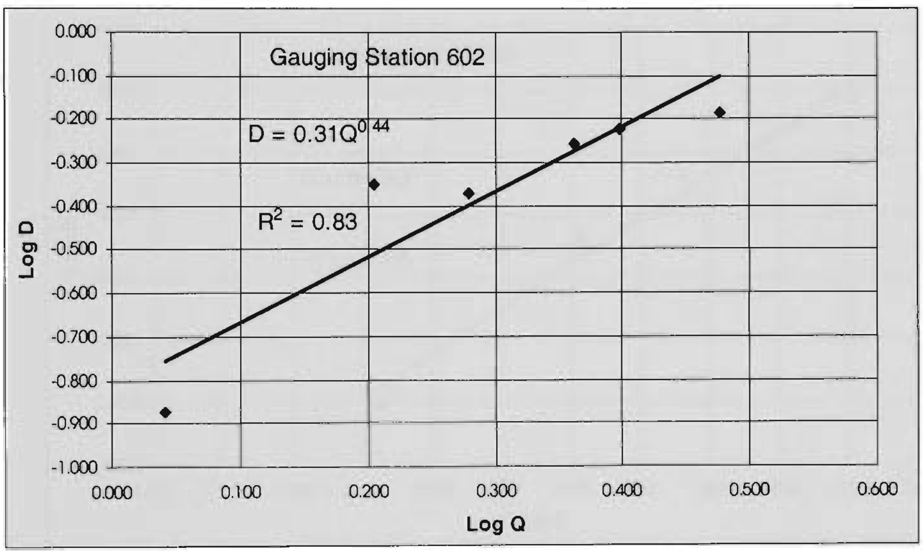


(c) V-Q Relationship

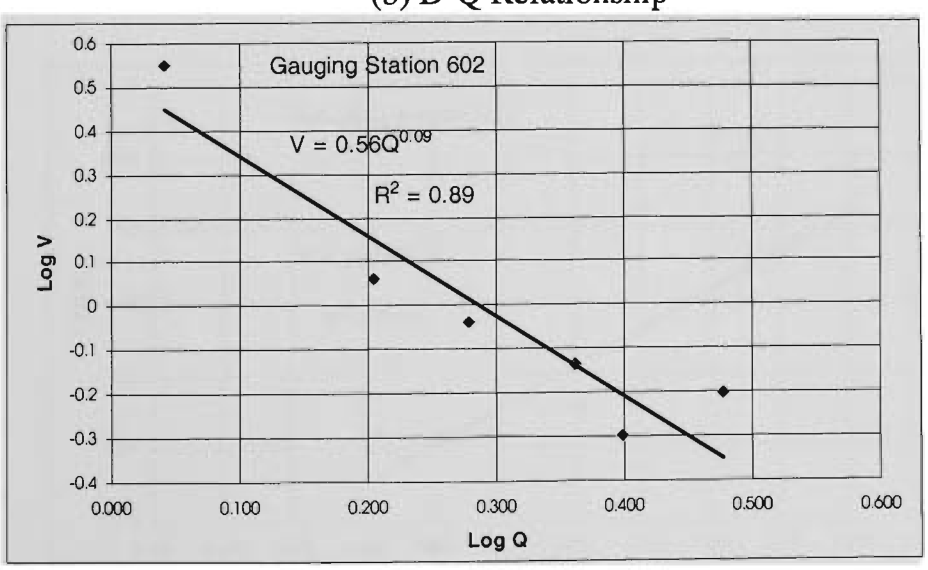
Figure A1-2 Cross-Section for Gauging 212 and Power Function for Reach 3



(a) Cross-Section Gauging Station 602

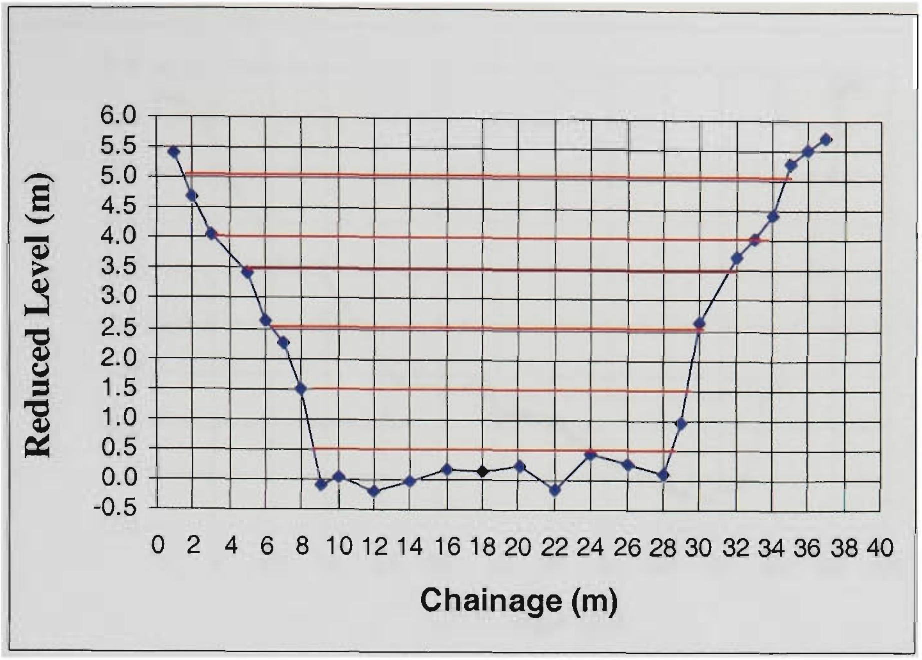


(b) D-Q Relationship

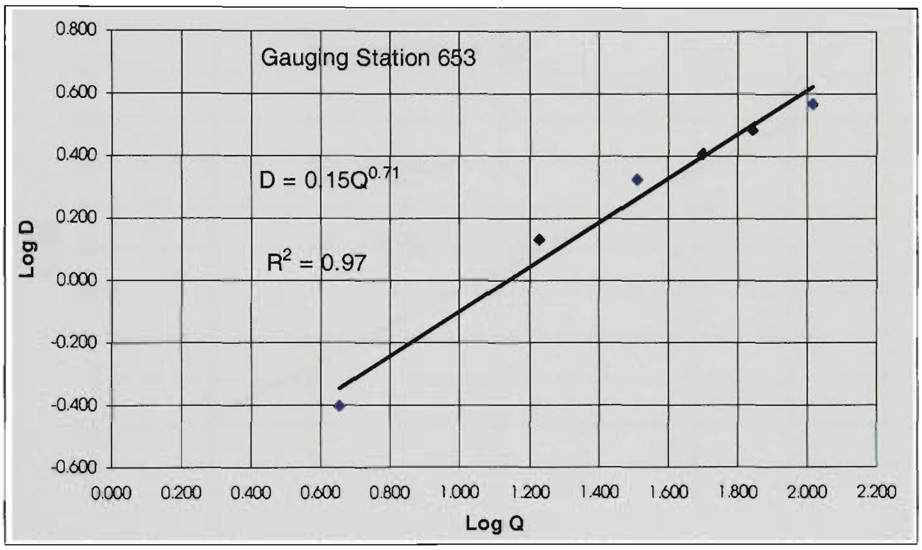


(c) V-Q Relationship

Figure A1-3 Cross-Section for Gauging Station 602 and Power Function for Reaches 7-9, 14 and 16



(a) Cross-Section for Gauging Station 653



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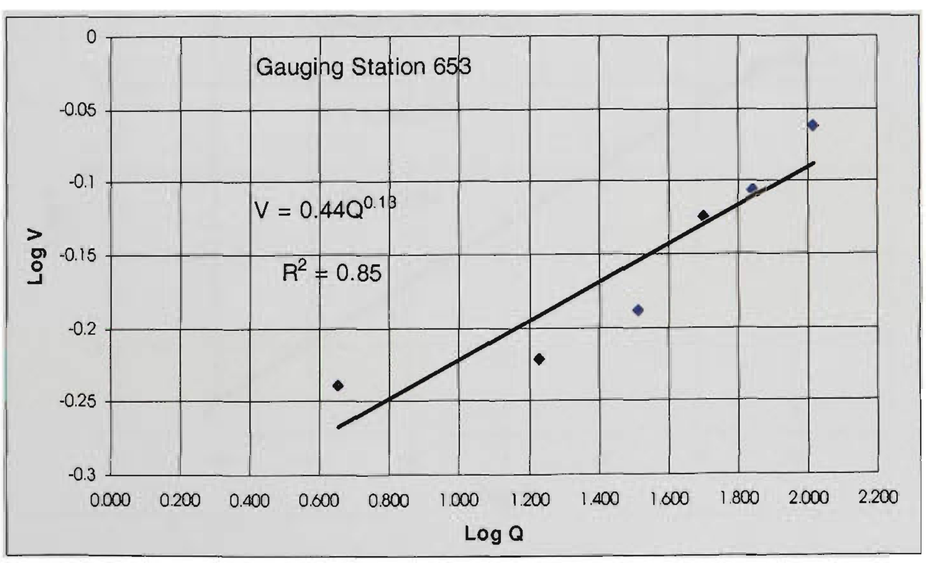
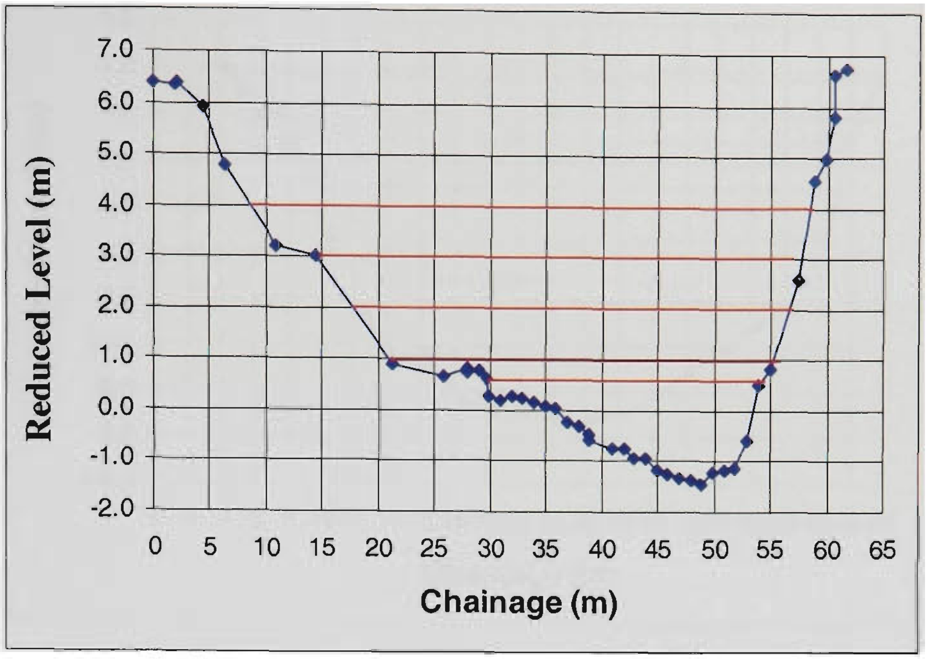
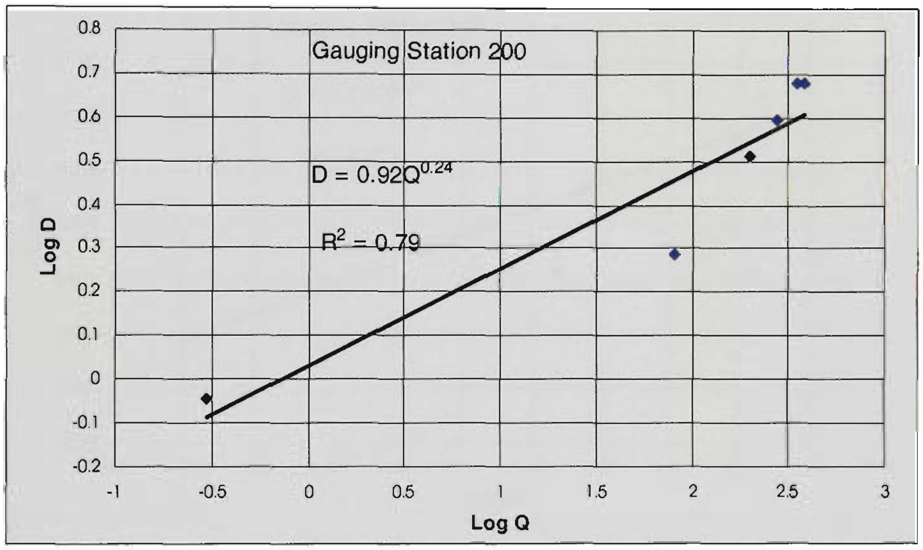


(c) V-Q Relationship

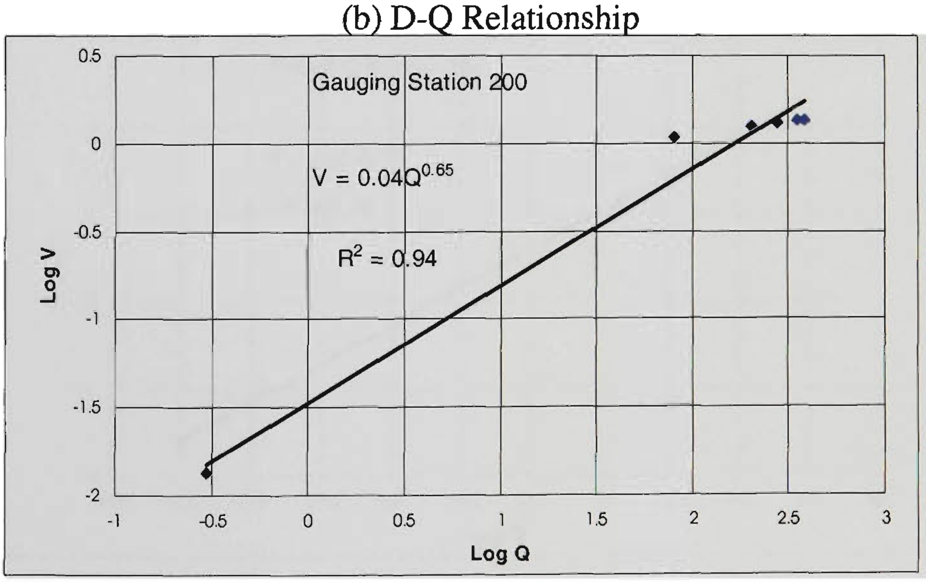
Figure A1-4 Cross-Section for Gauging Station 653
and Power Function for Reach 11



(a) Cross-Section for Gauging Station 200

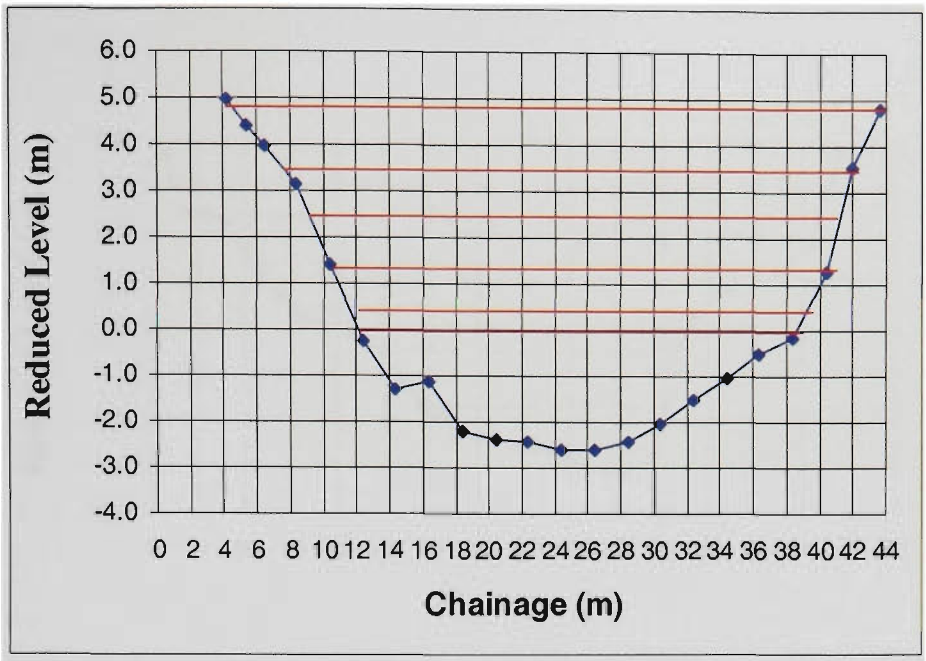


(b) D-Q Relationship

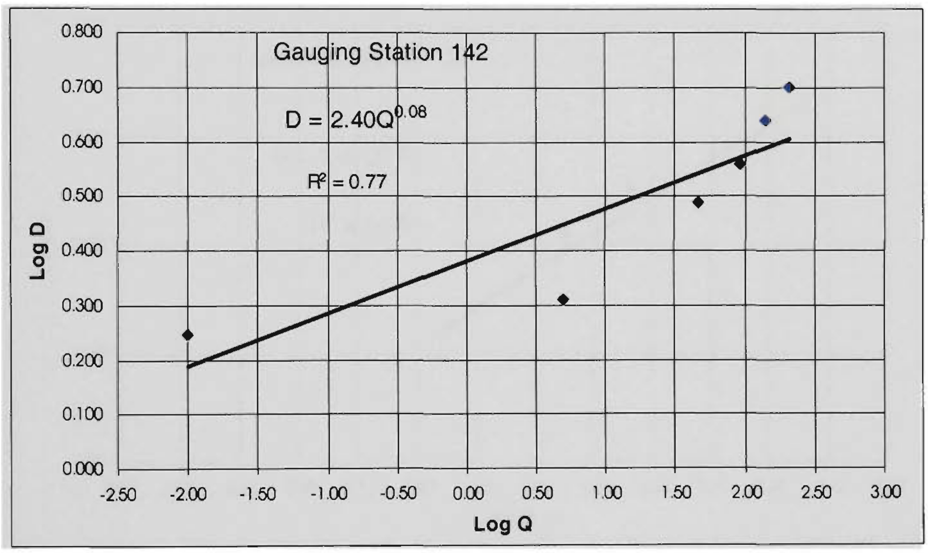


(c) V-Q Relationship

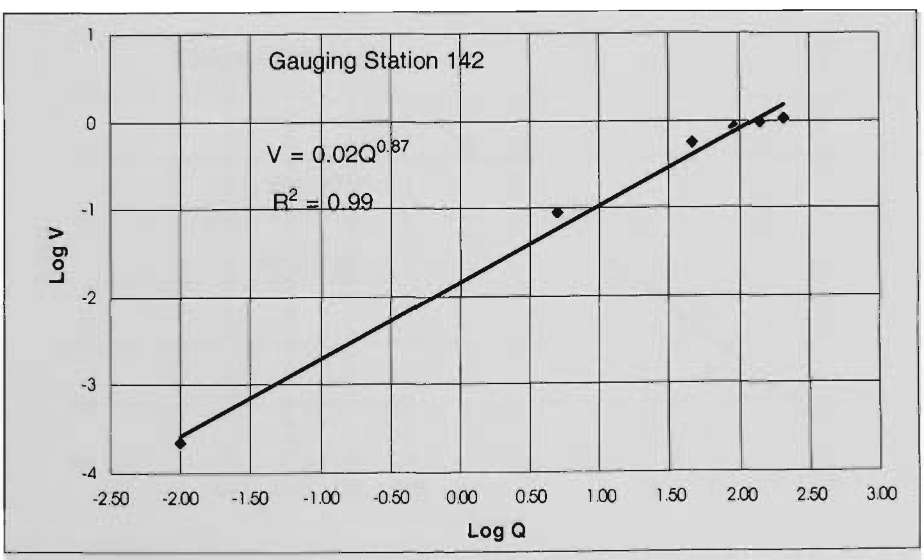
Figure A1-5 Cross-Section for Gauging Station 200
and Power Function for Reach 19



(a) Cross-Section for Gauging Station 142

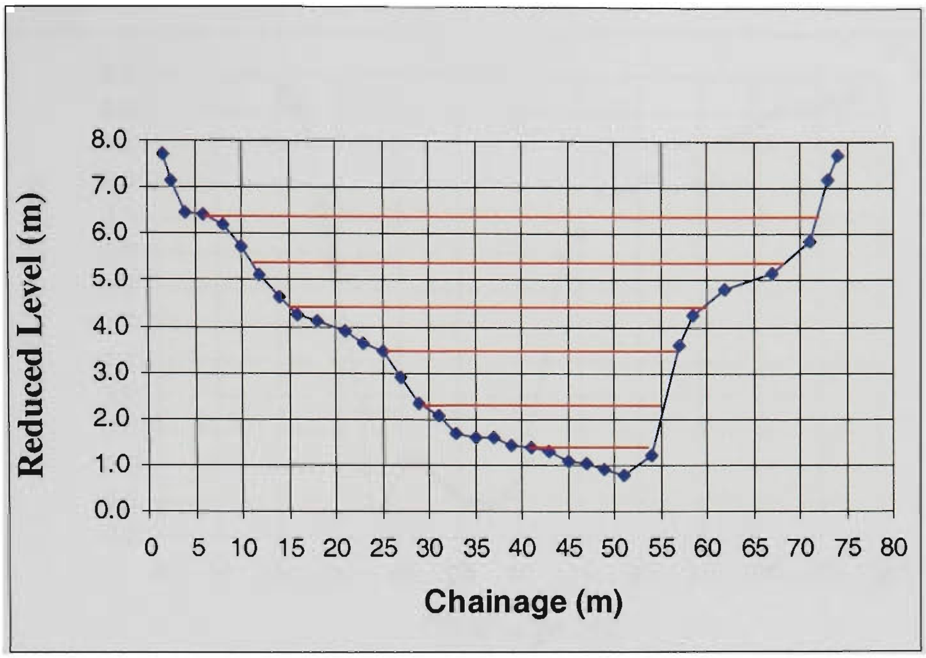


(b) D-Q Relationship

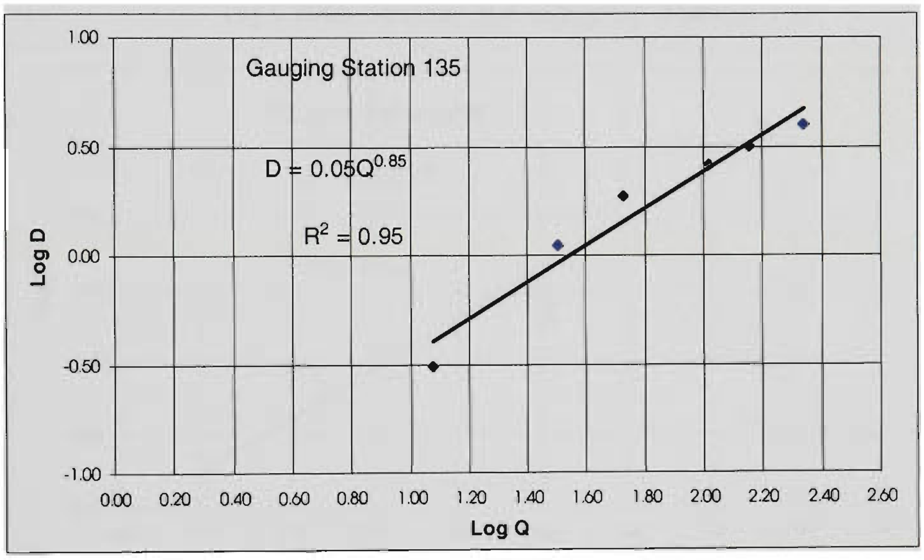


(c) V-Q Relationship

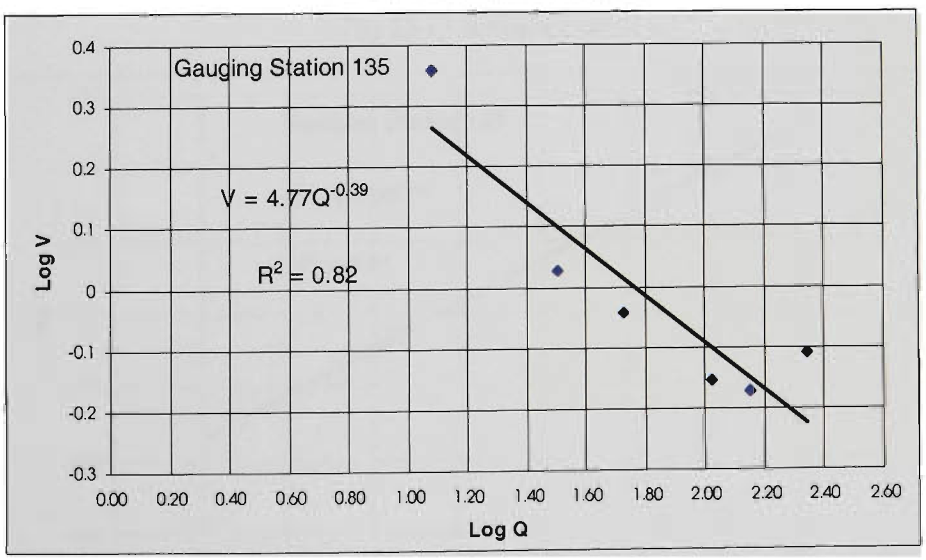
Figure A1-6 Cross-Section for Gauging Station 142
and Power Function for Reach 20



(a) Cross-Section for Gauging Station 135

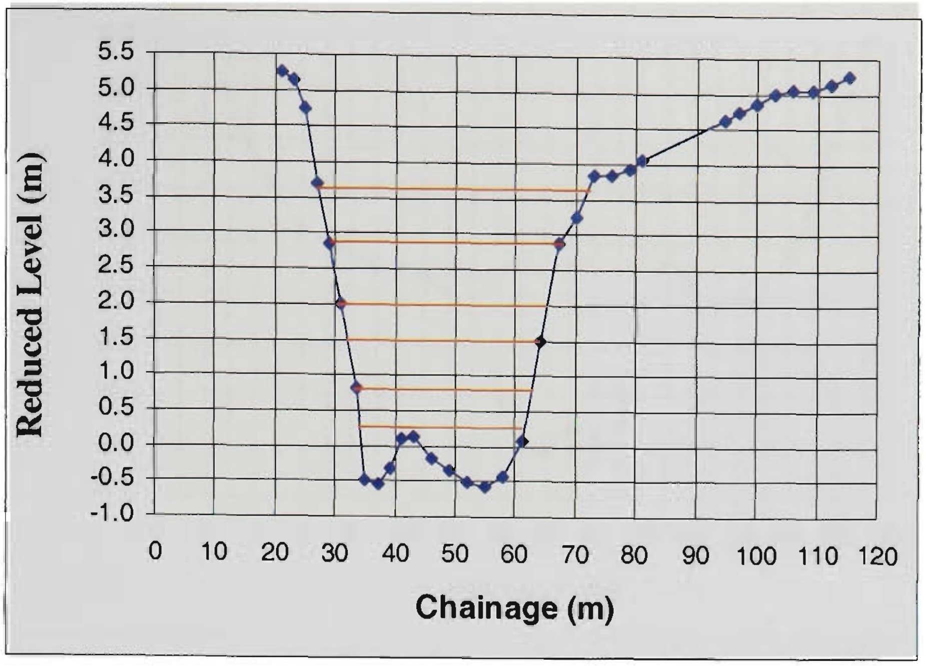


(b) D-Q Relationship

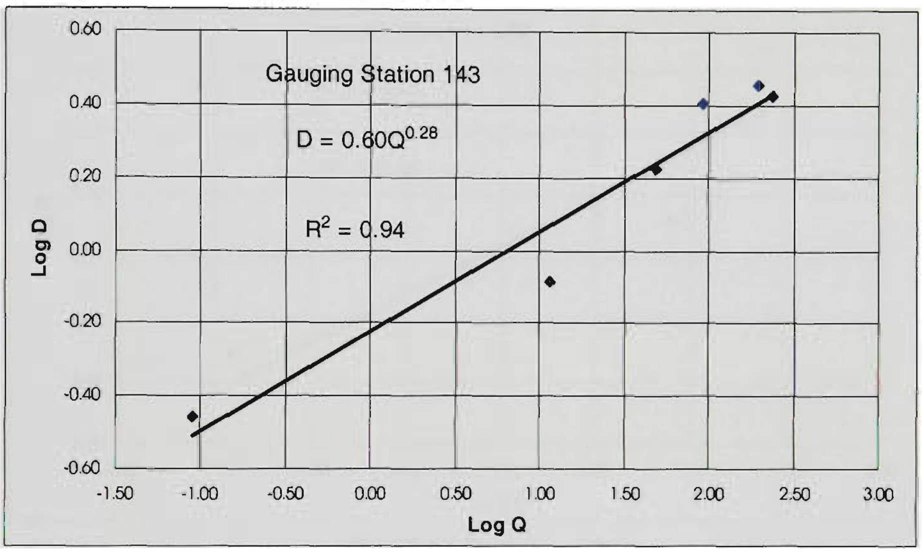


(c) V-Q Relationship

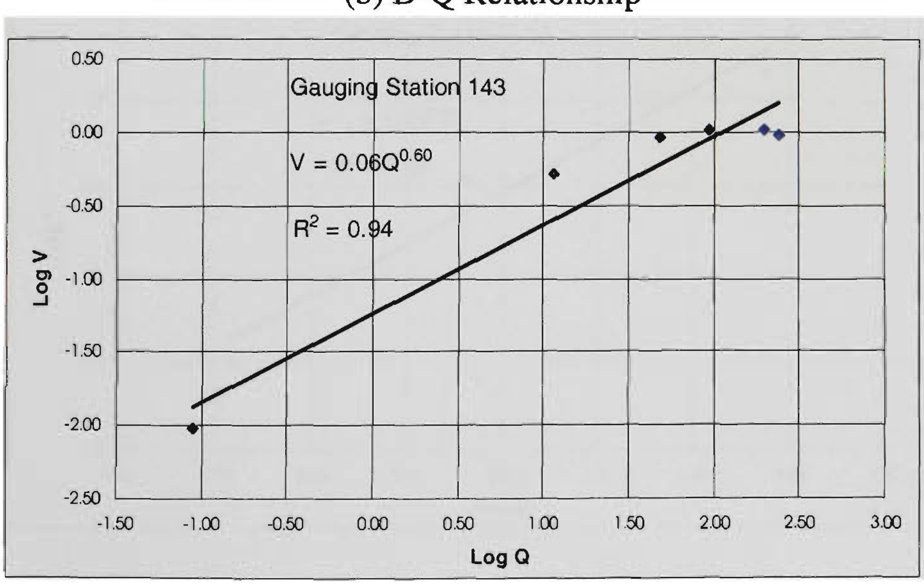
Figure A1-7 Cross-Section for Gauging Station 135
and Power Function for Reach 24



(a) Cross-Section for Gauging Station 143

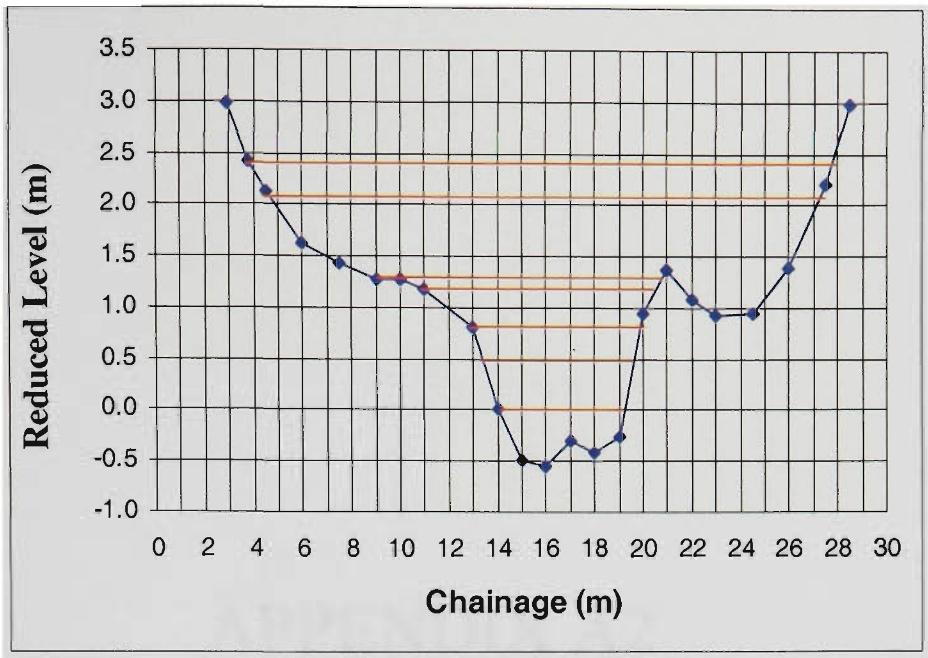


(b) D-Q Relationship

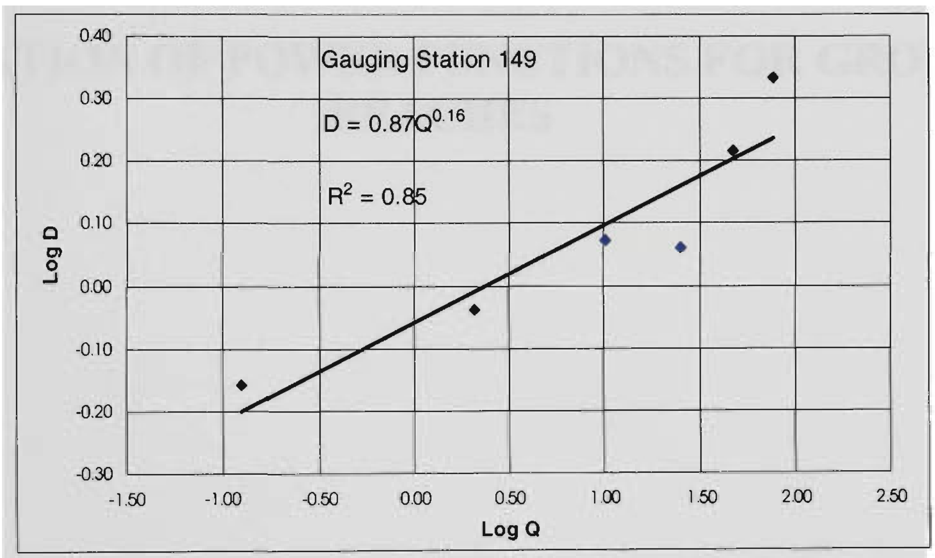


(c) V-Q Relationship

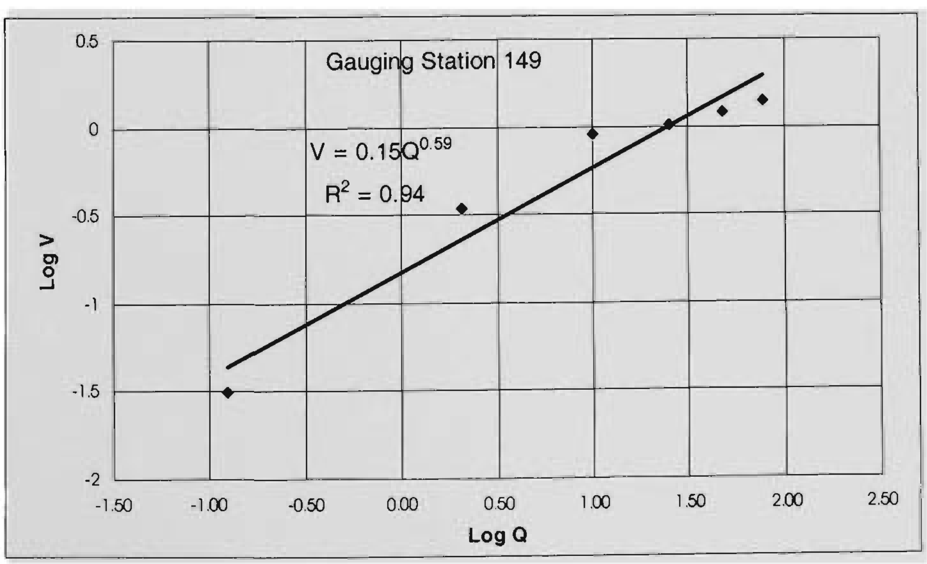
Figure A1-8 Cross-Section for Gauging Station 143
and Power Function for Reach 25



(a) Cross-Section for Gauging Station 149



(b) D-Q Relationship

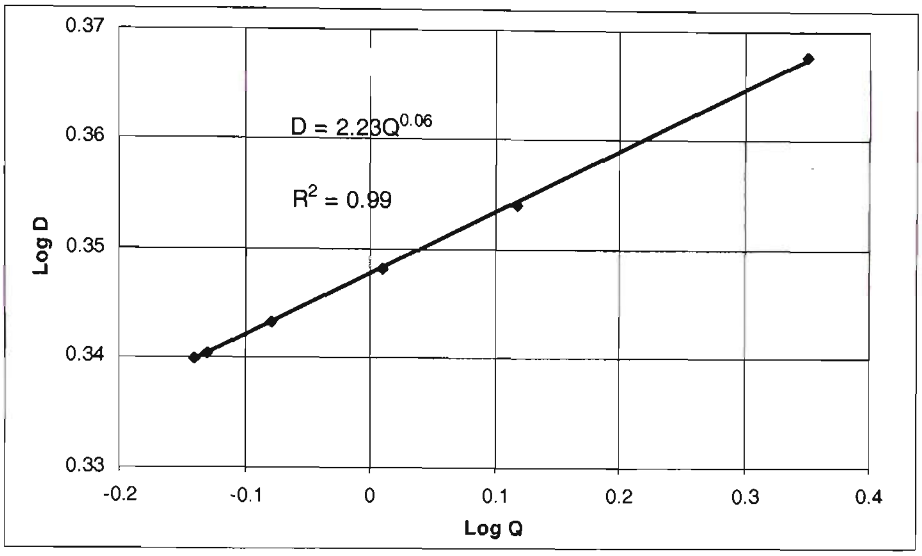


(c) V-Q Relationship

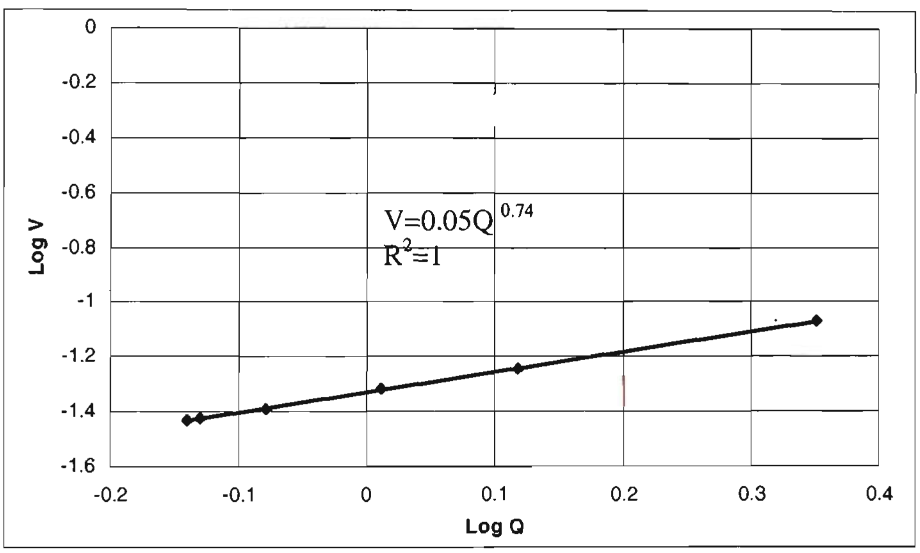
Figure A1-9 Cross-Section for Gauging Station 149 and Power Function for Reaches 21-23, 26-28

APPENDIX A2

DERVIATION OF POWER FUNCTIONS FOR GROUP (b) REACHES

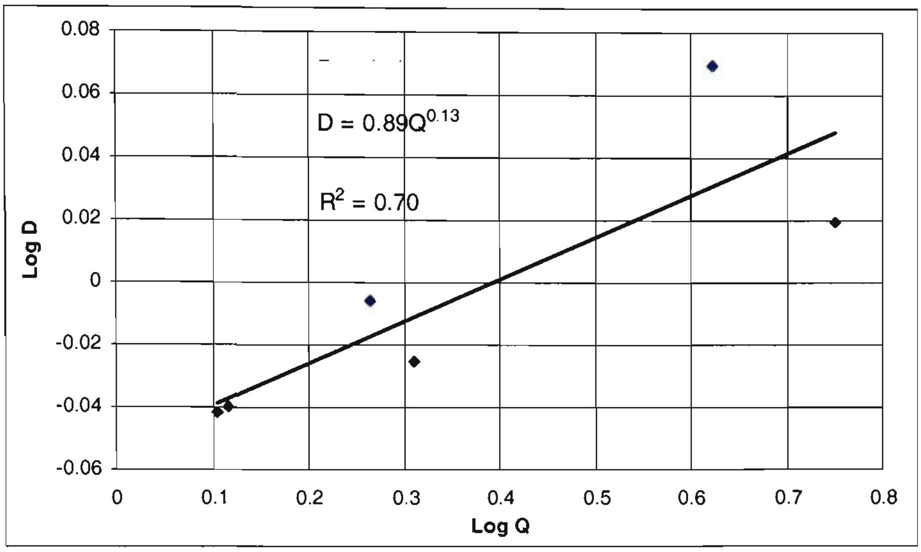


(a) D-Q Relationship

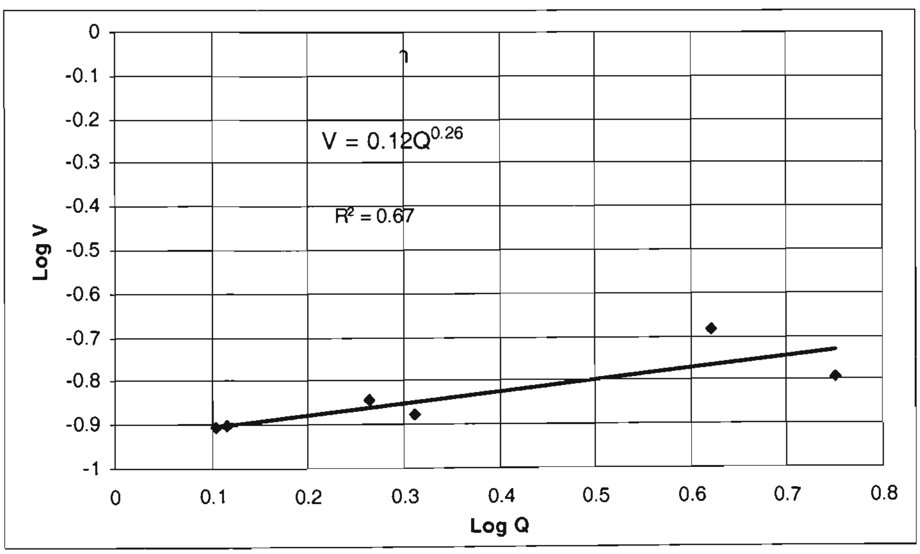


(b) V-Q Relationship

Figure A2-1 Power Functions for Reach 2

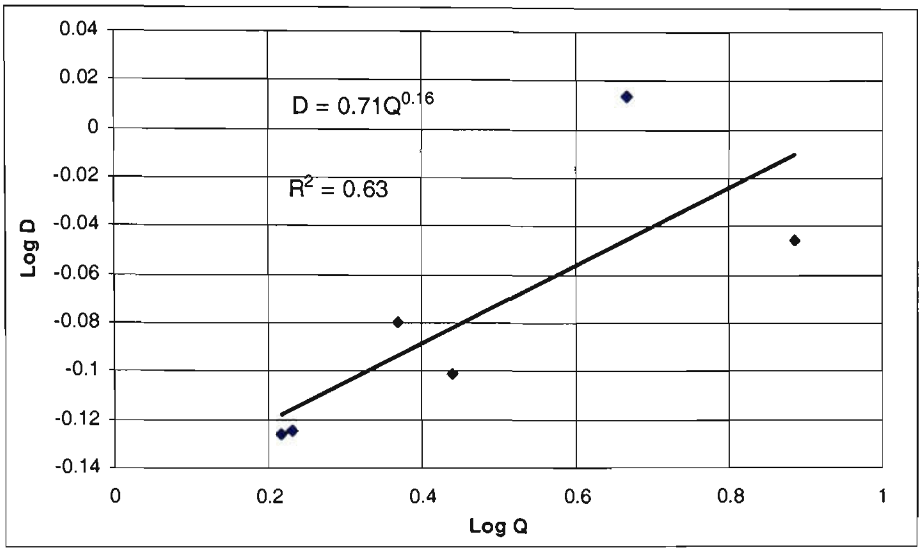


(a) D-Q Relationship

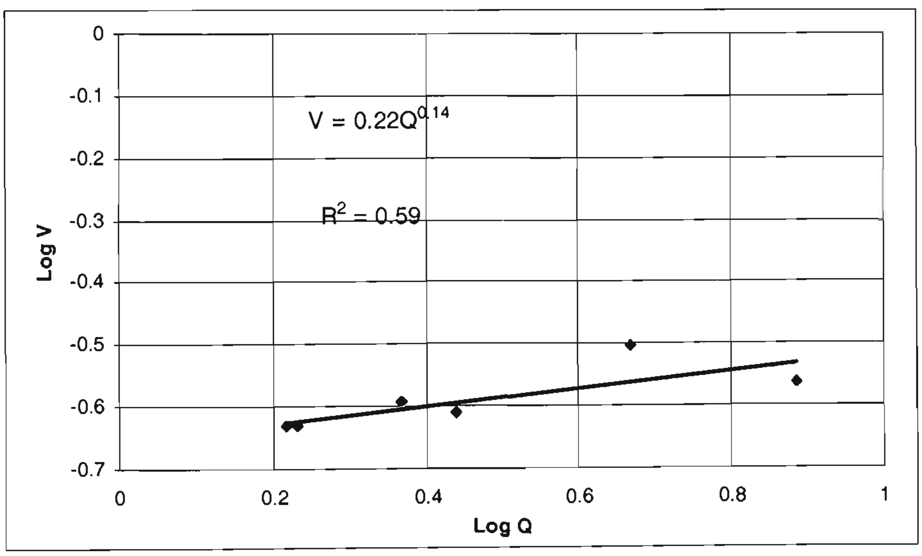


(b) V-Q Relationship

Figure A2-2 Power Functions for Reach 4

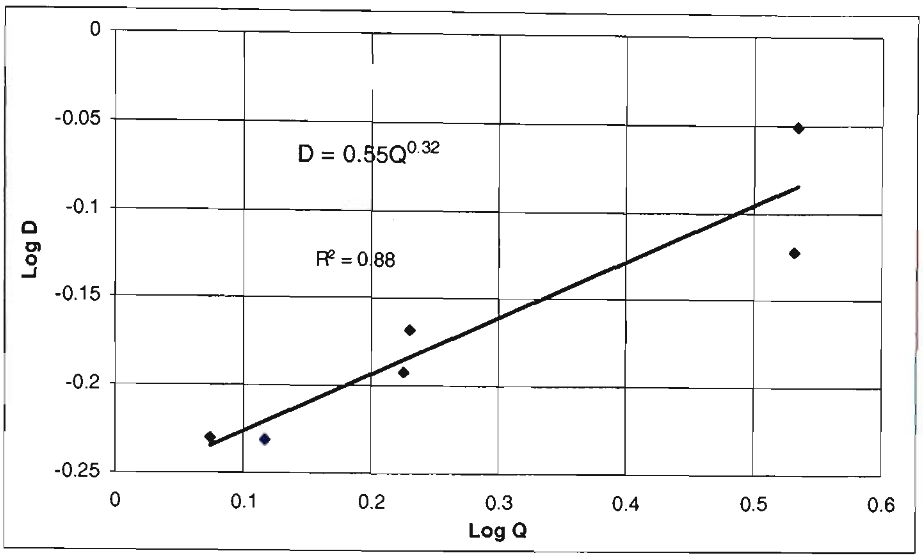


(a) D-Q Relationship

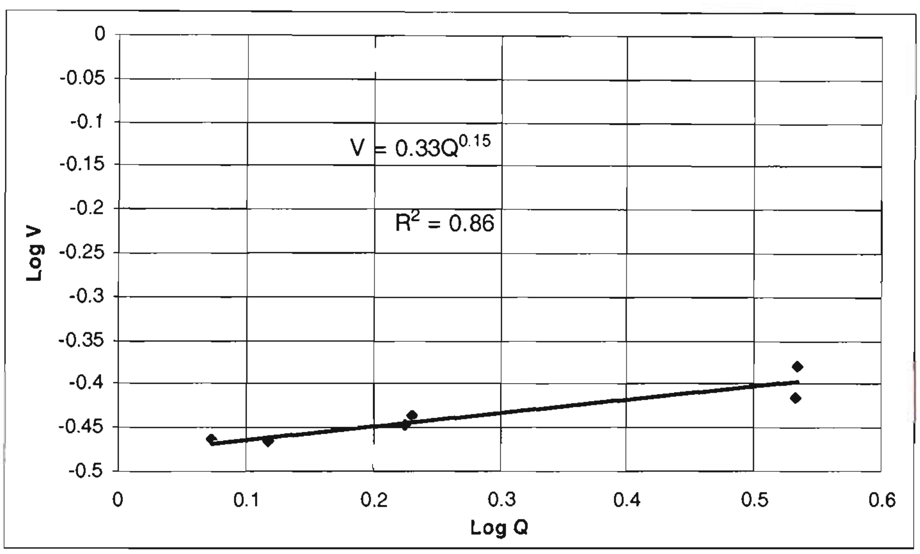


(b) V-Q Relationship

Figure A2-3 Power Functions for Reach 5

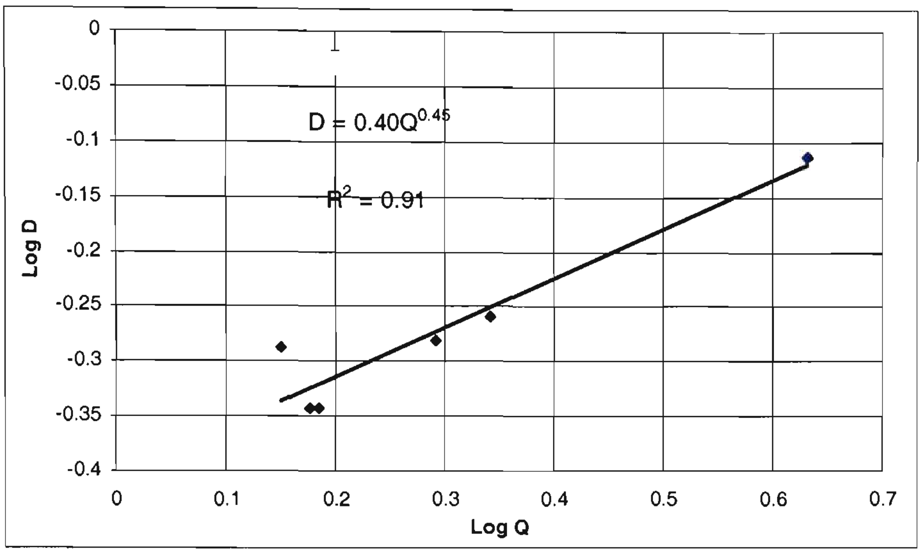


(a) D-Q Relationship

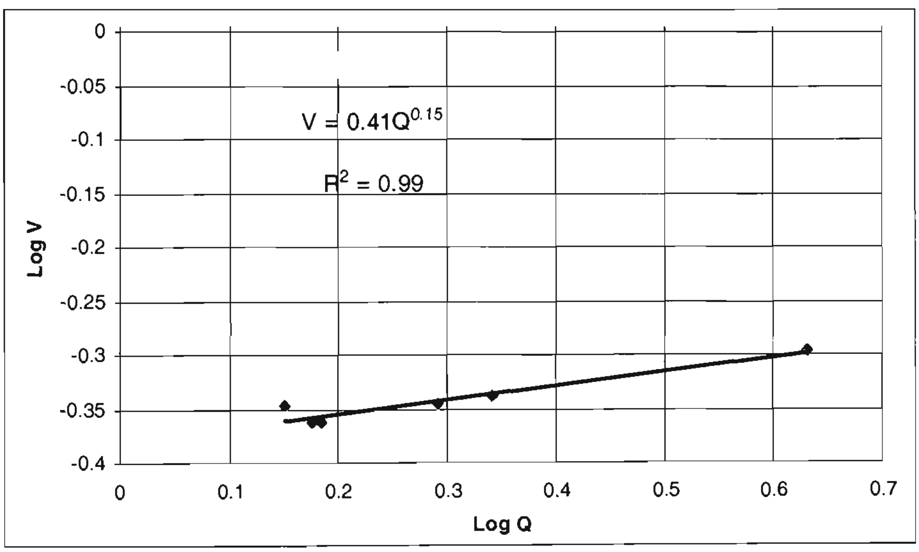


(b) V-Q Relationship

Figure A2-4 Power Functions for Reach 6

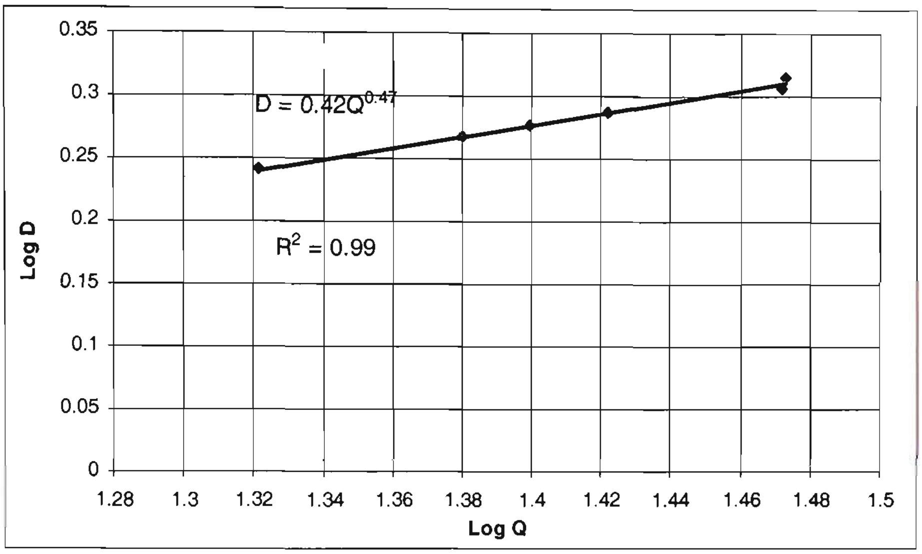


(a) D-Q Relationship

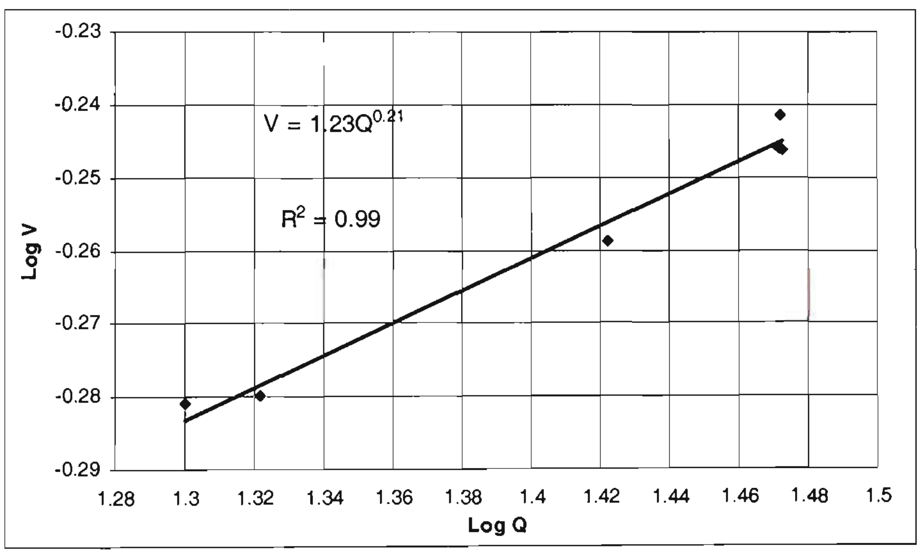


(b) V-Q Relationship

Figure A2-5 Power Functions for Reach 10

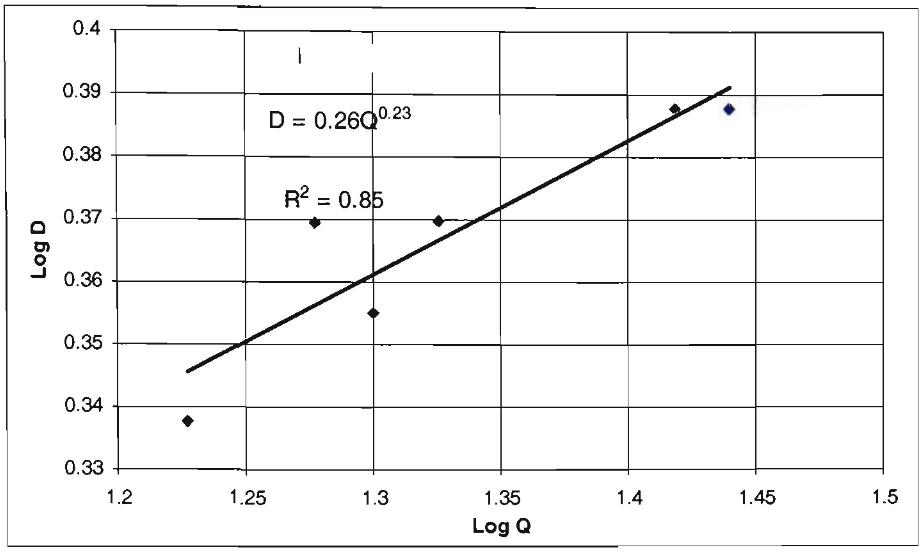


(a) D-Q Relationship

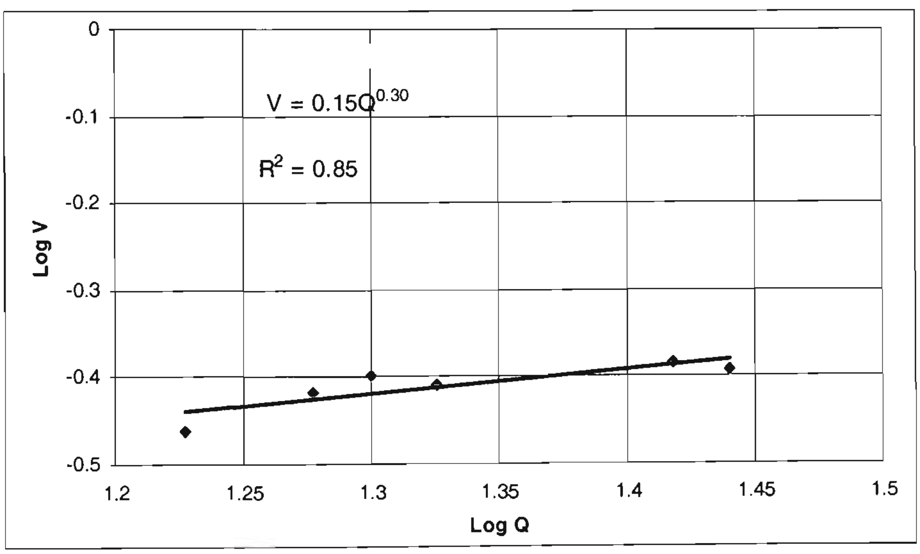


(b) V-Q Relationship

Figure A2-6 Power Functions for Reach 12

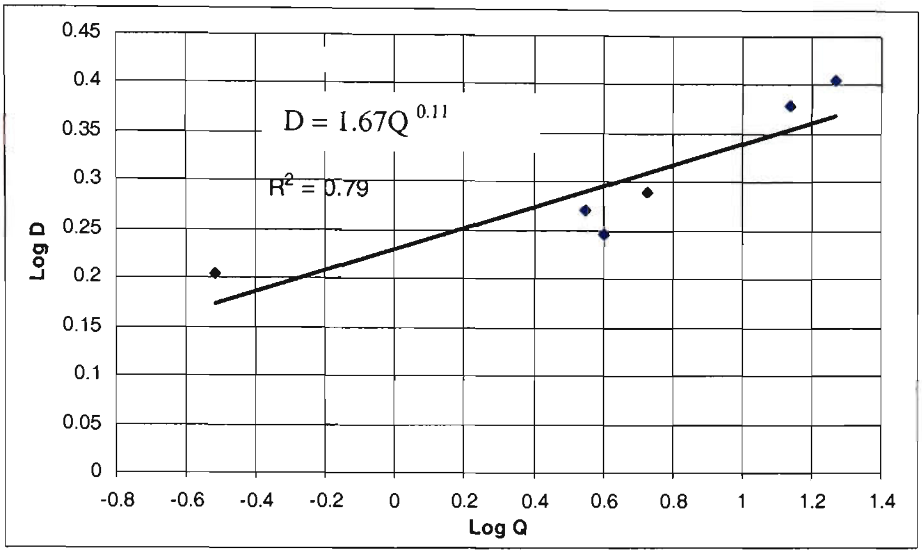


(a) D-Q Relationship

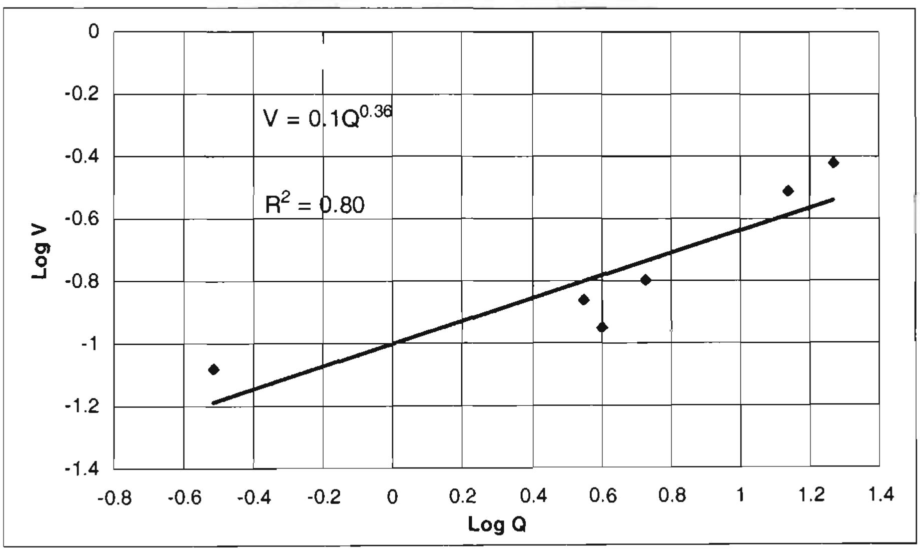


(b) V-Q Relationship

Figure A2-7 Power Functions for Reach 13

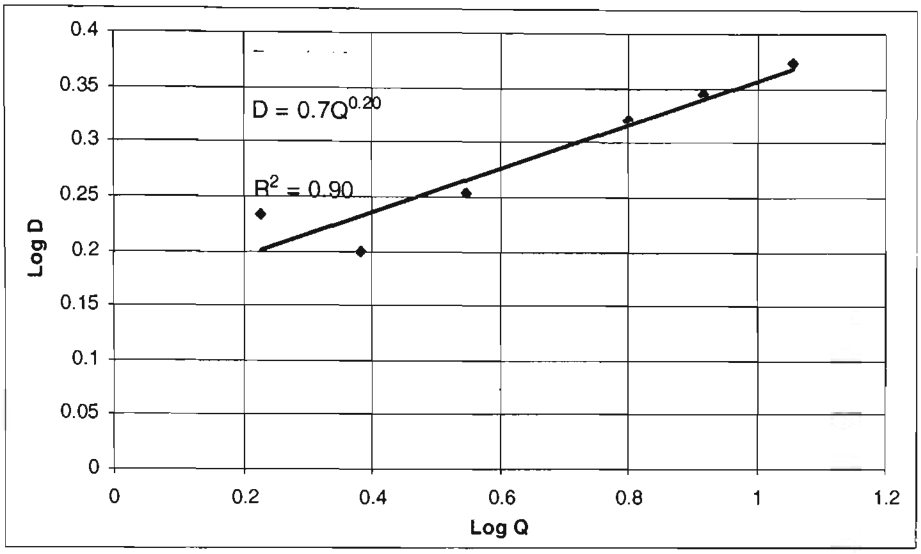


(a) D-Q Relationship

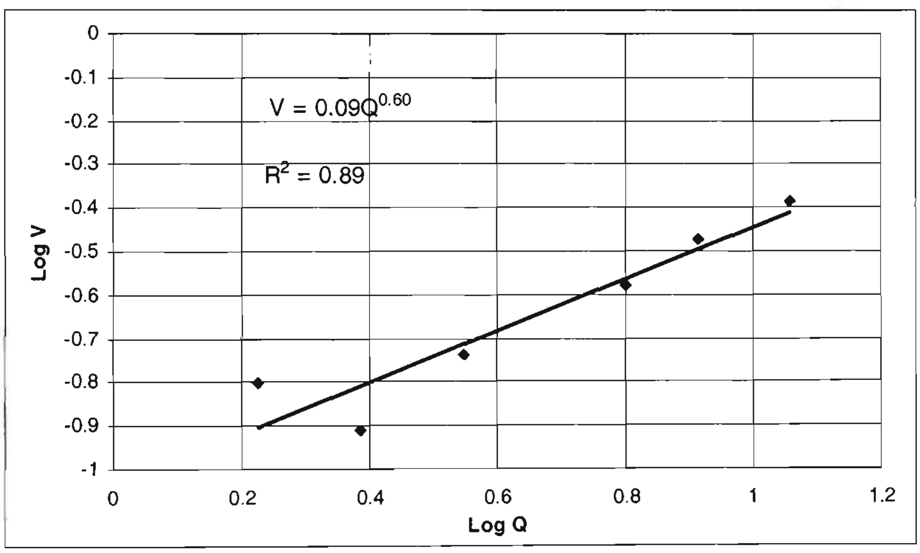


(b) V-Q Relationship

Figure A2-8 Power Functions for Reach 17



(a) D-Q Relationship



(b) V-Q Relationship

Figure A2-9 Power Functions for Reach 18

APPENDIX B

EFFECT OF INPUT PARAMETER COEFFICIENT OF VARIATION ON SENSITIVITY OF PARAMETER GROUPS

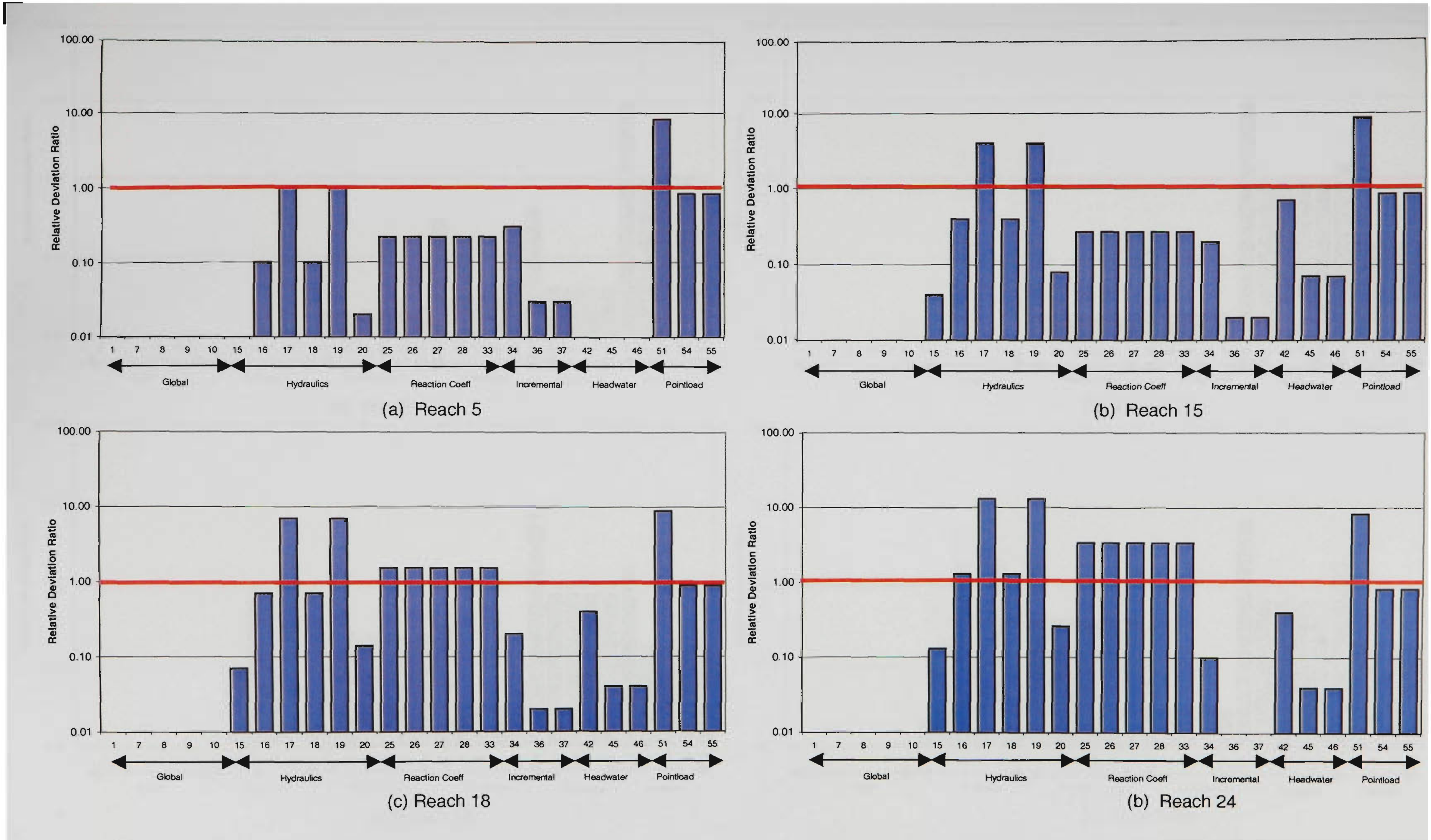


Figure B1 Relative Deviation Ratio of TKN for 'Lower Range' Flow Event (Minimum CV)

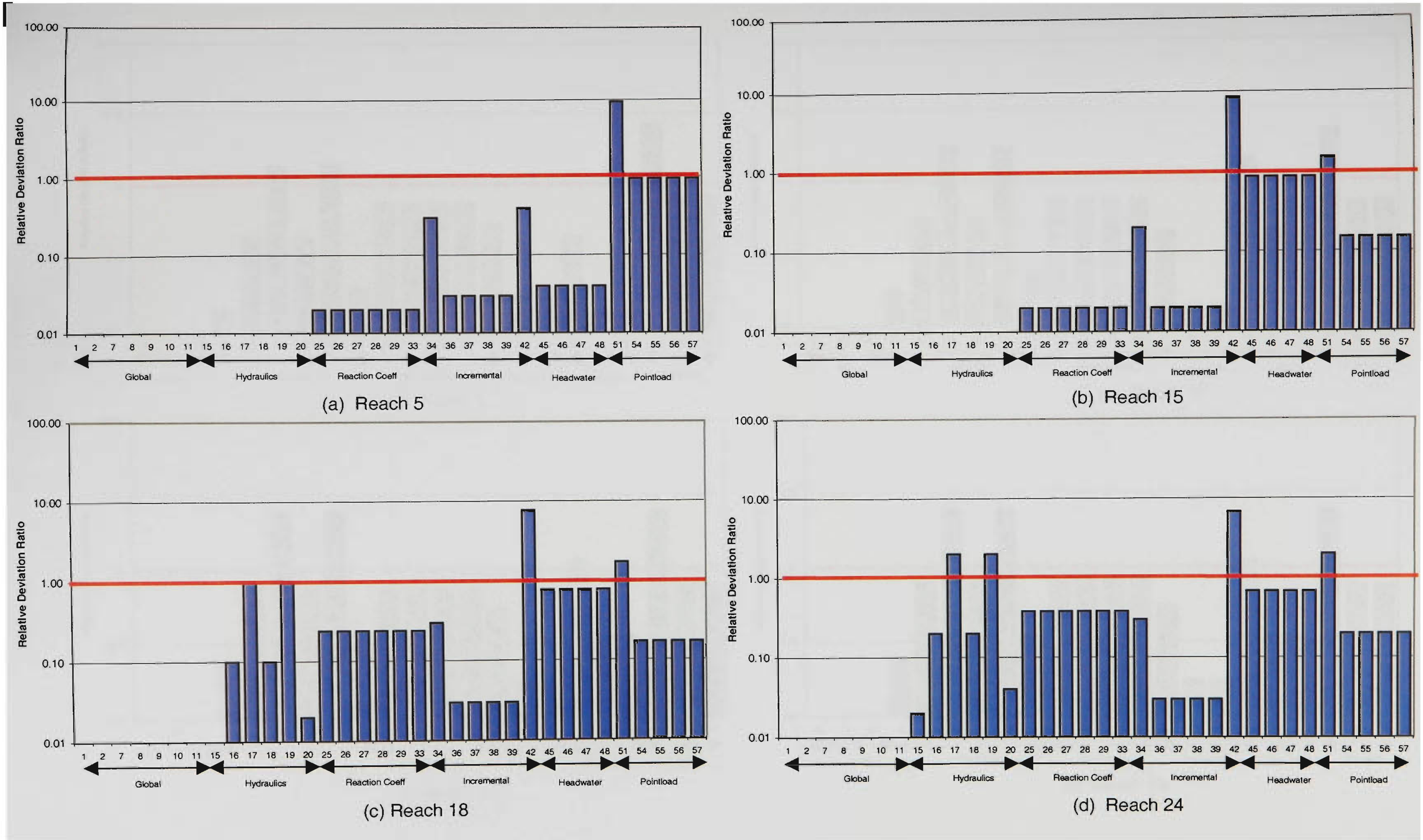
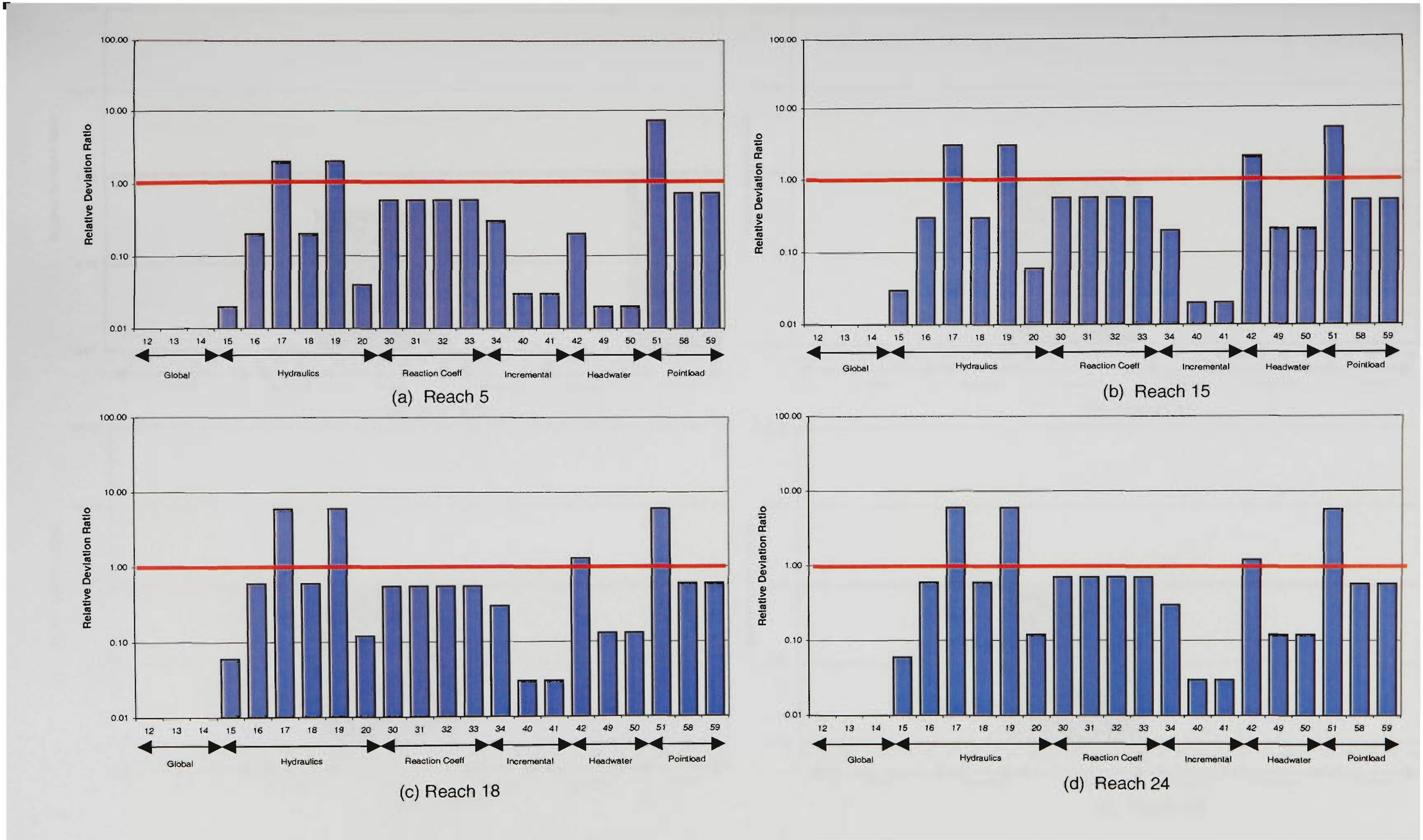


Figure B2 Relative Deviation Ratio of TN for 'Lower Range' Flow Event (Minimum CV)



(a) Reach 5

(b) Reach 15

(c) Reach 18

(d) Reach 24

Figure B3 Relative Deviation Ratio of TP for 'Lower Range' Flow Event (Minimum CV)

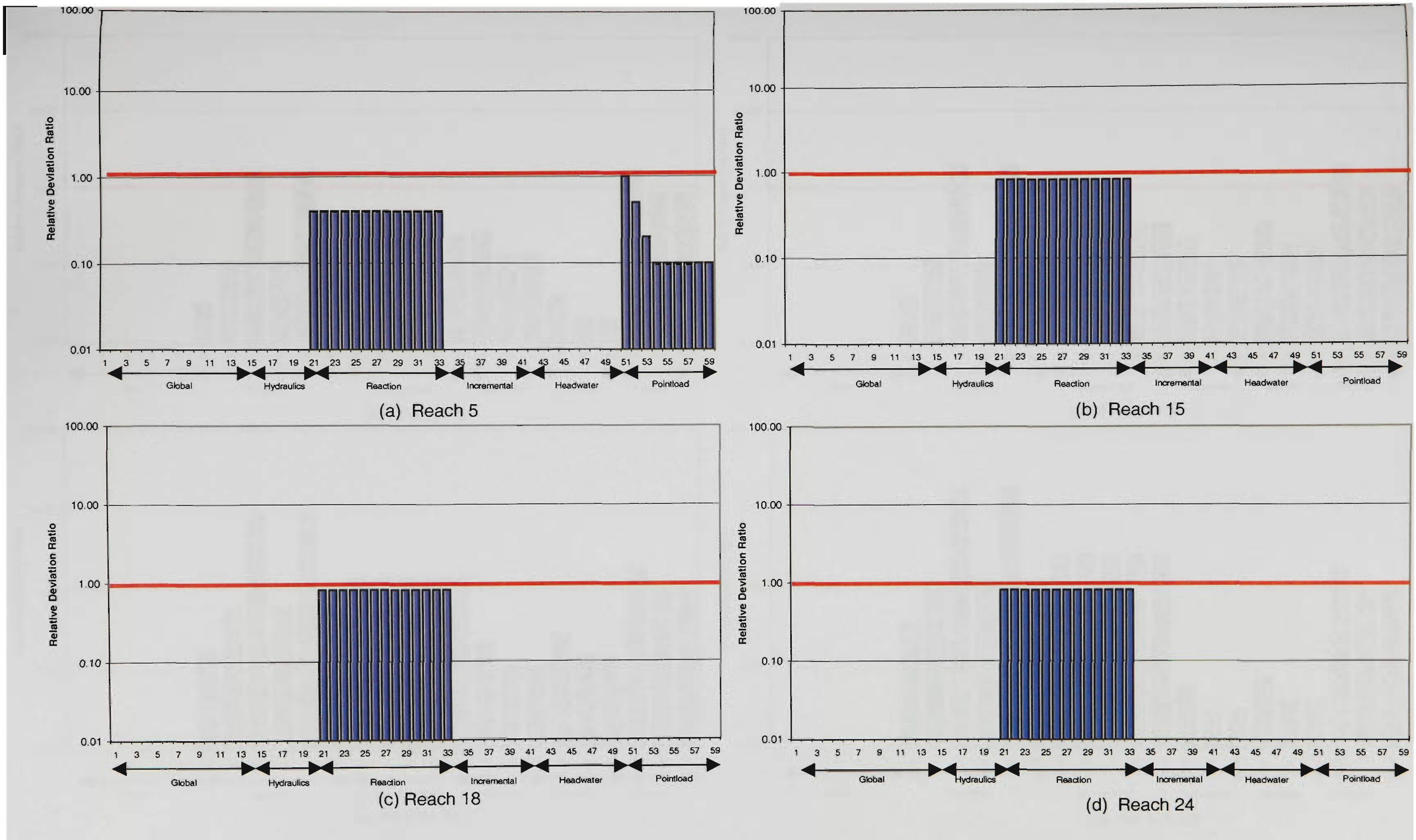


Figure B4 Relative Deviation Ratio of DO for 'Lower Range' Flow Event (Minimum CV)

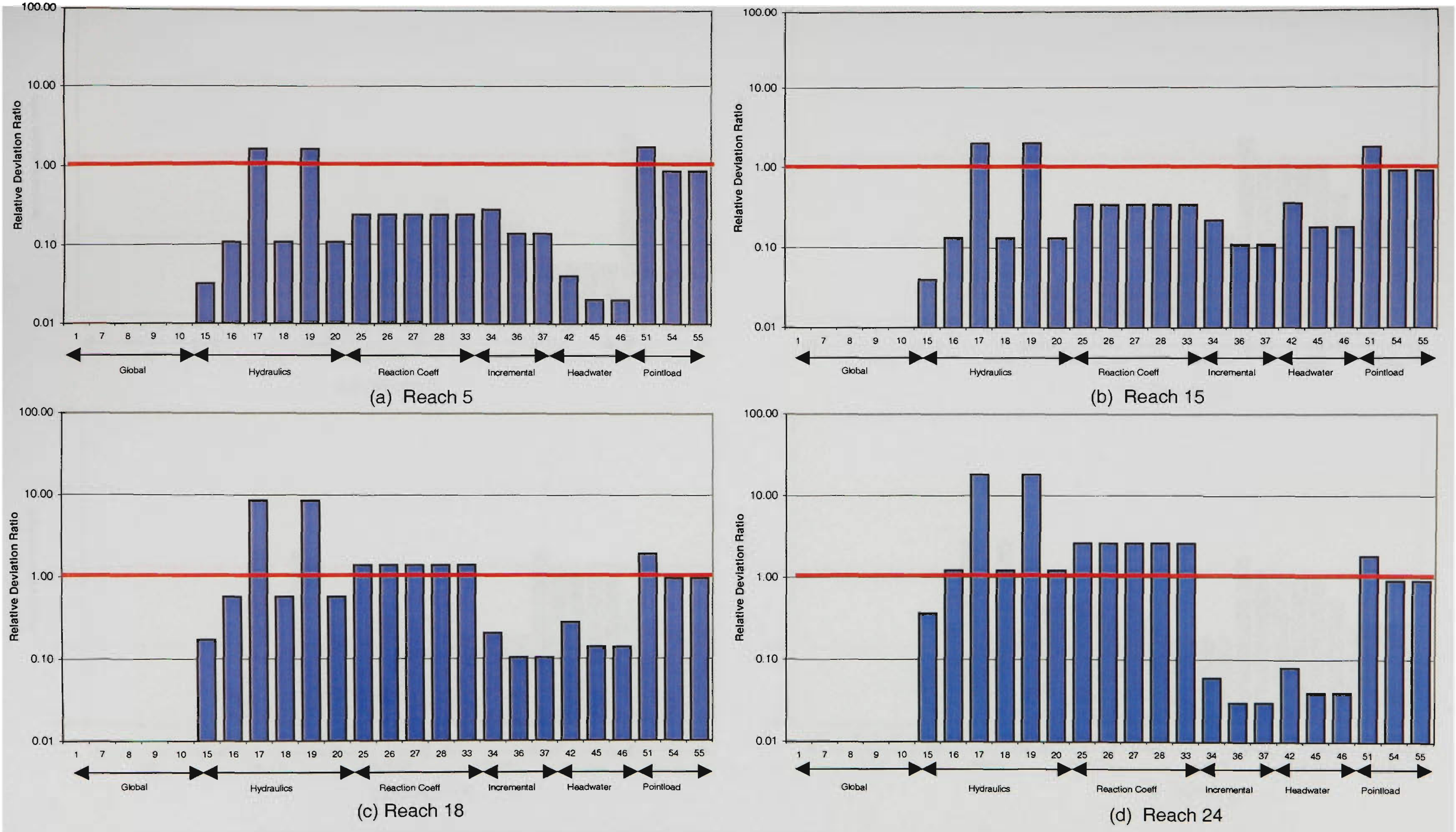


Figure B5 Relative Deviation Ratio of TKN for 'Lower Range' Flow Event (Maximum CV)

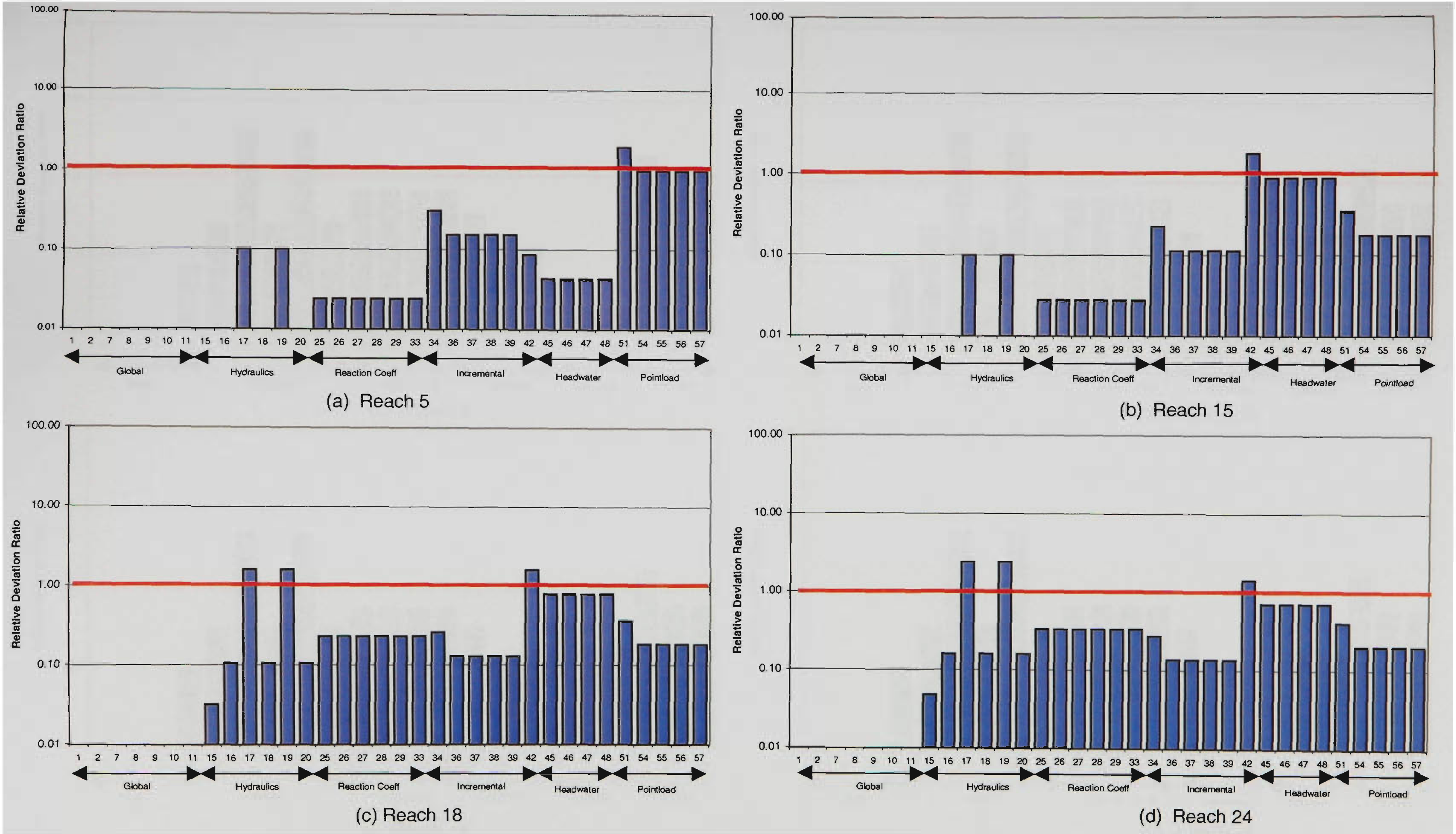


Figure B6 Relative Deviation Ratio of TN for 'Lower Range' Flow Event (Maximum CV)

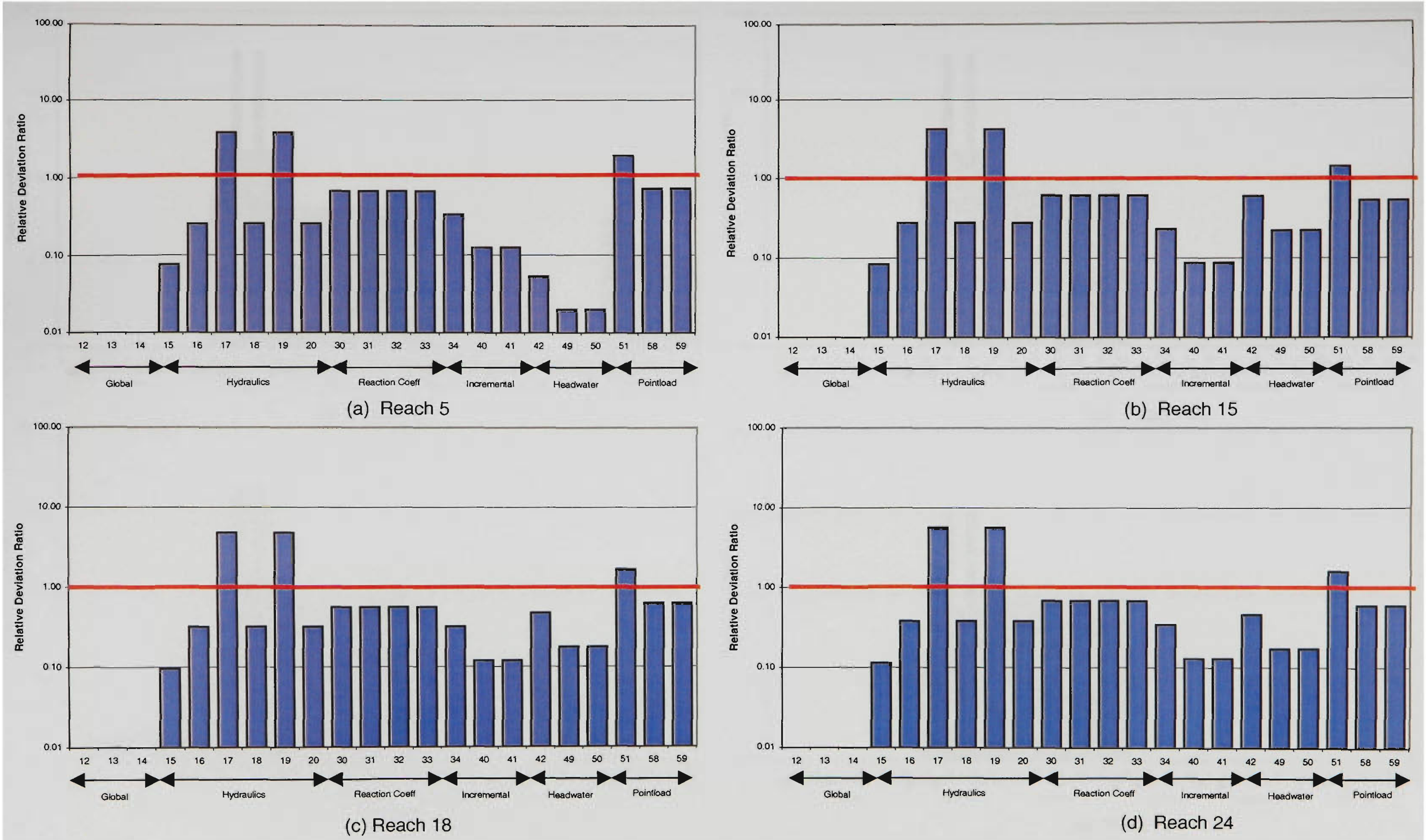


Figure B7 Relative Deviation Ratio of TP for 'Lower Range' Flow Event (Maximum CV)

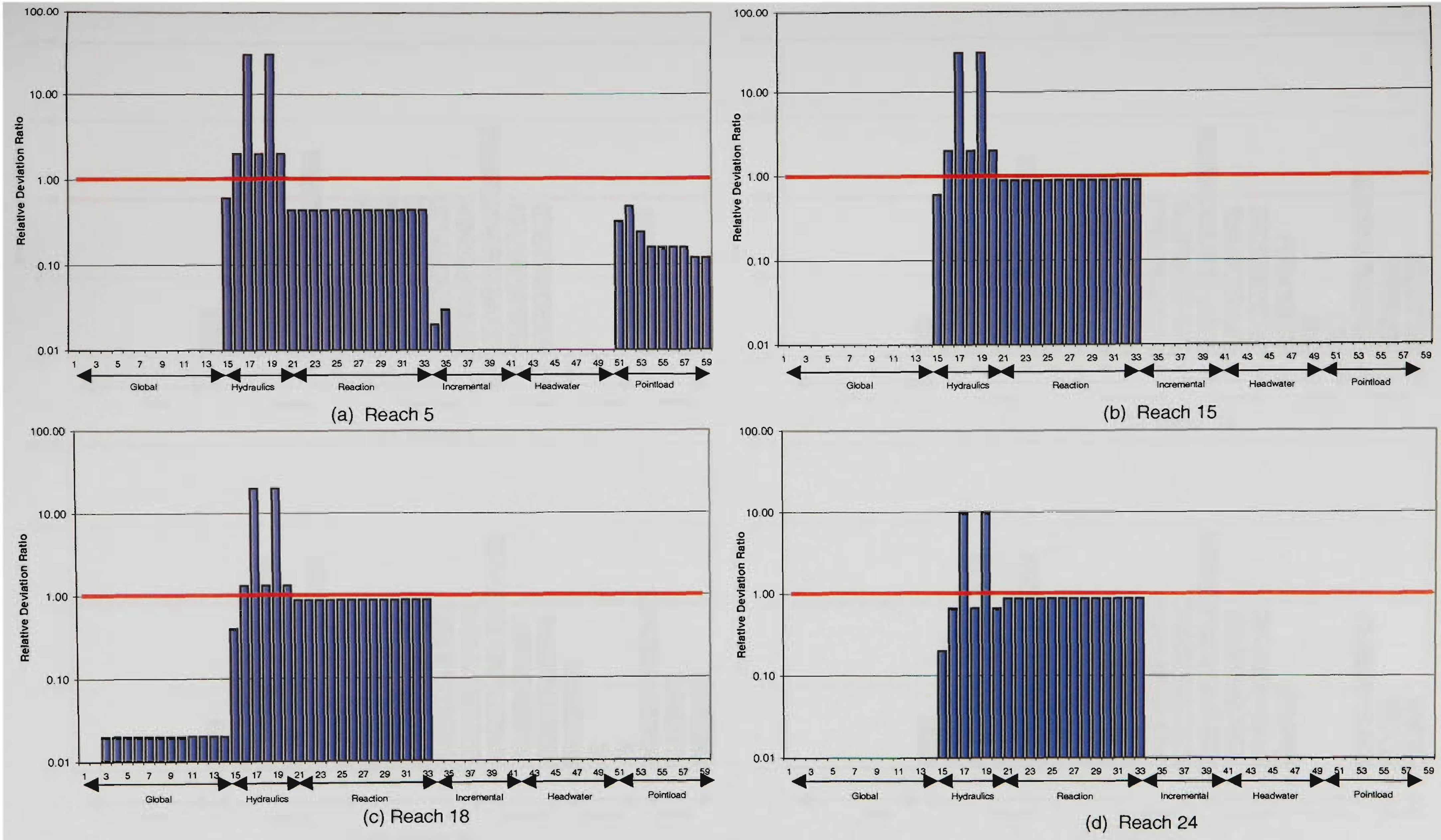
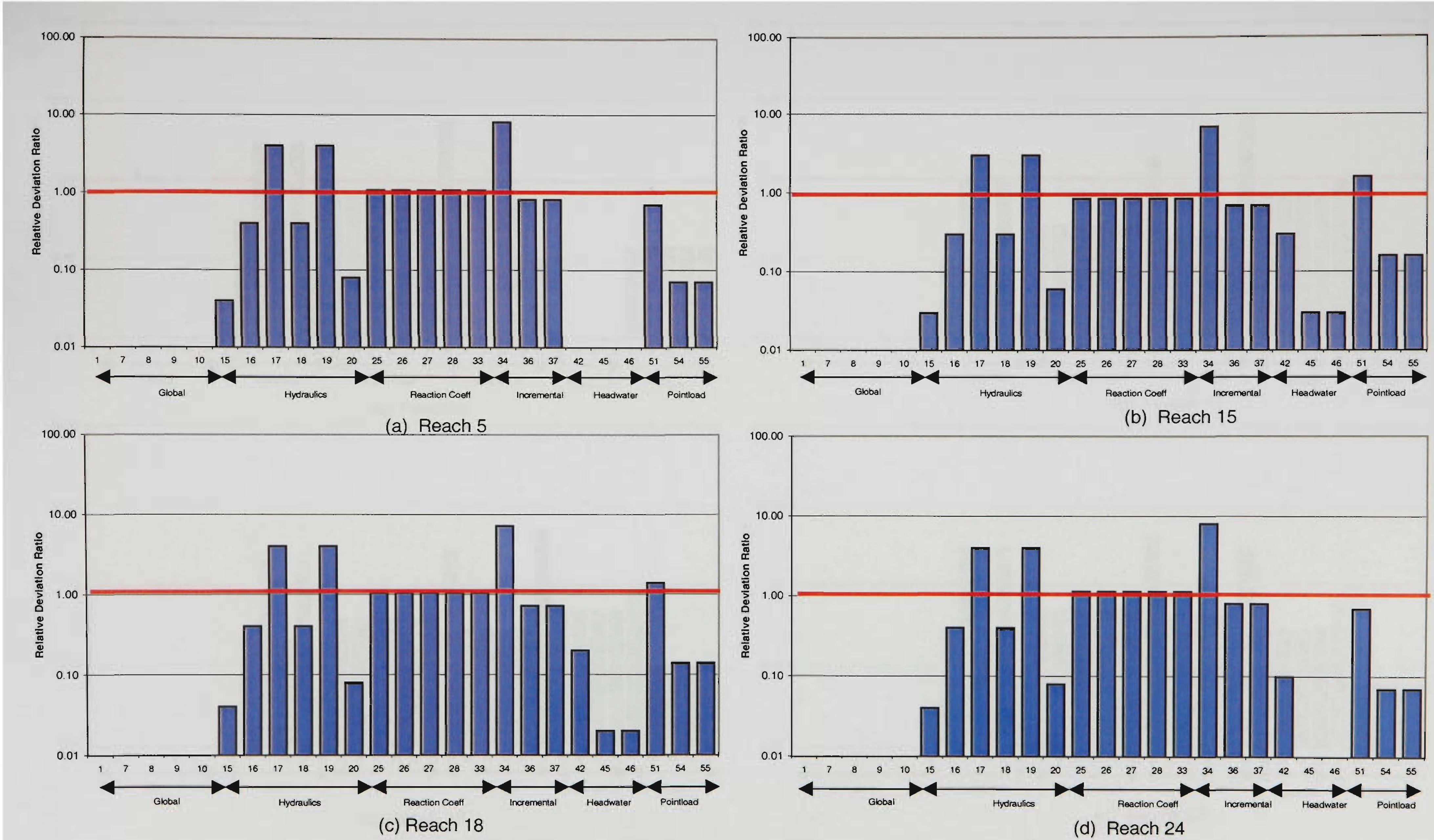


Figure B8 Relative Deviation Ratio of DO for 'Lower Range' Flow Event (Maximum CV)



(a) Reach 5

(b) Reach 15

(c) Reach 18

(d) Reach 24

Figure B9 Relative Deviation Ratio of TKN for 'Higher Range' Flow Event (Minimum CV)

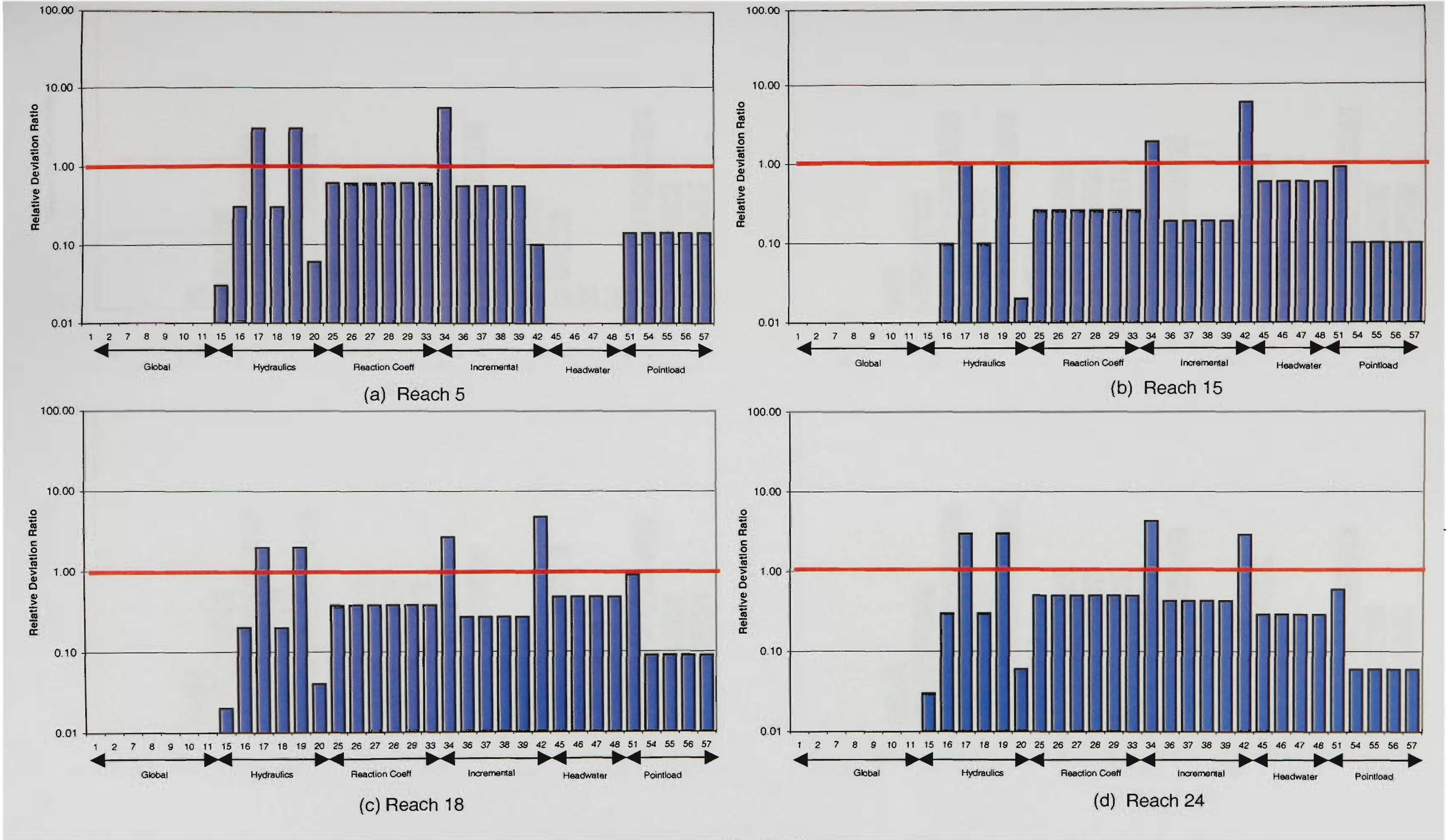


Figure B10 Relative Deviation Ratio of TN for 'Higher Range' Flow Event (Minimum CV)

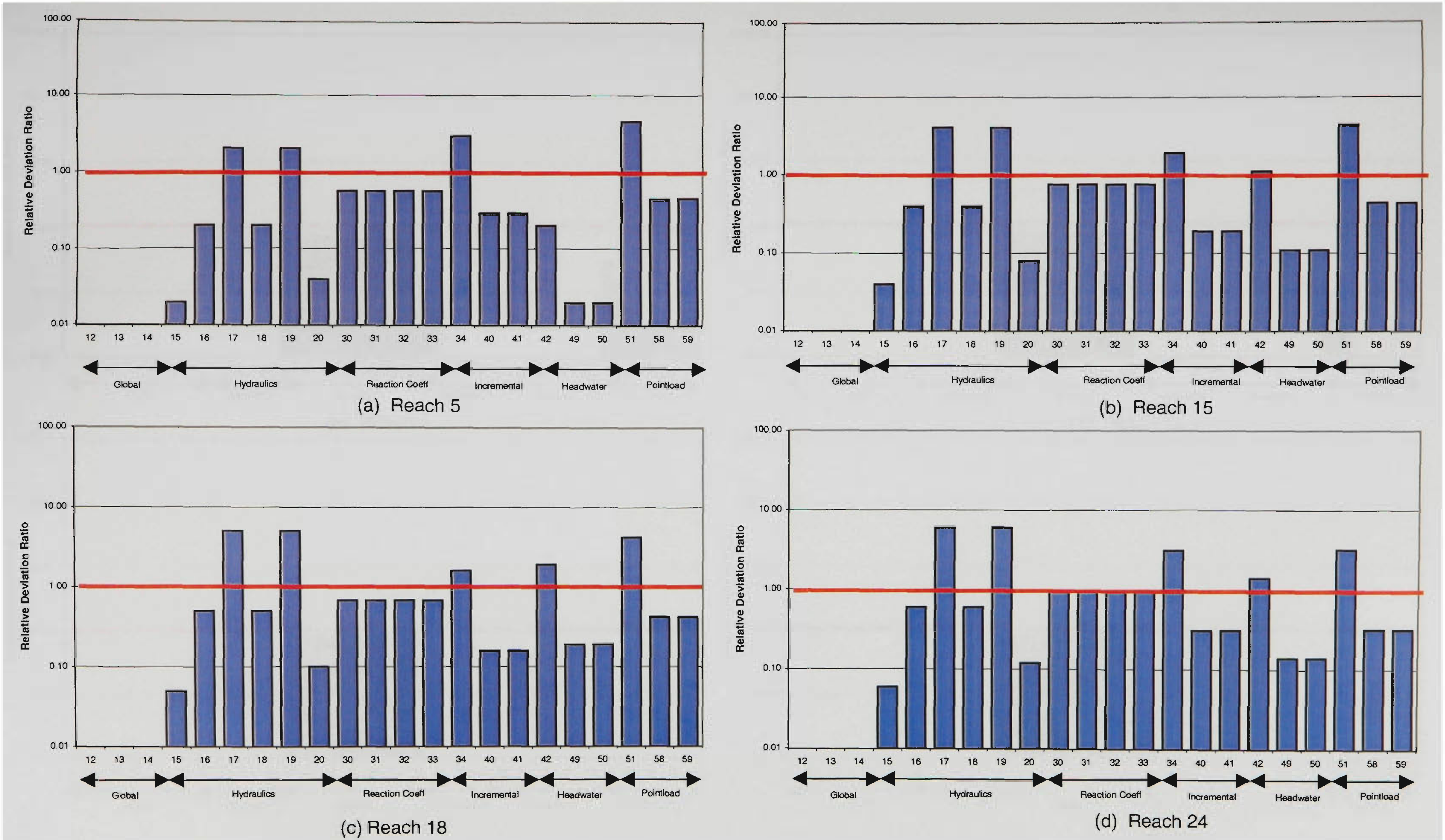


Figure B11 Relative Deviation Ratio of TP for 'Higher Range' Flow Event (Minimum CV)

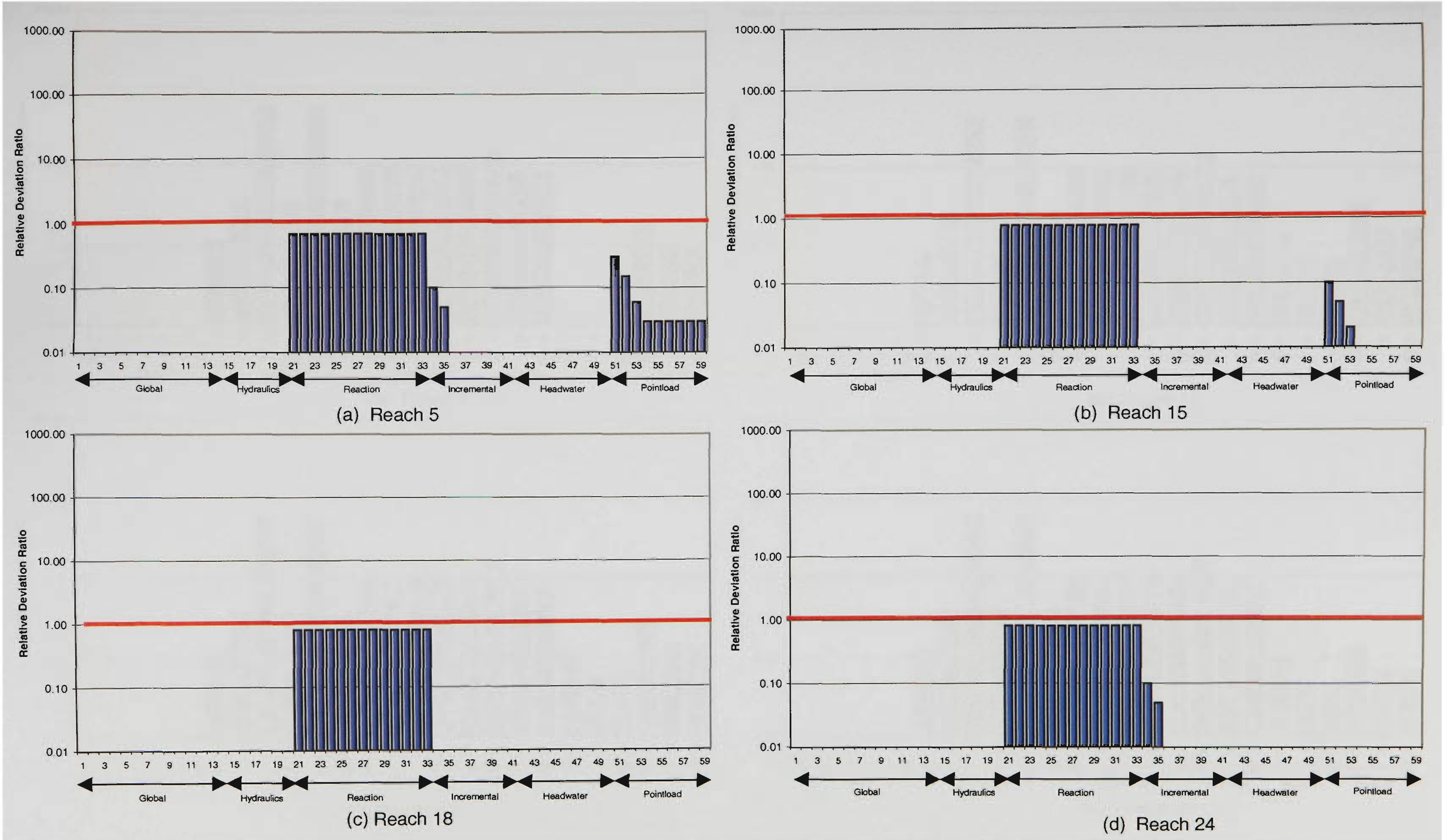


Figure B12 Relative Deviation Ratio of DO for 'Higher Range' Flow Event (Minimum CV)

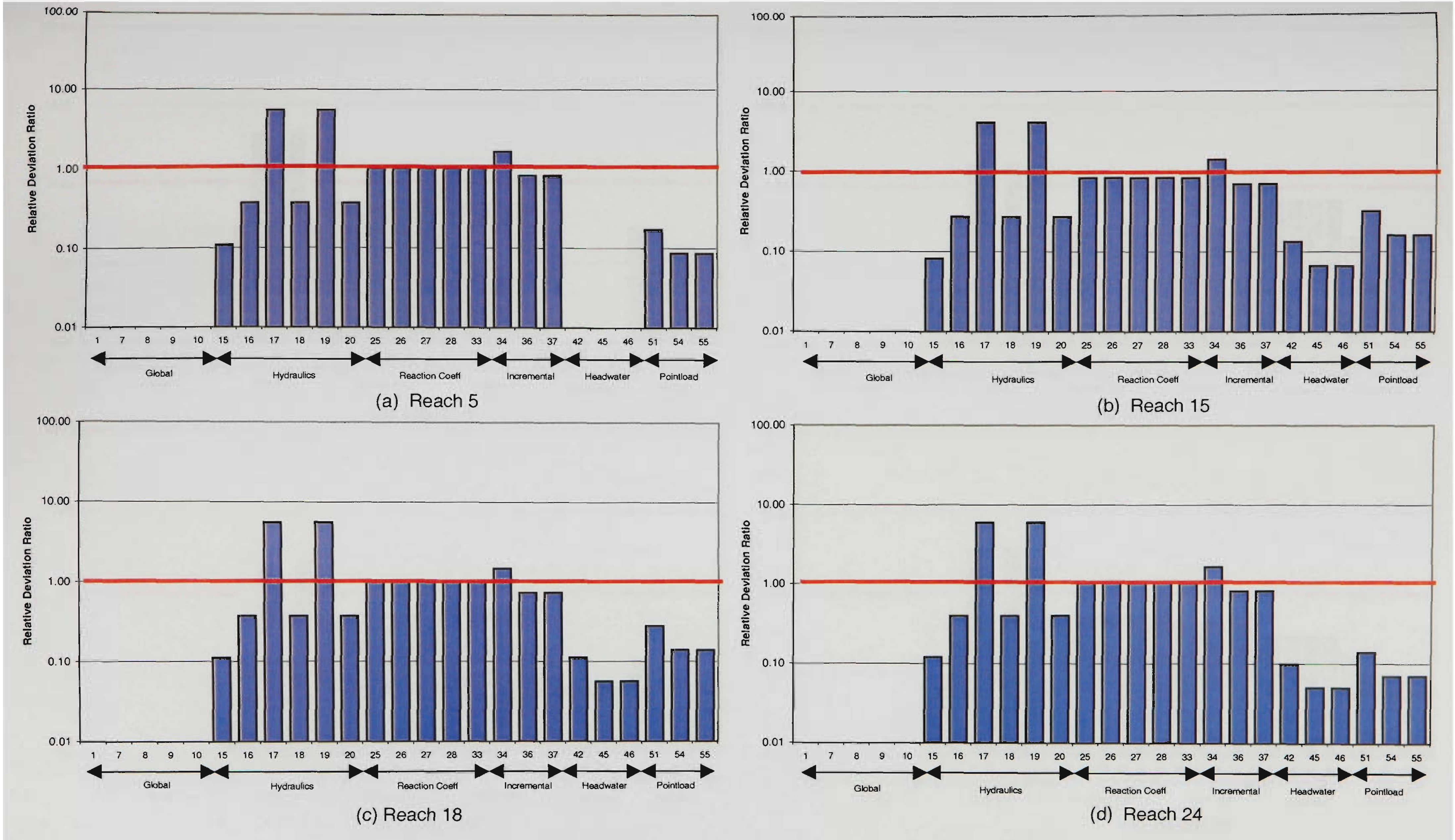


Figure B13 Relative Deviation Ratio of TKN for 'Higher Range' Flow Event (Maximum CV)

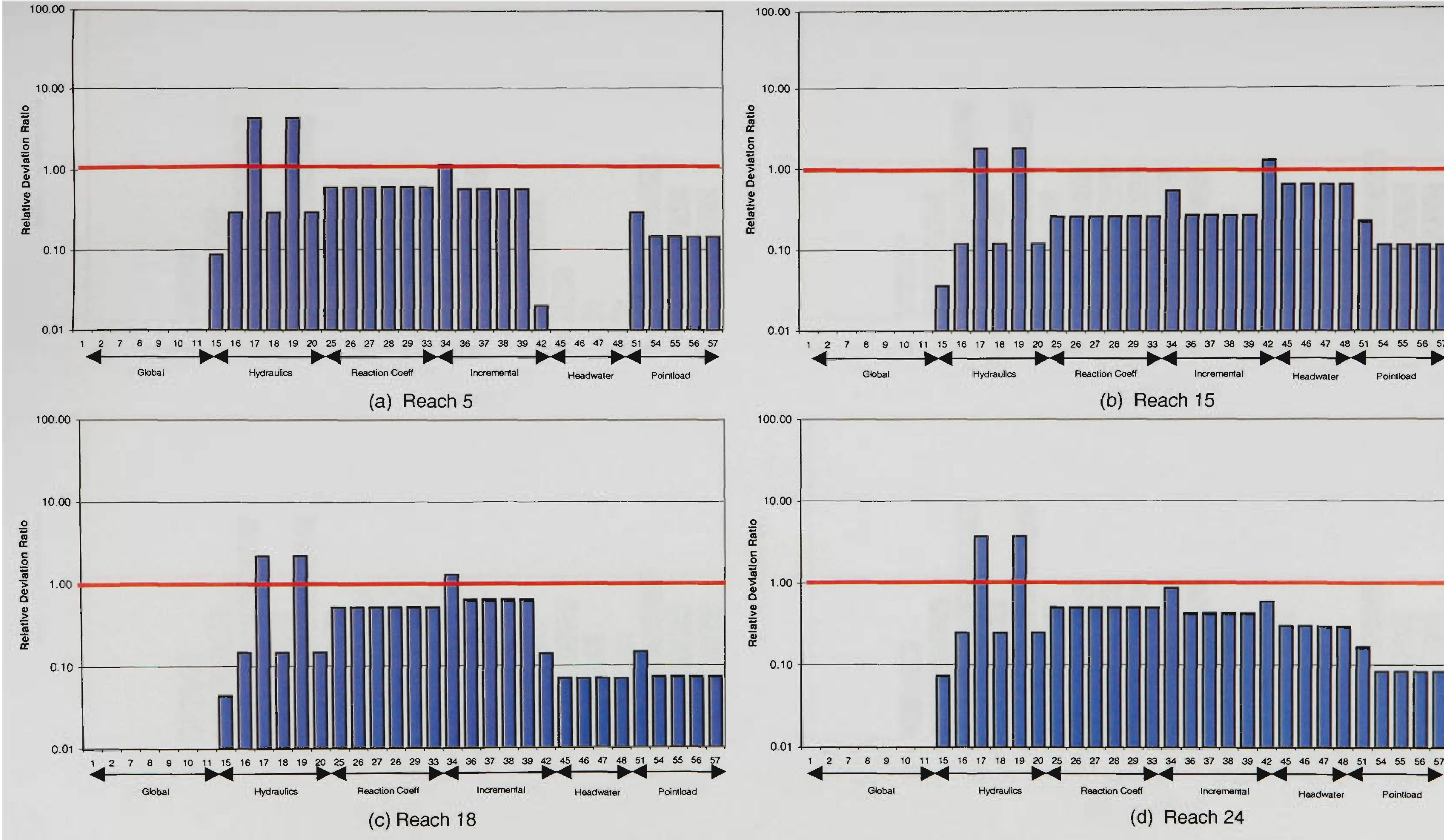
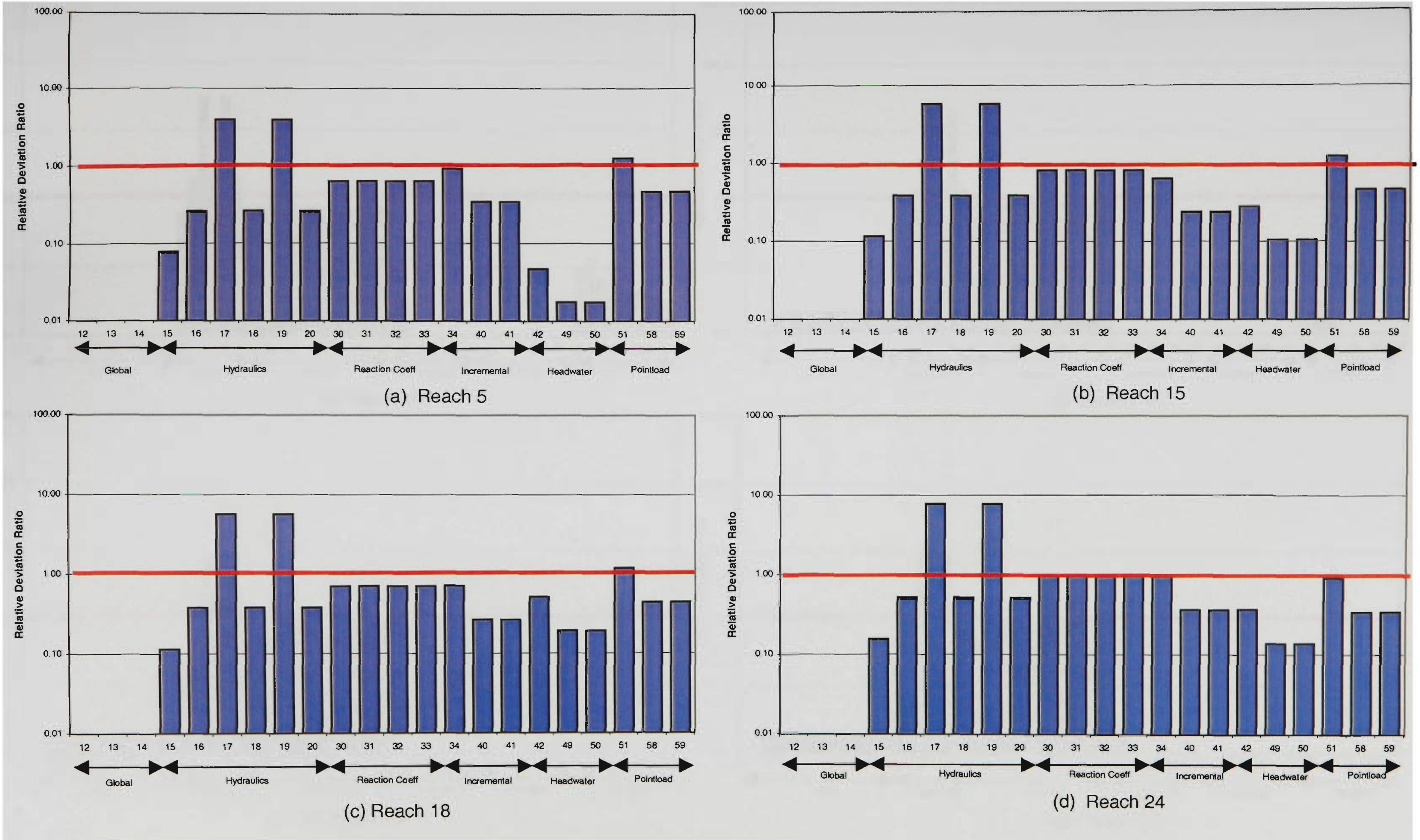


Figure B14 Relative Deviation Ratio of TN for 'Higher Range' Flow Event (Maximum CV)



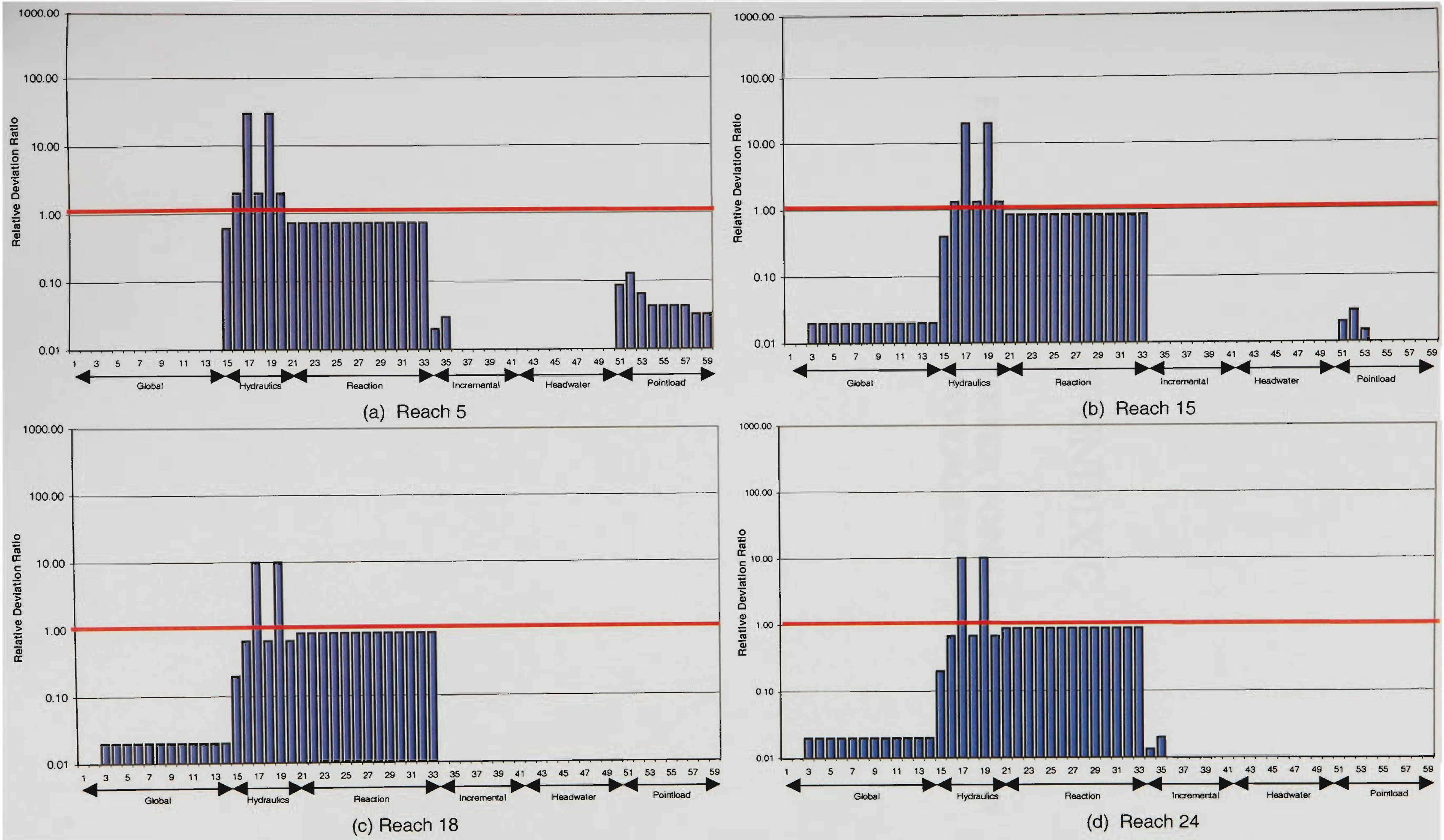
(a) Reach 5

(b) Reach 15

(c) Reach 18

(d) Reach 24

Figure B15 Relative Deviation Ratio of TP for 'Higher Range' Flow Event (Maximum CV)



(a) Reach 5

(b) Reach 15

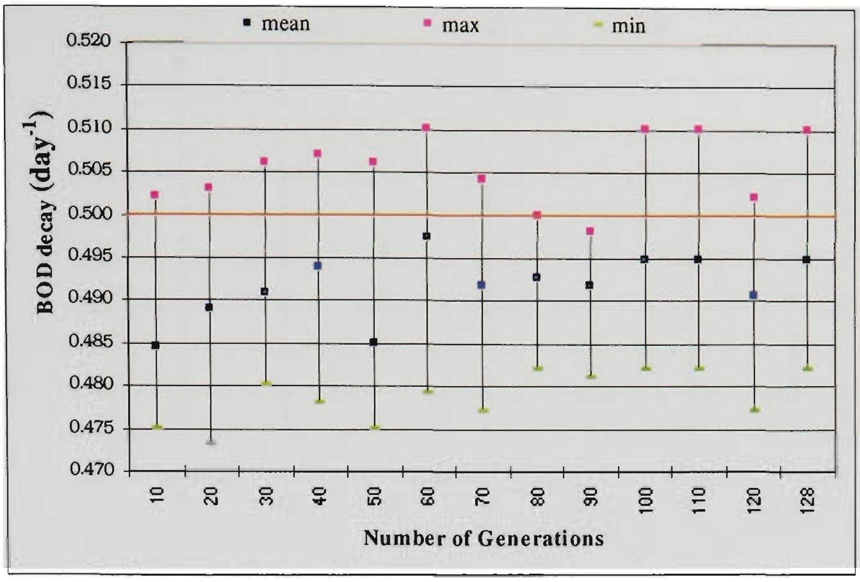
(c) Reach 18

(d) Reach 24

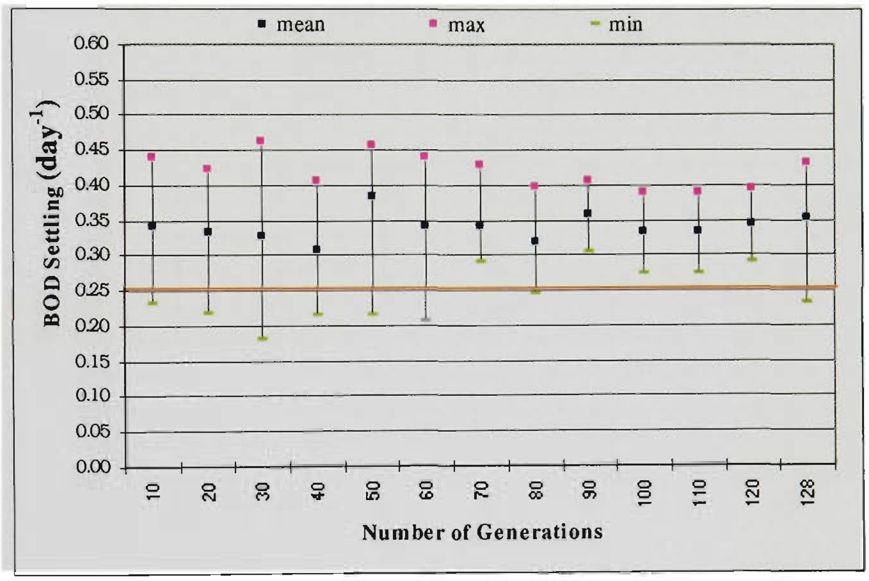
Figure B16 Relative Deviation Ratio of DO for 'Higher Range' Flow Event (Maximum CV)

APPENDIX C

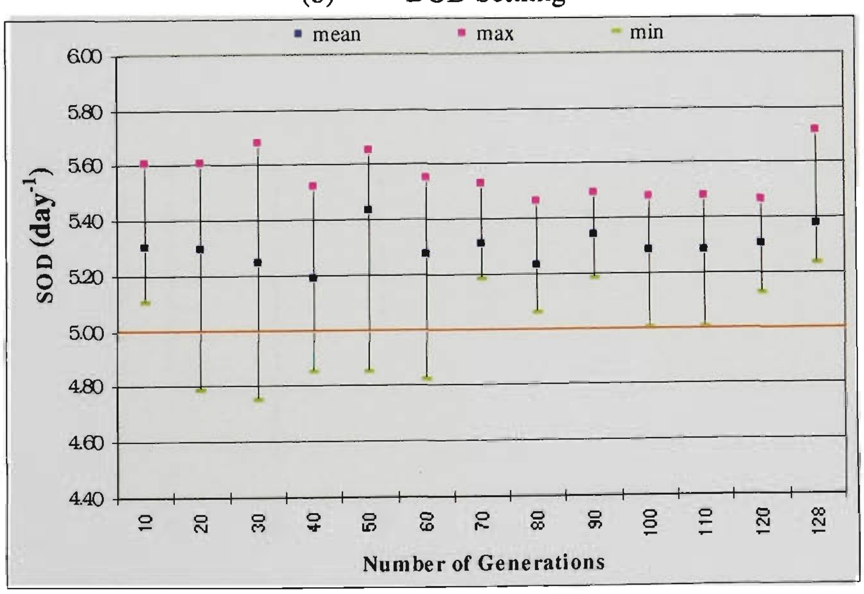
EFFECT OF PARAMETER POPULATION SIZE ON CONVERGENCE



(a) BOD Decay

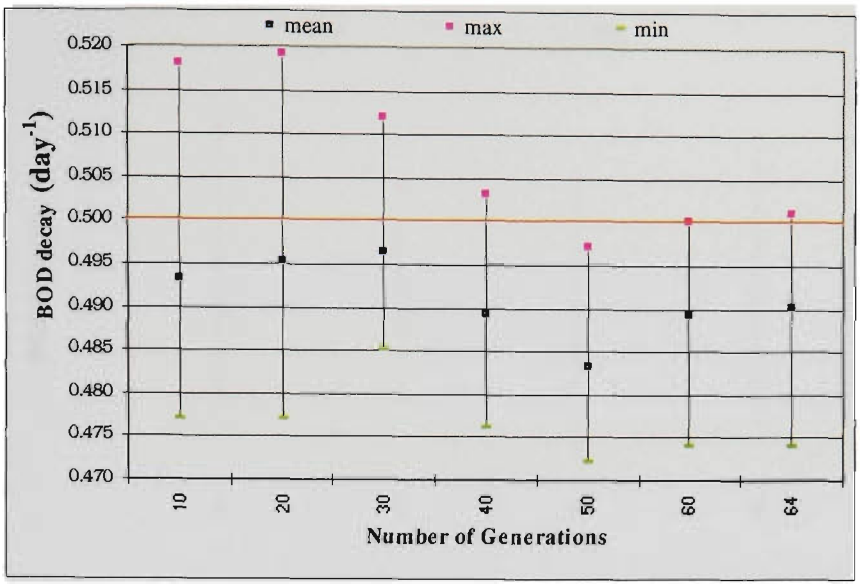


(b) BOD Settling

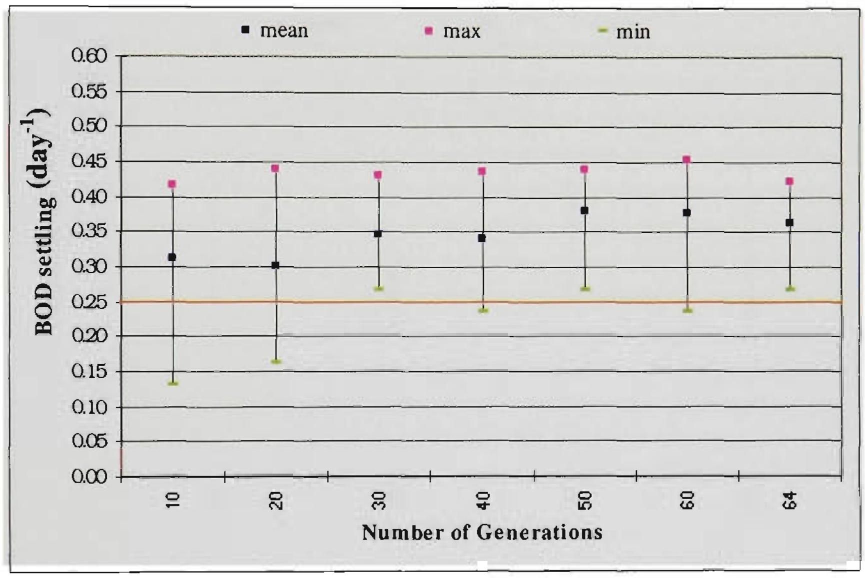


(c) SOD

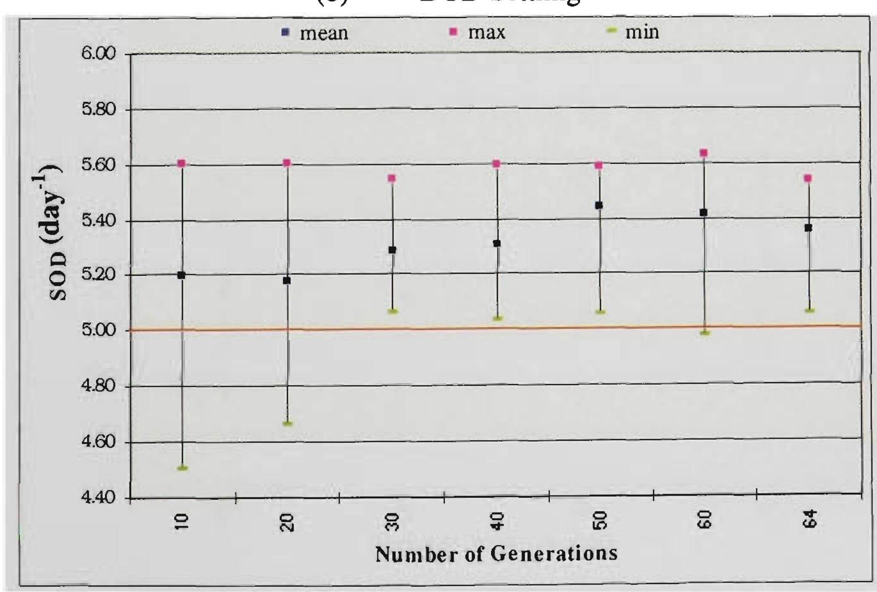
Figure C1 Convergence with Parameter Population Size of 250



(a) BOD Decay

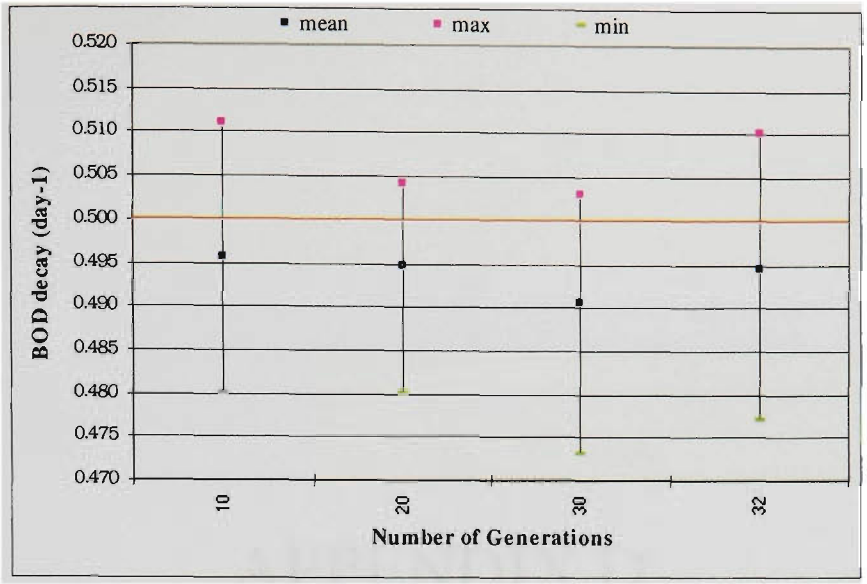


(b) BOD Settling

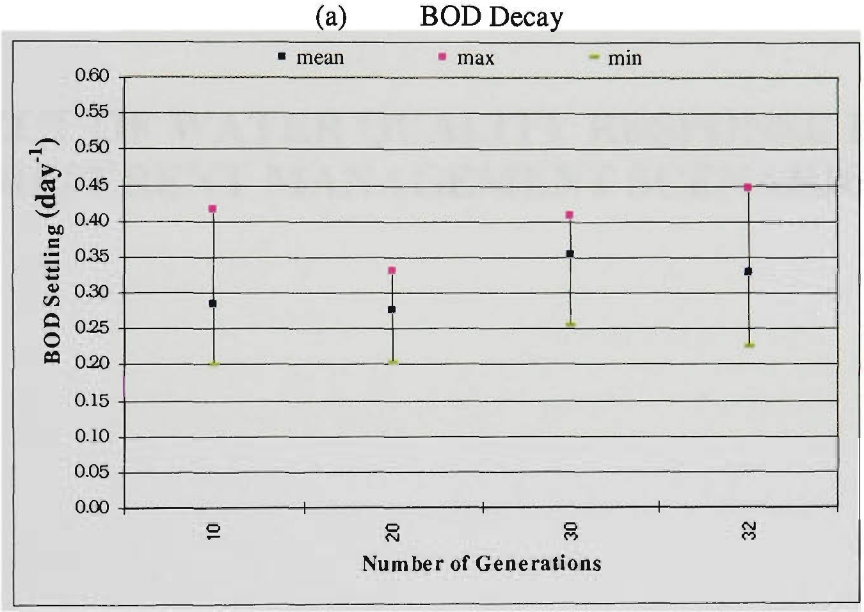


(c) SOD

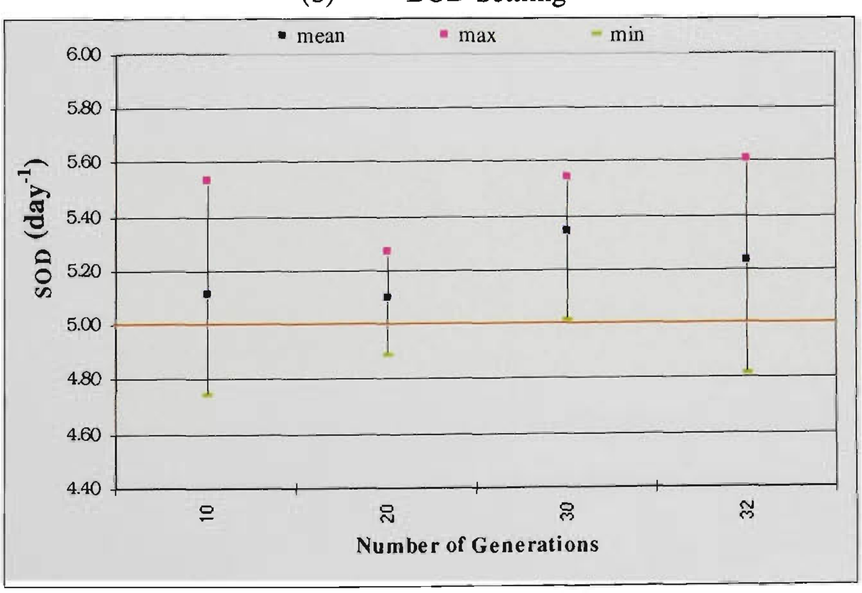
Figure C2 Convergence with Parameter Population Size of 500



(a) BOD Decay



(b) BOD Settling



(c) SOD

Figure C3 Convergence with Parameter Population Size of 1000

APPENDIX D

EFFECT OF WATER QUALITY RESPONSE FROM DIFFERENT MANAGEMENT SCENARIOS

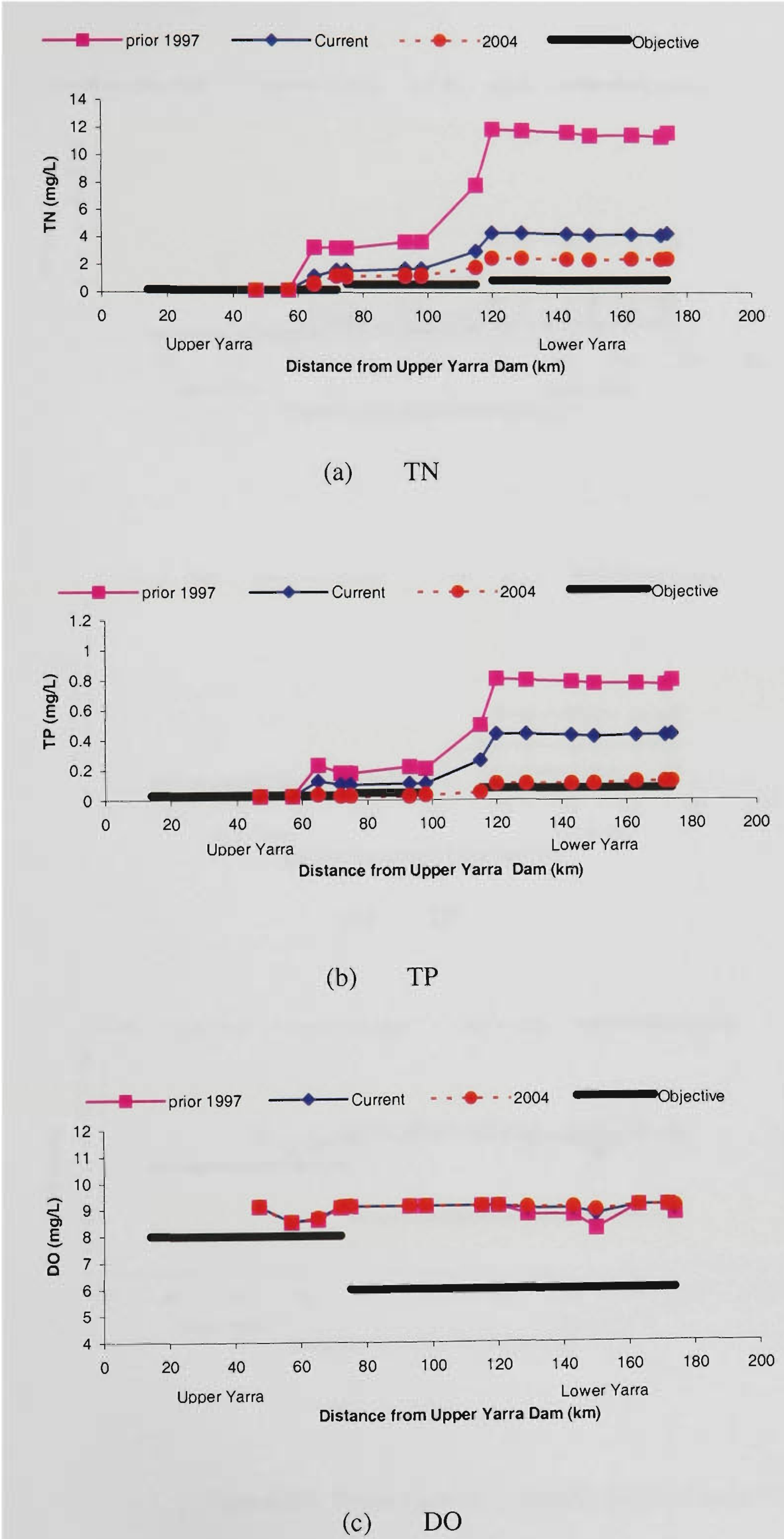
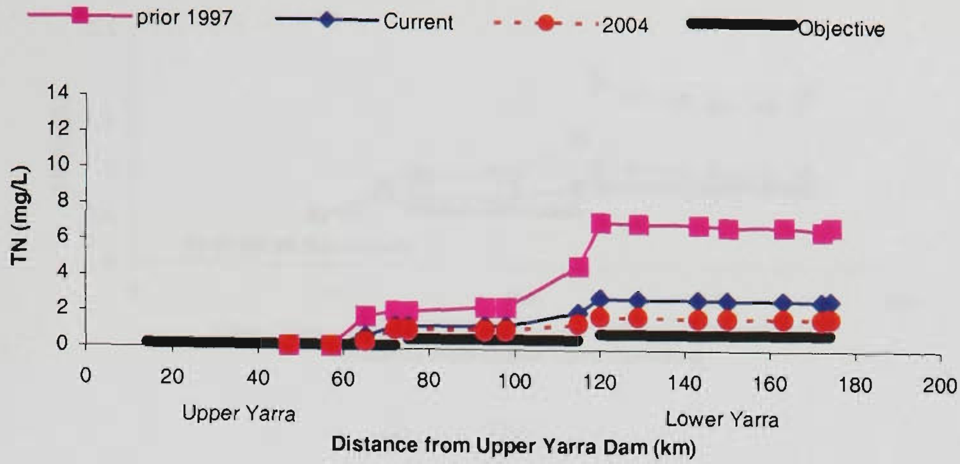
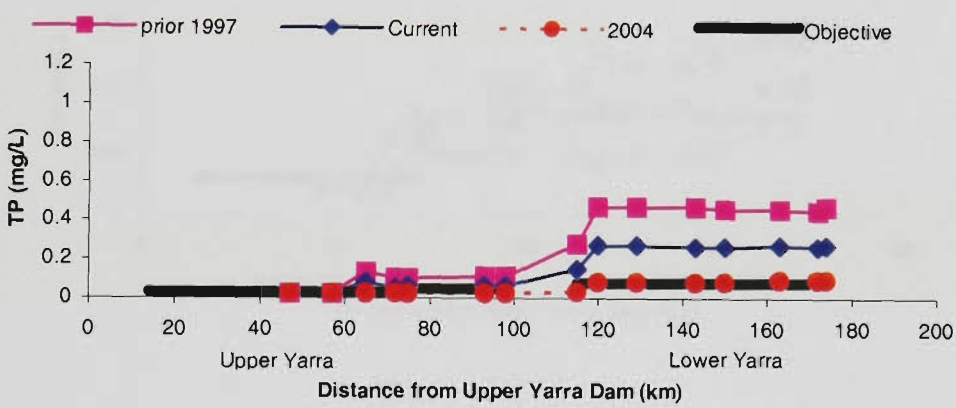


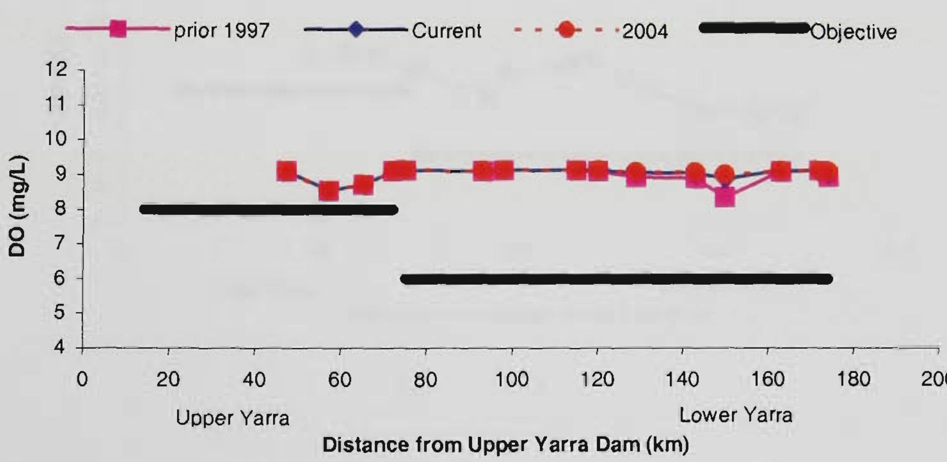
Figure D1 Water Quality Comparisons for Design Case B



(a) TN



(b) TP



(c) DO

Figure D2 Water Quality Comparisons for Design Case C

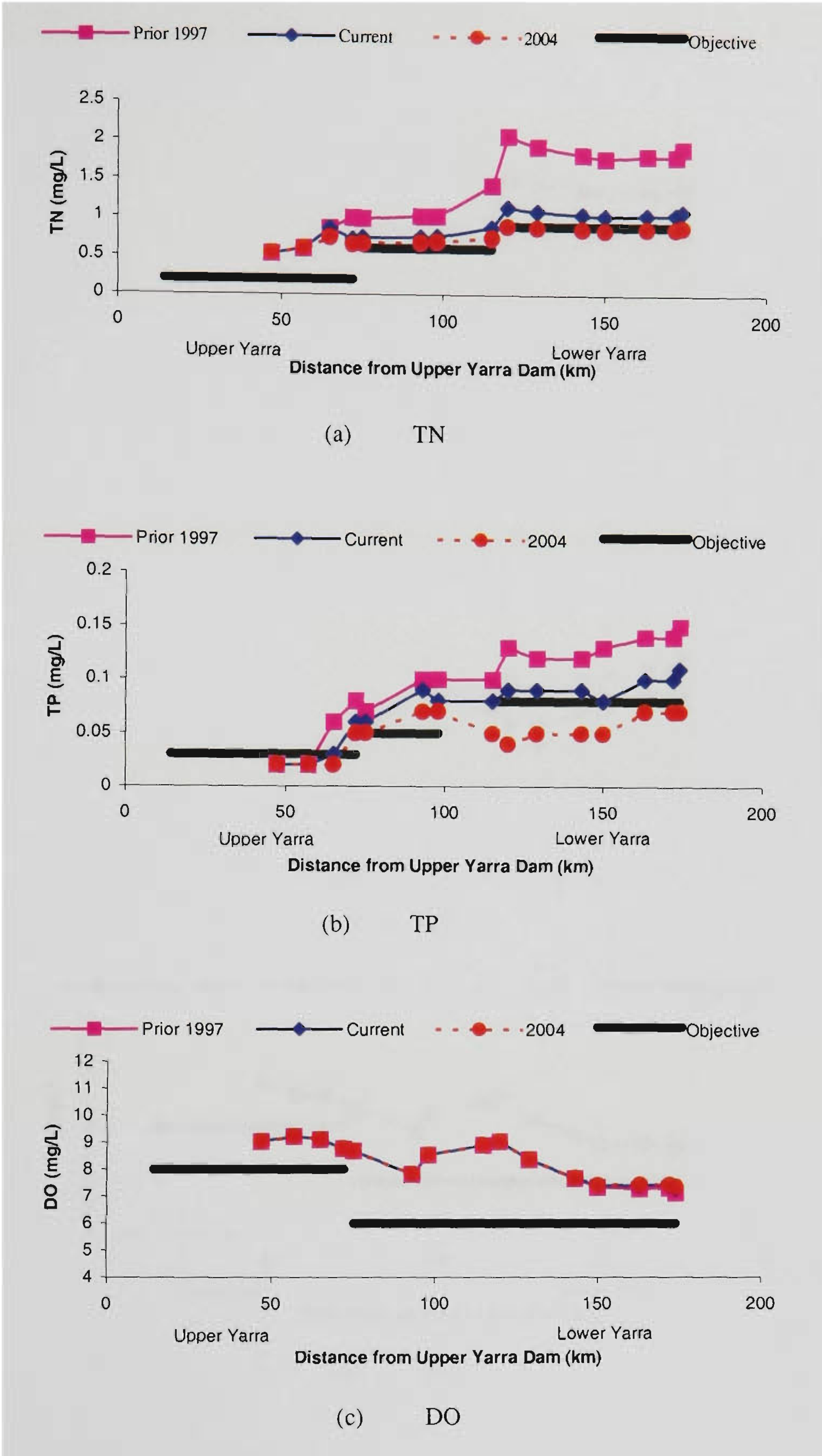
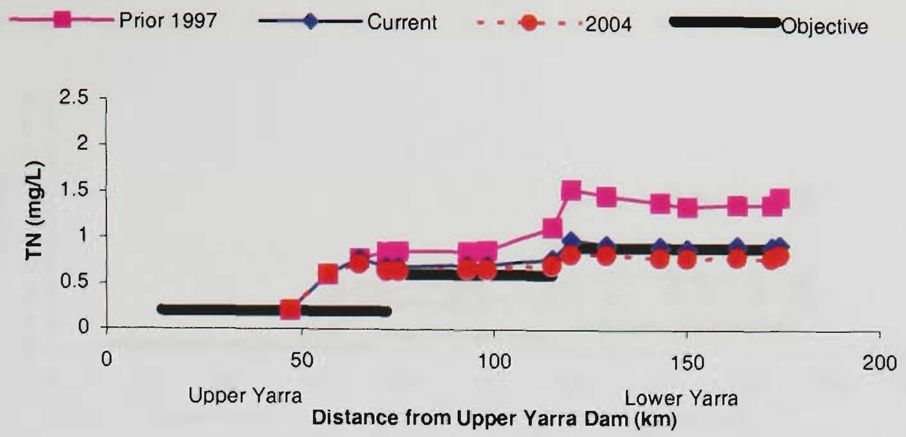
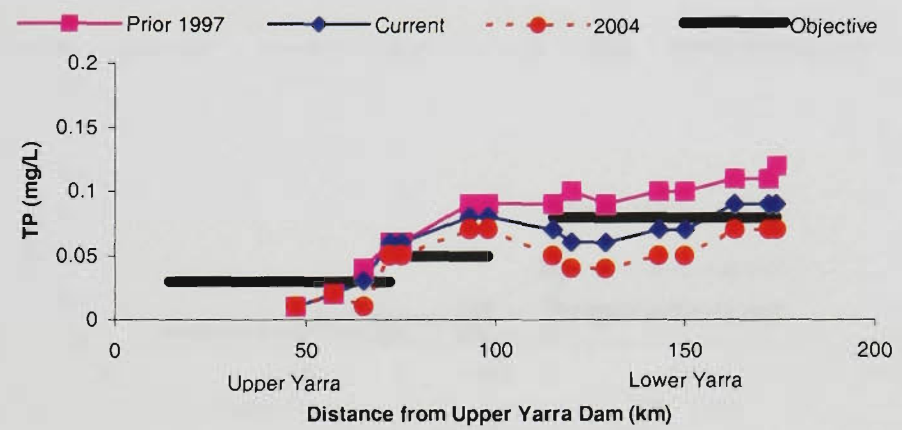


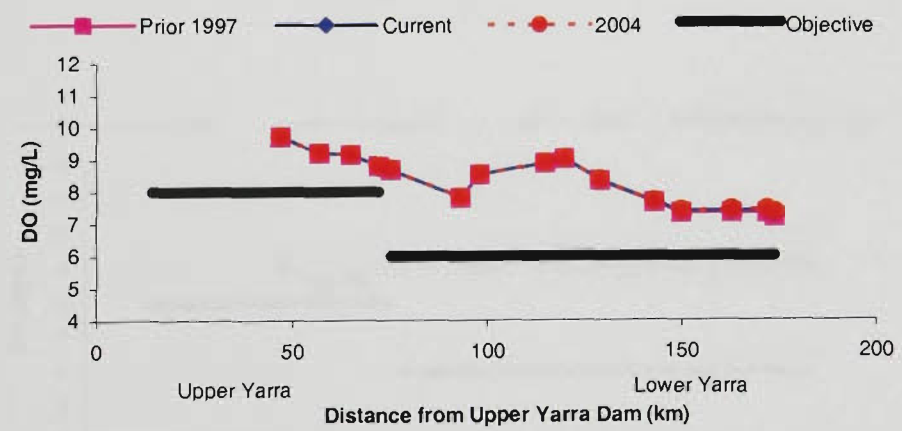
Figure D3 Water Quality Comparisons for Event 18/3/92 with 100% STP Discharge Volume



(a) TN

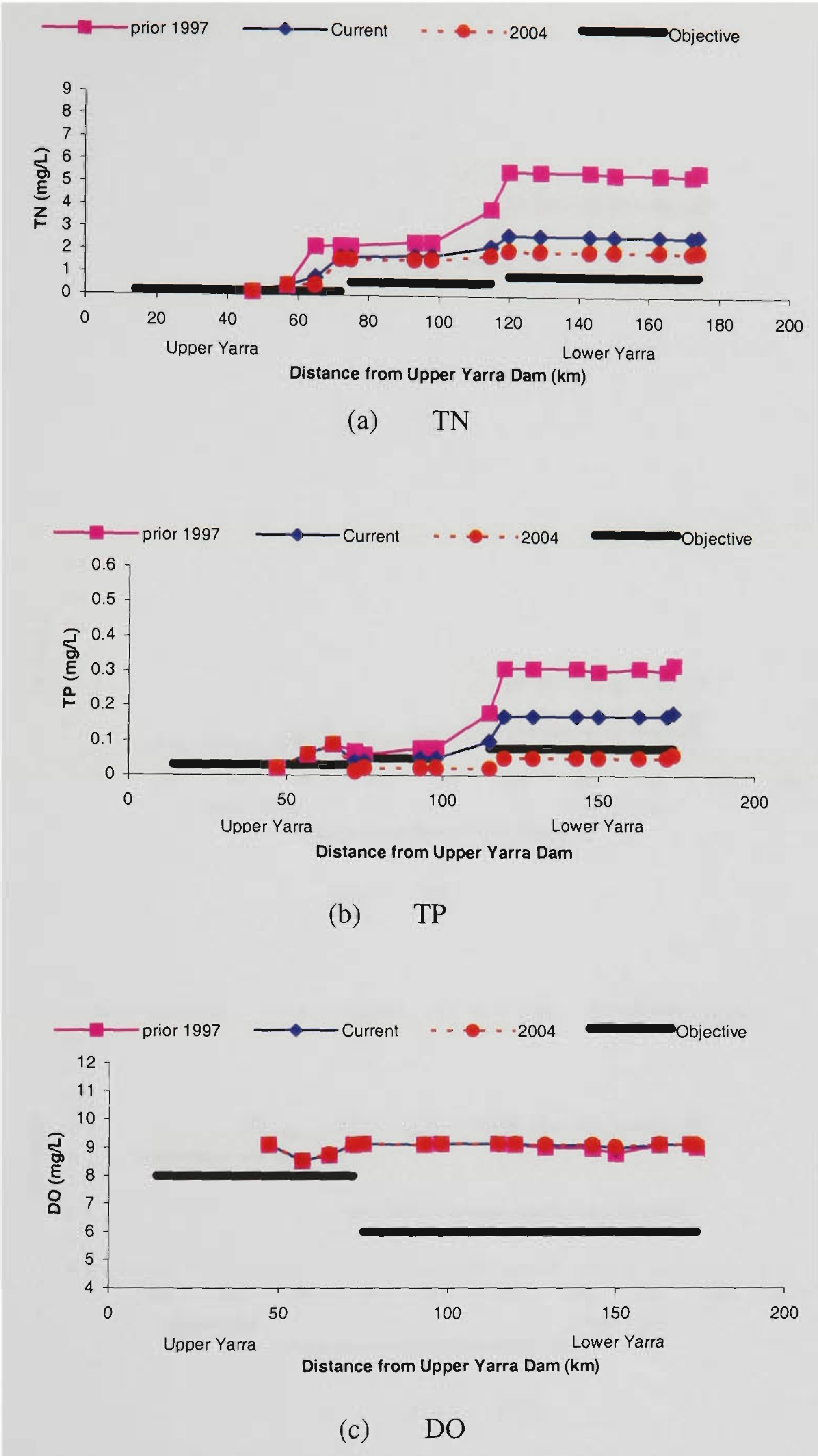


(b) TP



(c) DO

Figure D4 Water Quality Comparisons for Event 18/3/92 with 60% STP Discharge Volume



(a) TN

(b) TP

(c) DO

Figure D5 Water Quality Comparisons for Scenario D2 of Seasonal Discharge Program

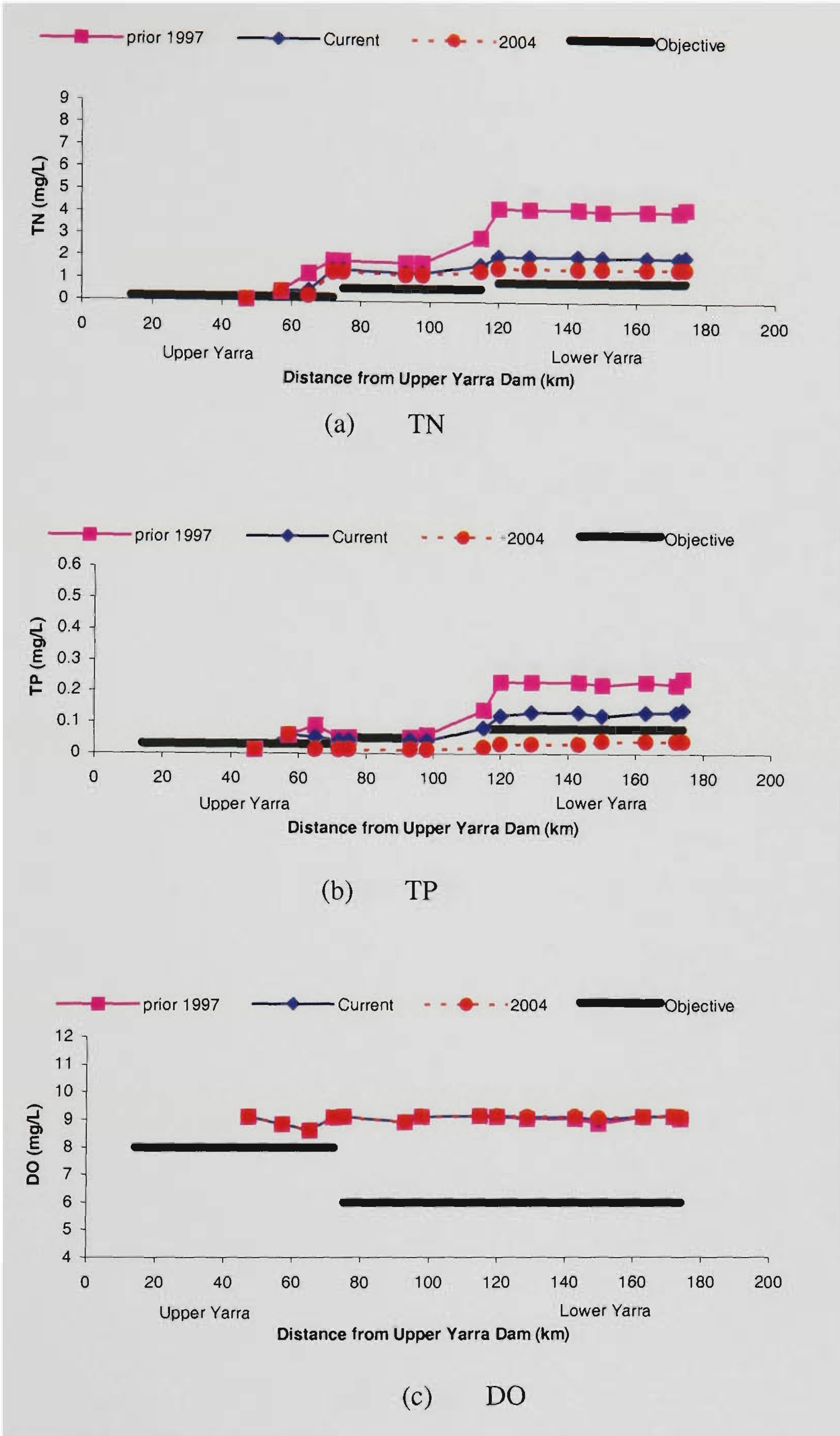


Figure D6 Water Quality Comparisons for Scenario W2 of Seasonal Discharge Program

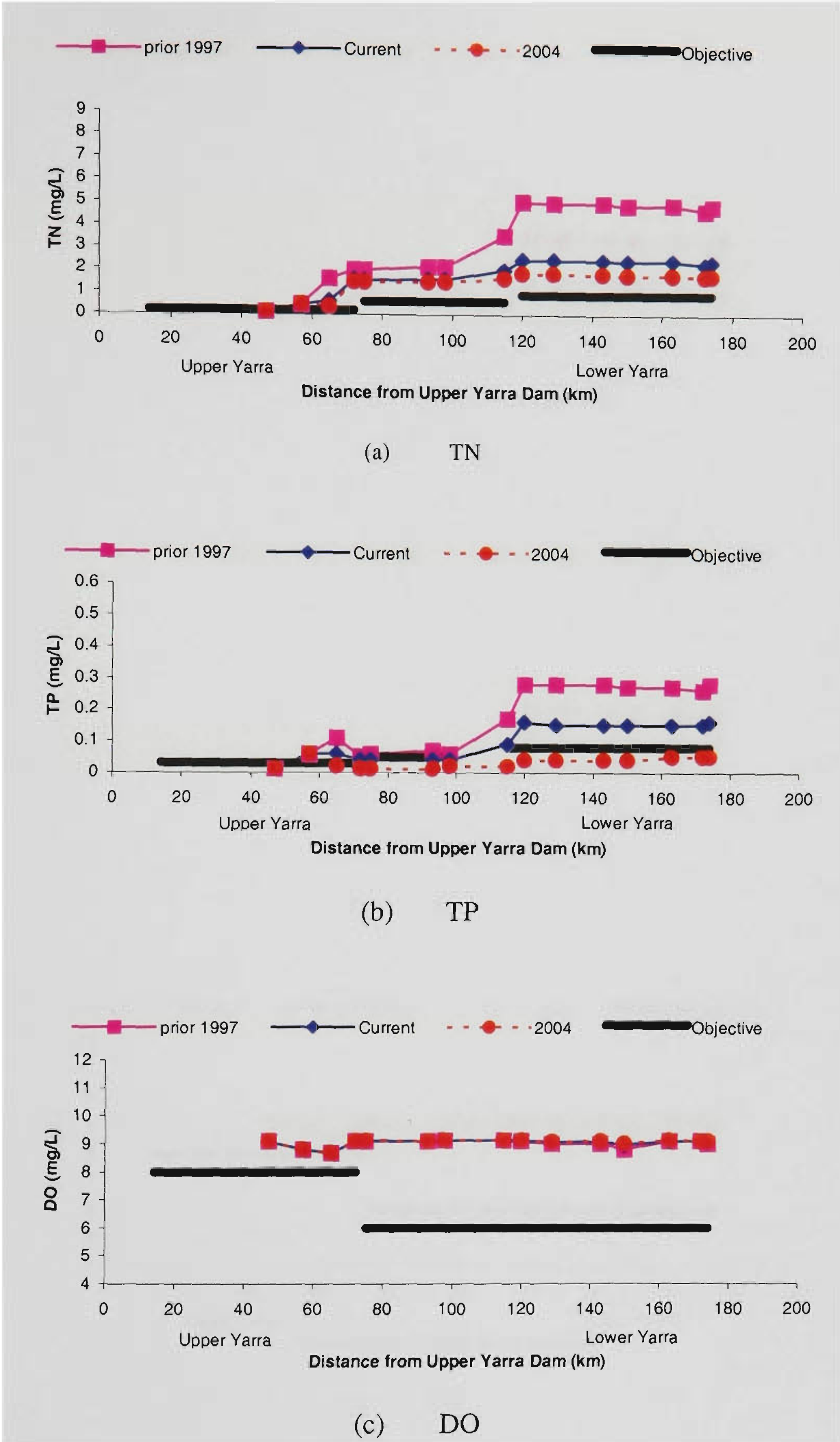


Figure D7 Water Quality Comparisons for Scenario D3 of Seasonal Discharge Program

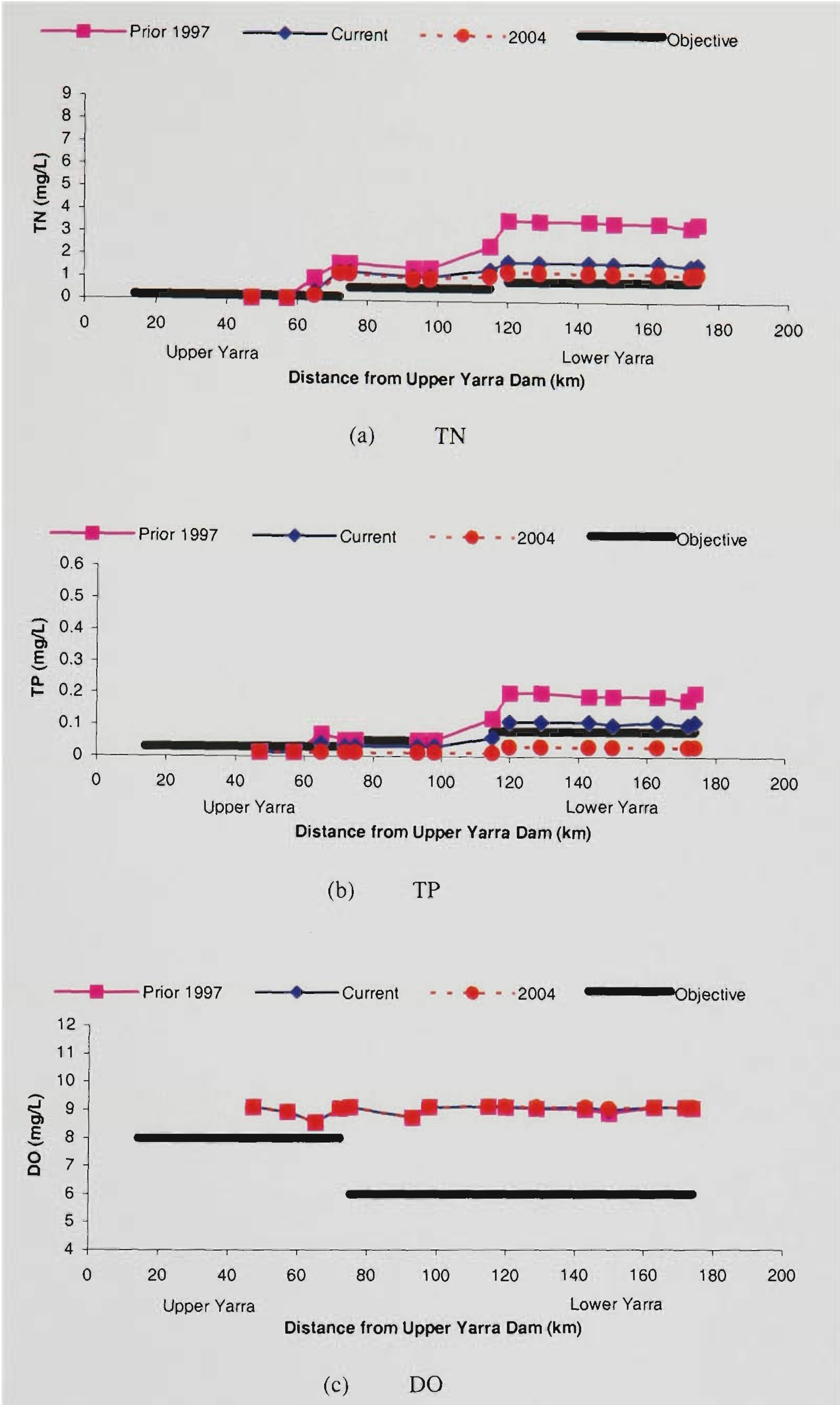


Figure D8 Water Quality Comparisons for Scenario W3 of Seasonal Discharge Program